

DESIGN AND CONSTRUCTION OF DEEP SECANT PILE SEEPAGE CUT-OFF WALLS UNDER THE ARAPUNI DAM IN NEW ZEALAND

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ABSTRACT

Arapuni Dam was completed in 1927 and is a 64m high curved concrete gravity structure across the Waikato River in New Zealand. A series of foundation leakage events related to piping and erosion of clay infill within joints in the rock foundation have occurred since the dam was built. Leakage was evidenced by increased drainage flows and uplift pressures.

The paper describes the design and construction features of the deep seepage cutoff walls that have recently been completed to control piping and erosion in the foundation. These include:

- selection and development of the cutoff wall solution
- construction through an existing dam
- construction with a full reservoir and the systems used to manage this risk
- assuring continuity in a 400mm diameter 90m deep secant pile 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.

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.

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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 (MRP) 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 was not affected and electricity generation 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.

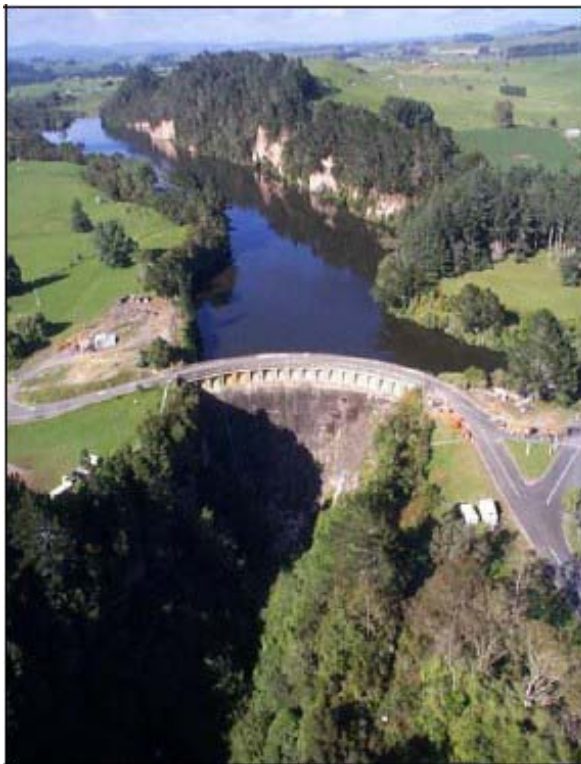


Figure 1. Arapuni Dam, New Zealand,

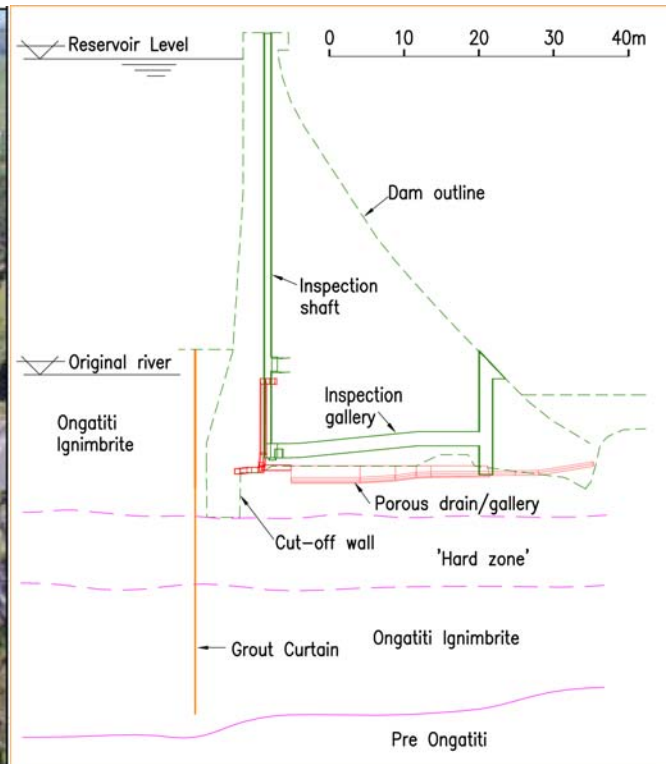


Figure 2. Cross Section of Arapuni Dam looking West Dam (Note the spatial separation of the grout curtain from the dam)

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 4), 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 4). 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.

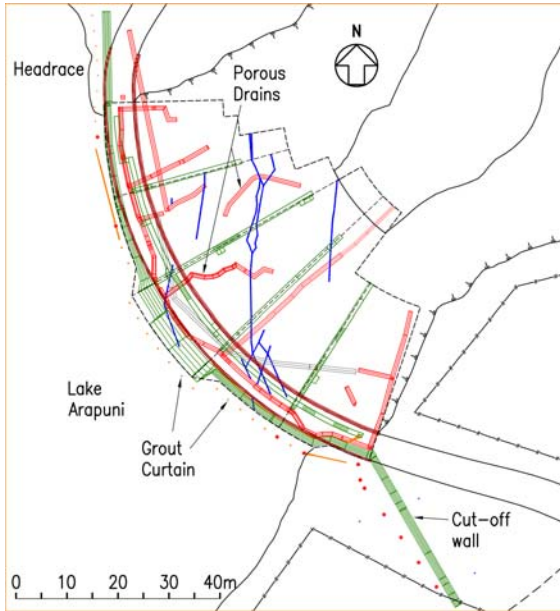


Figure 3. Plan view of Arapuni Dam (The positions of the foundation fractures noted during construction are shown. The diversion tunnel is south of the right abutment cut off wall)

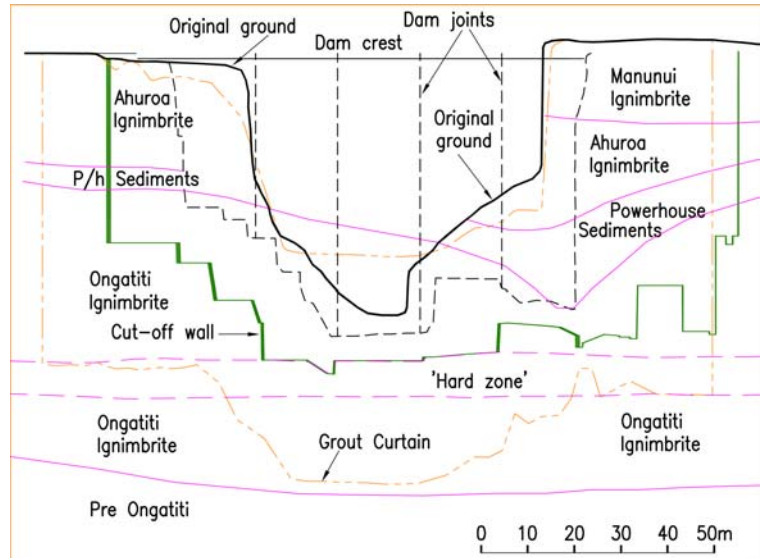


Figure 4. Elevation of Arapuni Dam, looking downstream

At interfaces between ignimbrite sheets there tends to be unwelded material, either airfall tephra or unwelded ignimbrite. The most extensive interface deposit is between the Ahuroa and Ongatiti ignimbrite units, known as the Powerhouse Sediments (Figure 4), 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, destabilizing 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.

Subsequent investigations also identified zones under the dam where fractures were open, i.e. without joint infill, and hydraulically connected to the lake. Hence near-lake pressures were present in areas under the dam, but the pressurised fractures did not have associated leaks at the downstream toe of the dam.

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 program 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 a North-South orientation (Figure 3) and a fourth set of fractures was identified in 2003 (Figure 5).

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. Most holes were 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 zones of vertical fractures, at a range of dip angles to provide data at the top, middle and bottom of the ignimbrite sheet. Holes extended past obvious fractures into rock with few and relatively minor joints to verify the lateral extent of fracture zones. Other holes were drilled from the abutments through the Ongatiti sheet. All core was logged by experienced engineering geologists and samples selected for unconfined compression and modulus tests. Drilling records and foundation piezometer readings during drilling were carefully correlated to identify hydraulic connections between areas of foundation.

The investigations clearly indicated the zones where vertical joints were present, and

hence the width of treatment panel could be determined with some confidence. 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 in the foundation rock

The investigations, in particular the piezometric responses to drilling, have supported the groundwater model developed in 2001, namely discrete south-north flow paths along parallel vertical fracture zones with little hydraulic conductivity between the parallel flow paths.

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.

MRP 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).

EARLY CONTRACTOR INVOLVEMENT TO FINALISE CUT-OFF METHOD

Prior to engaging the contractor, several methods were considered by the owner for installing the cut-off 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 (described in Amos et al., 2007);

- Vertical overlapping concrete piles drilled from the dam crest.

- “Waterknifing”, which targets the infilled joint with high pressure water, removing the clay and replacing it with a seam of grout.
- Combination of wire saw and high pressure jet grouting, which uses a combined process to cut a vertical slot from the dam crest (with the wire saw) and create vertical panels of grout under the dam (with jet grouting equipment).

Other methods, such as a diaphragm wall method using rock cutter equipment, were considered but rejected for a number of technical and safety reasons.

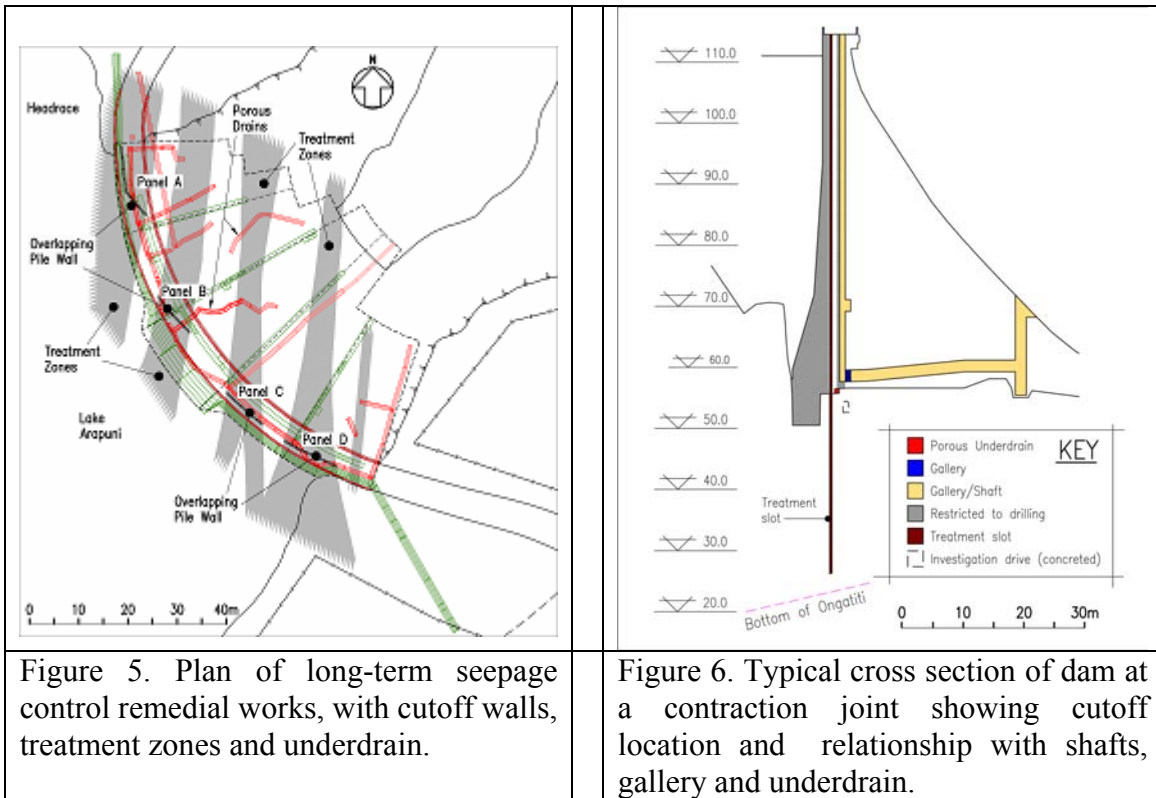


Figure 5. Plan of long-term seepage control remedial works, with cutoff walls, treatment zones and underdrain.

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

The three short-listed options all extended existing foundation engineering technology and practice and required thorough consideration of risks, both technological and dam safety impacts. The dam owner recognised the merits of early contractor involvement to develop the final methodology in association with the design team. An Alliance was selected as the best delivery means for construction of the cut-off walls.

The commercial framework for an 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 involving the contractor at an early stage and choosing an Alliance for construction delivery were:

- the clear need for contractor involvement in the selection and development of the preferred construction method;
- continuity of the body of knowledge from investigation through to completion;
- personnel selected on a best-for-project basis;
- allowance for subsequent modifications of methodology as the works progressed;
- the equitable sharing of construction and methodology risks in the execution of the work with a full reservoir, and
- to minimise the risk of contractual dispute.

Contractor involvement in the project followed three stages:

- Stage 1 was the selection of a 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 with the selection of the preferred option. For this stage the contractor was employed in a consultancy services 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 and the agreed project key performance indicators (KPI’s) for assessment of these incentive payments.

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

The client separately engaged the design consultant Damwatch Services Ltd of New Zealand 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. Contractual relationships are shown in Figure 7.

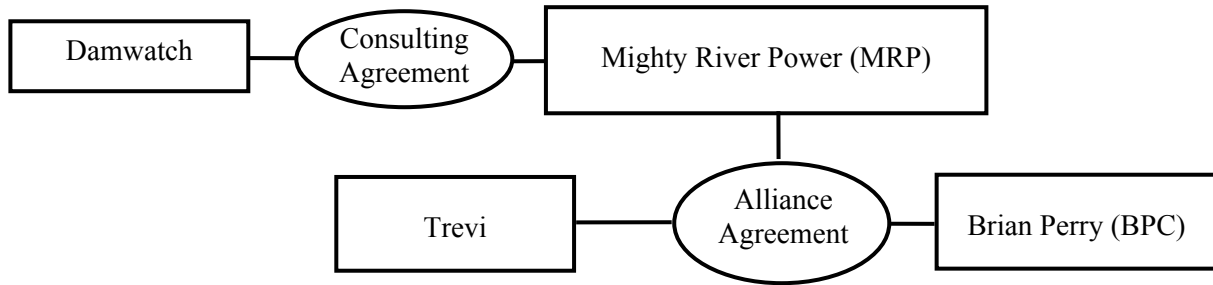


Figure 7. Contractual Relationships

OVERLAPPING PILE CUT-OFF

A comparison by the project team 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.

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;
- The contractor proposed to fabricate equipment that would physically link the hole being drilled to the previous hole, thereby resulting in a panel which must be a continuous cut-off if successfully constructed;
- 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 notable recent diaphragm wall dam foundation project with full reservoir has been completed at Walter F George Dam in Alabama (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., 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 8) to form the required overlap. The holes were drilled from the dam crest (i.e. above reservoir level) to minimize 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 11,600m.

Each of the four cutoff walls was constructed in discrete segments or slots (Figure 8) 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.

A slot would be completed and backfilled with concrete before progressing to the next slot.

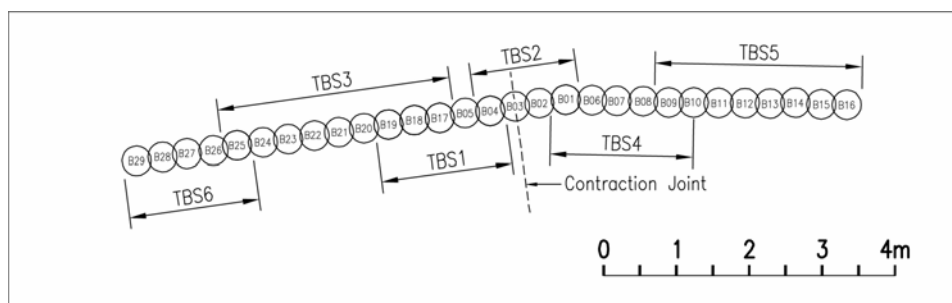


Figure 8. Plan of Treatment Panel B showing Slot sequence

CONSTRUCTION THROUGH THE EXISTING DAM

The minimum thickness of unreinforced dam concrete between a slot and the upstream face of the dam is 1.2m. The potential tensile and shear stresses in this cover concrete caused by unequal internal and external lateral pressures must be minimized to prevent damage to the upstream face of the dam. Ideally, fluid pressure in the slot would be maintained equal to lake pressure. However tensile stresses will occur in the face

concrete if water is lost from a slot (net external pressure) and during slot backfilling with concrete (net internal (bursting) pressure). The main risk occurs in the top 22m below dam crest level, while beneath this level the slot cover concrete thickens as the dam face flares upstream.

The cover concrete is also subject to other loading conditions during construction that cause tension in the surrounding dam concrete, such as the cooling effect of slot water on surrounding dam concrete. These stresses are largely independent of slot length but act in combination with the water and backfill pressure loadings.

Detailed 2-D finite element analysis of the horizontal section of a typical 8 hole slot and upstream concrete 1.2m from the dam face was carried out for load combinations of:

- water in the slots that is colder than adjacent dam concrete (seasonal lag of dam temperature behind water temperature change)
- water pressure on the end of slots during drilling
- external lake pressure from dewatered slot
- net backfill bursting pressure
- heat of hydration from backfill concrete

A 2.4m maximum slot length for 1.2m cover was set based on a maximum allowable tensile stress at the slot ends of 1.0MPa. The allowable stress criterion was based on a Factor of Safety against cracking of 2.5 for an average direct tensile concrete strength of 2.5MPa in dam concrete (from test samples and tensile strength estimates based on compressive strength using Raphael (1984)).

The reverse circulation drilling method used to create slots required a priming reservoir chamber of pile holes 4.5m deep the full length of the panel to be created at the start of the project for temporary fluctuations in slot water during stages of drilling. When water filled this 10-15m length of upstream face concrete above lake level, a net outward (bursting) load was present on the upstream face of the dam. Vertical stressing rods were installed between the upstream face and slot 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 (Figure 9).

Restoration of the original structural integrity of the dam after completion of the cut-off panel is also important. The long term bond of backfill concrete or grout to the existing dam was assessed and a strength gain requirement specified for a completed slot before an adjacent slot could be drilled.

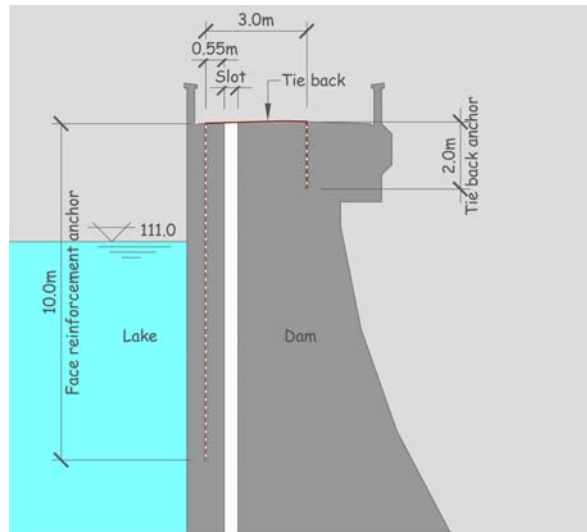


Figure 9. Reinforcement of Upstream Face

DRILLING OVERLAPPING PILES

Drilling accuracy was important to:

- avoid obstacles within the dam (such as drains and galleries),
- reduce opportunities for the guide system to jam, and
- to achieve the target cutoff area in the foundation rock.

Computer controlled directional drilling was used to establish an initial pilot hole at the start of each panel. This 150mm diameter hole was then reamed to 400mm diameter, thereby providing the straightest starting pile possible.

A rotary tricone drill bit with reverse circulation flushing was the preferred drilling technology for the 400mm diameter piles. 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 in soft and fractured rock. Reverse circulation to flush cuttings up the inside of the drill rods reduces the risk of drilling fluids eroding clay infill in the dam foundation.

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). Vertical cored investigation holes on the panel alignment were used to confirm the level of the Pre-Ongatiti interface.

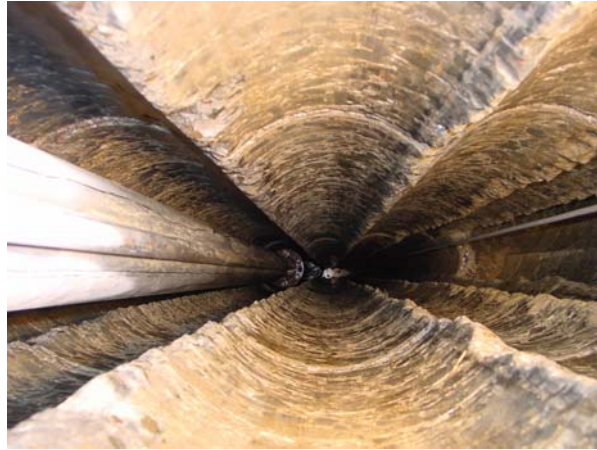


Figure 10. Downhole photograph of overlapping drill holes

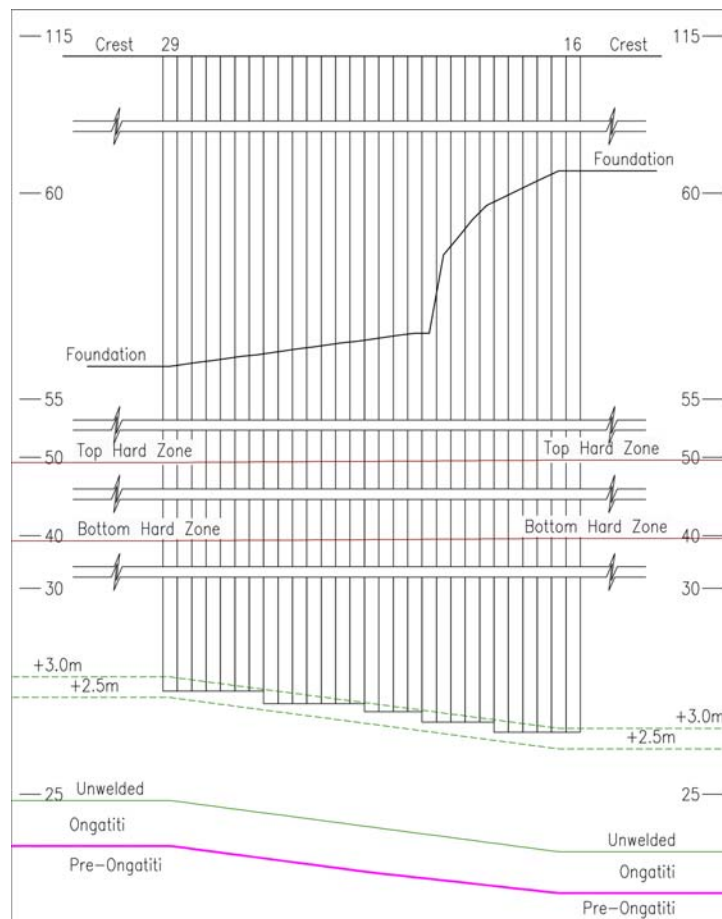


Figure 11. Typical Elevation of a Treatment Panel (Panel B)

While the main drill bit did not have directional controlling equipment, hole drift was minimised by real-time monitoring of drilling parameters and management of drill-bit rotation direction. An inclinometer was lowered down the drill rods at regular intervals

for hole orientation checks. Hole drift was usually small within dam concrete, tending to be greater in the weaker ignimbrite beneath the dam. Excessive hole drift was countered by drilling a slot from the other direction and where slots did not connect a heavy steel chisel would be used to remove offending slivers of rock and achieve continuity along the slot.

Continuity with the next slot was achieved by installing a 150mm diameter PVC pipe, centralized in the end hole of the open slot, just prior to concreting. This pipe was filled with weak bentonite-cement grout to improve rigidity during the concrete pour. At commencement of drilling for the adjacent slot, the PVC pipe was used as a pilot hole for a reaming tool to open the end hole back up to a 400mm diameter open hole. Alternatives similar in concept to diaphragm wall stop ends were trialled briefly but a reliable system to replace the PVC pilot pipe was not found.

SLOT BACKFILL

Concrete backfill was placed in each slot in a tremie operation after removal of all debris from the slot and final verification of slot continuity. Concrete was discharged direct from ready-mix truck into a tremie pipe located in the centre of the open slot. The concrete mix used was based on:

- Maximum aggregate size of 10mm
- Water cement ratio of 0.45
- Slump of 220mm
- Target compressive strength of 30 MPa
- Retarder admixture for 1 hour transport to site
- Water reducing agent and superplasticiser for flow

Concrete sample 28 day strength tests were in the range of 48 to 50 MPa. Post construction coring and compression tests gave similar strength results.

The rate of concrete rise was strictly controlled to reduce tensile stresses in the upstream face of the upper part of the dam due to lateral pressure from fresh concrete. The rapid concrete installation in rock paused at around 10m above foundation interface (i.e. concrete surface had risen inside the dam body) in order to lower the slot water level and hold it at 15m below dam crest level. Concreting continued at the same rate before terminating at approximately 15m below crest level. Any remaining slot water was then pumped out and after a time delay to allow initial set of the fluid concrete below, concreting continued in a dry slot at a rate of rise of approximately 2m/hr up to 4.5m below dam crest. The thermal stress state of the dam body at the time of the concrete pour was a key parameter for setting the controlled rate of concrete rise used on the day. The top 4.5m was backfilled at a controlled rate on completion of all slots.

DAM SAFETY DURING CONSTRUCTION

Because cut-off 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.

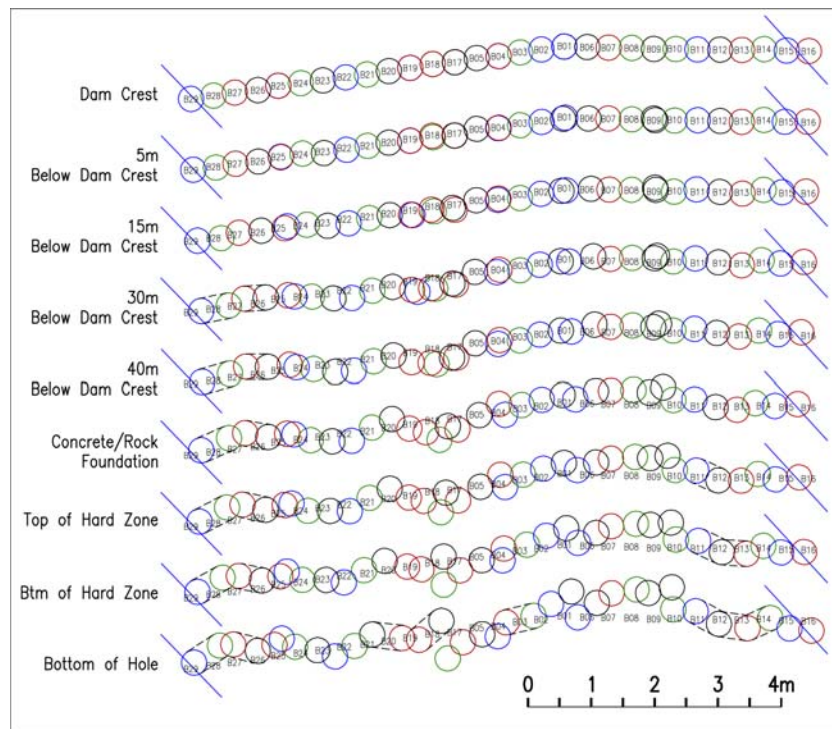


Figure 12. Elevation of Treatment Panel B, showing the relationship between inclinometer readings at depth for each drill hole

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.

RESULTS

Foundation drilling and panel construction was completed in September 2007. In terms of the verification criteria above:

1. The completed panel geometry covers the treatment area requirements.
2. Cut-off quality meets or exceeds the high standards set in specification.
3. Flow through the former high pressure fissure zones into the downstream area beneath the dam decreased by approximately 90%, while pressures downstream of panels in the high pressure fissure zones decreased by approximately 14m. Pressures and flows from areas where clay infill was present (i.e. no high pressure) have experienced moderate decreases in pressure.

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.

In order to prevent future leakage incidents from occurring in the foundation of Arapuni Dam, four concrete cut-off walls have been constructed through the ignimbrite sheet underlying the dam, while the reservoir remained in service. An innovative foundation treatment solution that significantly extends international overlapping/secant pile technology and experience was used. The project design and remedial works were reviewed by independent international specialists to ensure that the dam met internationally recognised dam safety standards.

The cut-off walls consist of 90m deep overlapping 400mm diameter holes drilled through the dam and underlying ignimbrite sheet before being backfilled with concrete. Construction practices had to be carefully managed where they could affect the integrity of the upstream face of the dam.

Construction was undertaken with close monitoring of the dam foundation to ensure that the construction activities did not generate another leak requiring emergency action.

The construction works were successfully completed with:

- no damage to the concrete dam,
- no dam safety incidents requiring intervention,
- no impact on power station operation, and
- at the end of the project the dam's underdrain system remains serviceable.

The outcome is the formation of four robust and verifiable cut-off walls beneath Arapuni Dam that will reduce the risk of future foundation leaks. The collaborative design process and use of the alliance procurement model delivered a mechanism for problem solving, equitable risk share and reward for successful completion of the project.

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