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

A series of foundation leakage events have occurred at Arapuni Dam in New Zealand from the time water was first impounded in 1927. Past foundation leakage incidents at the 64m (210ft) high curved concrete gravity dam were related to erosion and piping of weak clay infilling joints within the volcanic ignimbrite foundation bedrock. Seepage changes often involved sudden and significant increases, and could not usually be related to external events, such as earthquakes. The most recent seepage incident developed from 1995 and required grouting (completed in December 2001) to fill an open void within a foundation joint. With the deteriorating condition arrested, the owner of the dam, Mighty River Power Ltd., decided that a high quality and verifiable cut-off solution was to be constructed while the reservoir remained in service.

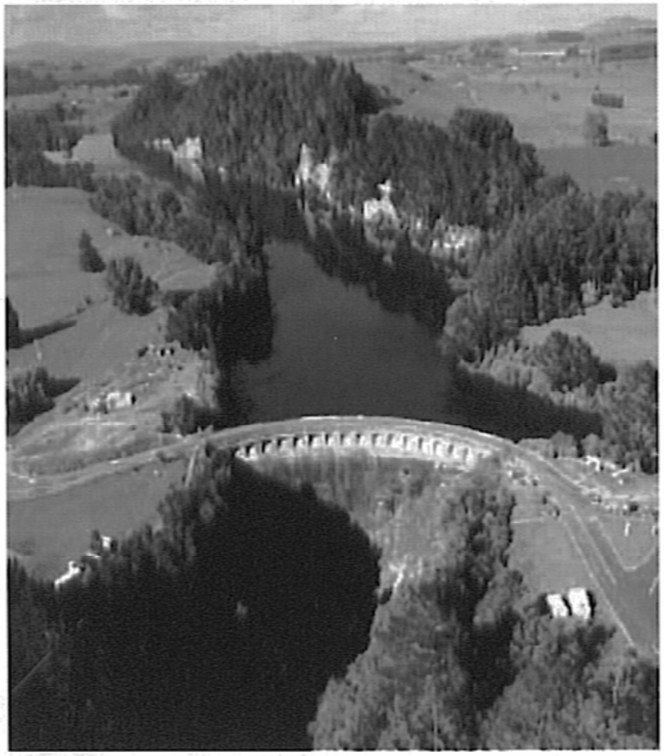

Figure 1. Arapuni Dam, New Zealand, looking West

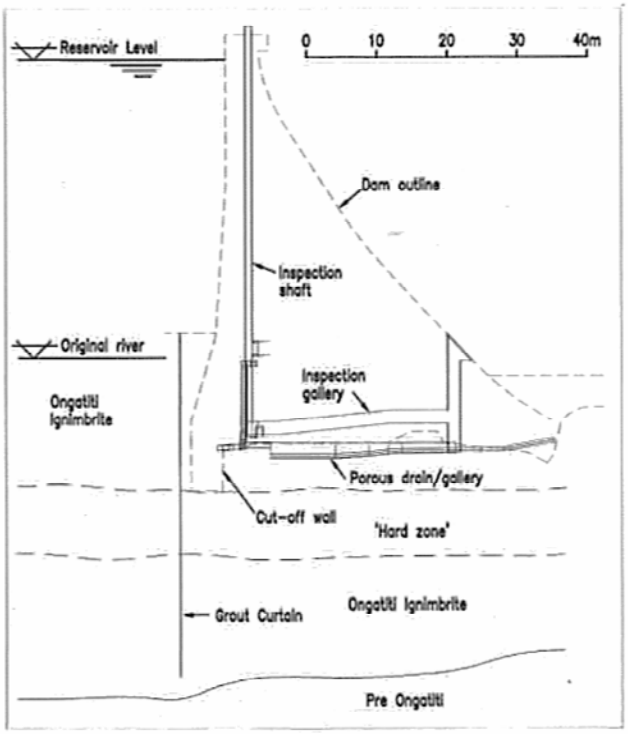

Figure 2. Cross Section of Arapuni Dam (Note the spatial separation of the grout curtain from the dam)

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A comprehensive investigation program was completed to determine the extent of foundation features requiring treatment to prevent further incidents from developing. Four zones beneath the dam were identified as requiring permanent cut-off walls.

An international Alliance between the dam owner (assisted by their designer) and a consortium of specialist foundation contractors was formed to identify cut-off options, develop them and implement the selected methodology. A cost-effective preferred approach was selected involving drilling and concreting overlapping vertical piles from the dam crest through the dam and underlying rock formation. The overlapping piles extend to a total depth of 90m (295ft) to form four separate cut-off walls. Construction of the cut-off walls commenced in September 2005 and was completed in September 2007. Operation of the reservoir was not affected and electricity generation continued throughout the project works.

The project successfully formed a robust and verifiable cut-off wall remediation. With few precedents for this type of work, and none constructed in weak rock or to 90m (295ft) depth, the Arapuni Dam seepage cut-off project significantly extends international small-diameter overlapping/secant pile technology and experience.

THE DAM AND ITS FOUNDATION

The 64m (210ft) high Arapuni Dam (Figures 1 and 2) is a curved concrete gravity structure of crest length 94m (308ft), sited on the Waikato River in the central North Island of New Zealand. The dam forms the reservoir for a 186 MW hydro-electric power station, sited 1 km downstream at the end of a headrace channel extending from the dam's left abutment. The spillway is also located on the headrace channel.

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 original cut-off walls extend beneath the dam to a depth of 65m (210ft) below the dam crest and extend 20m (66ft) and 33m (108ft) into the left and right abutments respectively, for the full height of the dam as shown on Figures 3 and 4.

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 (130-165ft) thick sheet of Ongatiti Ignimbrite (Figure 3), a point-welded tuff. The upper part of the unit is very weak at 2 to 6 MPa (300-800psi), while below the original dam cut-off wall the Ongatiti is considerably stronger (up to 28MPa (4,000psi) (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 (3in). Clay infill is generally present where the cooling fracture opened around the time of ignimbrite 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 in the presence of seepage flow. Where infill was not present in fractures, seepage pressures correlating to reservoir level were present in some areas of open joints under the dam.

At interfaces between ignimbrite sheets there is usually 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 4), with a thickness of 4 to 8m (13-26ft). There was genuine concern during the 2001 incident 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.

The 600mm high x 600mm wide (2ft x 2ft) "no-fines concrete" porous drain network (Figures 2 and 3) is the main uplift control at the dam/foundation interface. The underdrain includes a continuous drain sited parallel to, and immediately downstream of, the original cut-off wall with radial porous drains to discharge seepage water to the downstream toe.

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 cut-off walls. The grout curtain was a single row cement curtain with mostly vertical grout holes at 3m (10ft) centres constructed just upstream of the dam and cut-off walls, but the curtain is not physically connected to the dam (Figure 2). Where grout holes intersected open voids in vertical joints, grout would have sealed that void, and leakage at the dam was considerably less once the reservoir was refilled in 1932. However this grouting operation would not have intersected all vertical rock joints or displaced clay infill in joints, hence future leakage incidents were always likely to occur.

THE 2001 SEEPAGE INCIDENT

The most recent seepage incident occurred following the drilling in 1995 of two drainage holes into a fissure under the dam, when drilling unexpectedly intersected an area of fissure that was open and connected to the reservoir. Discharge from these drains was approximately 600 litres/minute (160 gallons/minute) at near-lake pressure, but remained relatively stable for the next five years. The situation then began to deteriorate with increasing pressure and drain discharge

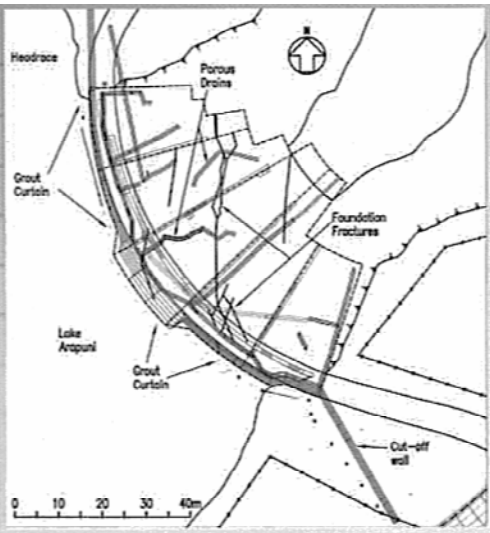


Figure 3. Plan view of Arapuni Dam (The positions of the foundation fractures noted during dam construction are shown.

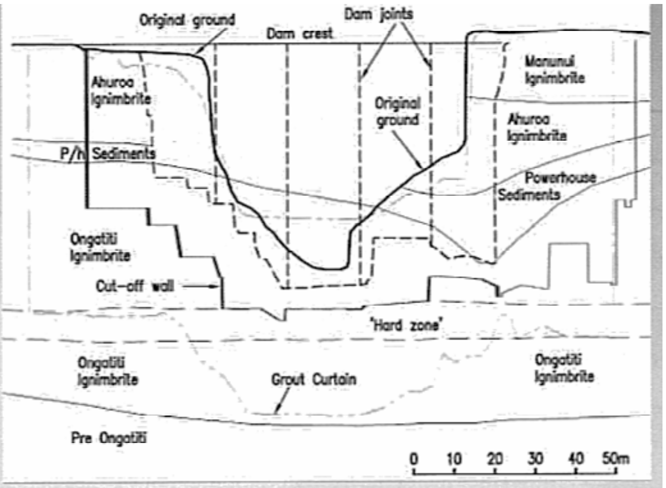


Figure 4. Elevation of Arapuni Dam, looking downstream

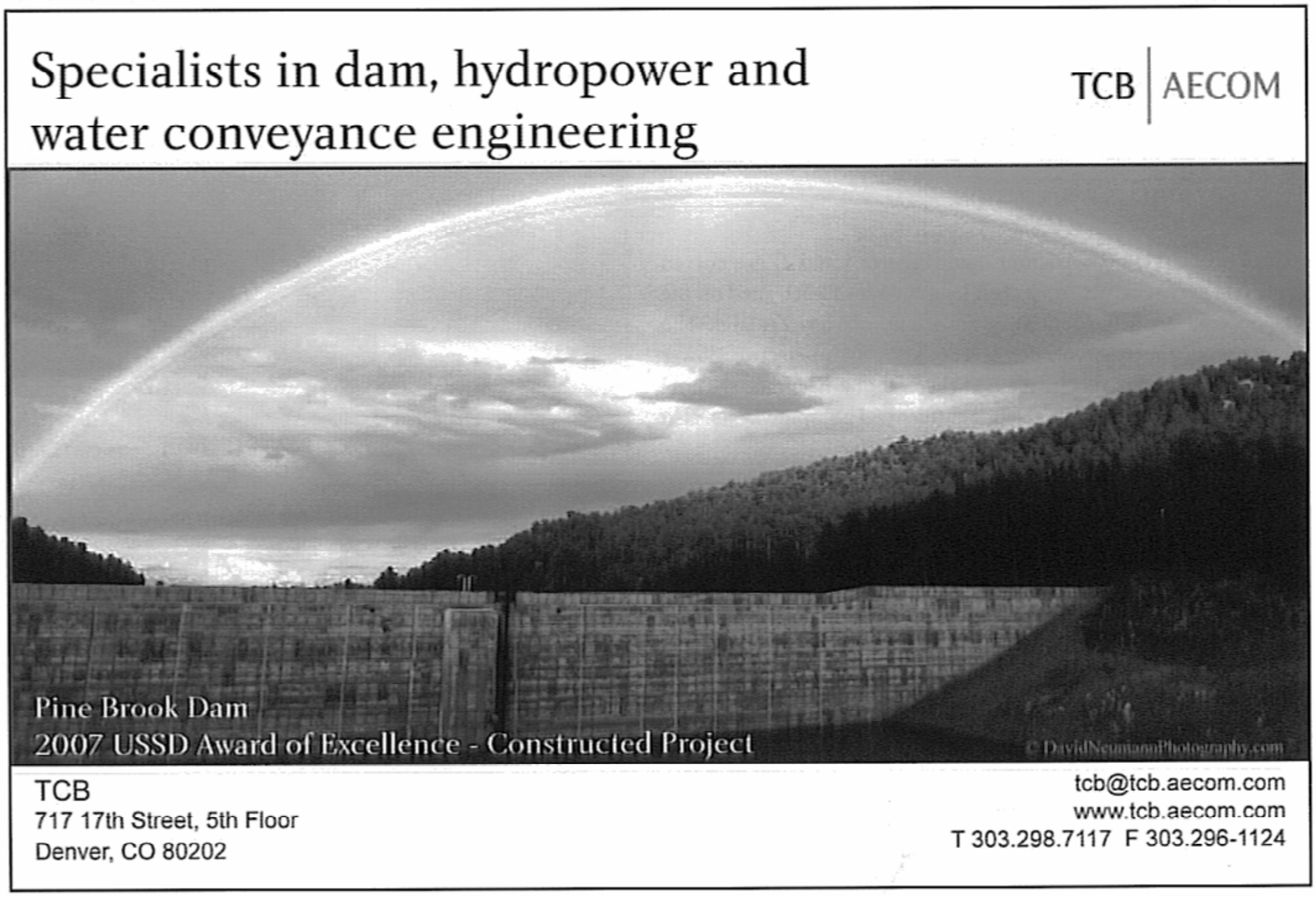
trends together with clay, bitumen and lake biota (including small fish and snails) being observed exiting the drains. A grouting program was carried out in December 2001 that successfully filled the void in the fissure under the dam (Amos et al, 2003).

FOUNDATION INVESTIGATIONS TO DETERMINE SCOPE OF CUT-OFF WORKS

An extensive programme of investigation core drilling and detailed foundation mapping was carried out between 2000 and 2005 to determine the extent and nature of the fissure systems. A total of 122 cored investigation holes were drilled from the downstream face of the dam or from inside the dam galleries, generally angling perpendicularly across the north-south trending fracture system. The investigations clearly identified four zones where vertical rock fractures were present, and hence the width of treatment panels targeting each zone could be determined. There were few joints in the areas between the four obvious fracture zones and also none of the orthogonal joints commonly seen in ignimbrites in this area.

PRINCIPLES FOR REMEDIAL WORKS

With these investigation findings, Mighty River Power committed to upgrading the dam so that the risk of further foundation piping



incidents would become extremely low and high pressures under the dam would be controlled. Furthermore, the objective was to complete remediation with no interruption to power station operations. The reservoir remained at normal operating levels to avoid the environmental effects of mobilizing downstream lake bed sediment, and to maintain continuous electricity generation.

The concrete cut-offs were to be located in the fracture zones as far upstream as possible to restore the normally accepted uplift profile under the dam (Figures 5 and 6). Because the reservoir was to remain full throughout, dam safety was an important consideration in selection of the final remediation technique.

The length of each panel was to cover only the zones of vertically jointed Ongatiti Ignimbrite identified in the geological investigation, supported by piezometer responses indicating hydraulic connections in the foundation. Some foundation joints were identified between the treatment zones, but these were confirmed to be sufficiently minor that they would not present a future risk of a concentrated leak. Target depth for cut-off panels was just above the interface with the underlying Pre-ongatiti Ignimbrite unit to avoid disturbance of the unwelded sediments between the ignimbrite units (Figure 7).

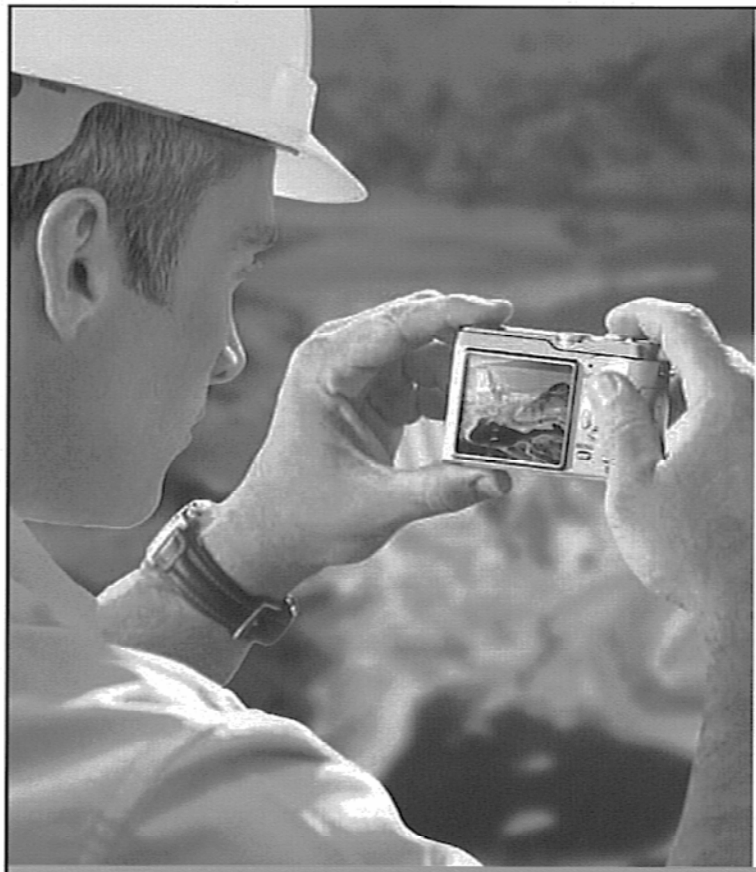
CONTRACTOR PROCUREMENT USING ALLIANCE FRAMEWORK

Prior to engaging the contractor, several methods were considered by the owner for installing the cut-off barrier. Three remedial options were identified utilising overlapping piles, diamond wire saws, or high pressure water and air jets to cut rock for further investigation (Amos et al. 2007). Other methods, such as using rock cutter diaphragm wall methods, were considered but rejected for various reasons such as the risk of damaging the dam.

The three feasible options all extended existing foundation engineering technology and the state of the practice. The decision process required thorough consideration of risks, both technological and to dam safety. Mighty River Power recognised the merits of early contractor involvement to develop the final methodology in association with the design team. An Alliance involving international specialist foundation contractors was selected as the best delivery means for the project.

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
- Equitable sharing of construction and methodology risks in the execution of the work with a full reservoir
- Minimise the risk of contractual dispute



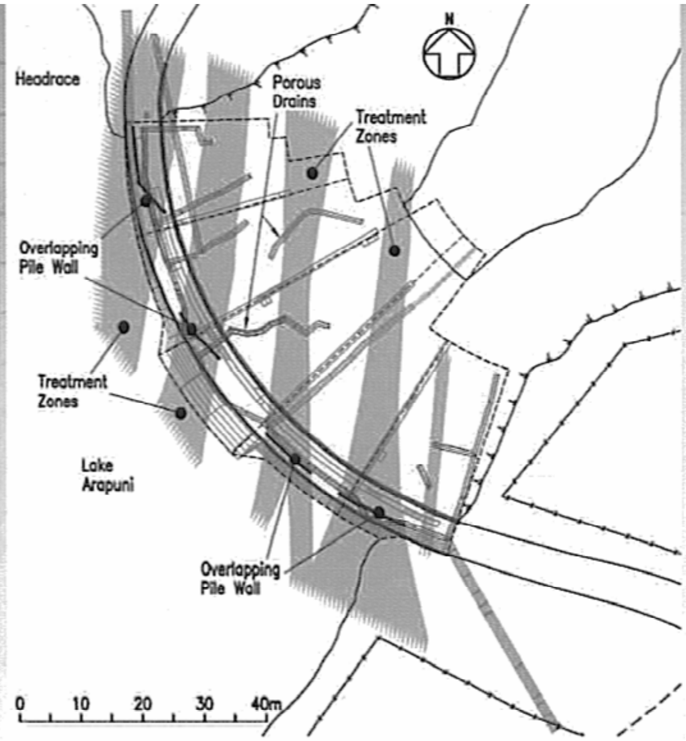


Figure 5. Plan of long-term seepage control remedial works, with cut-off walls, treatment zones and underdrain

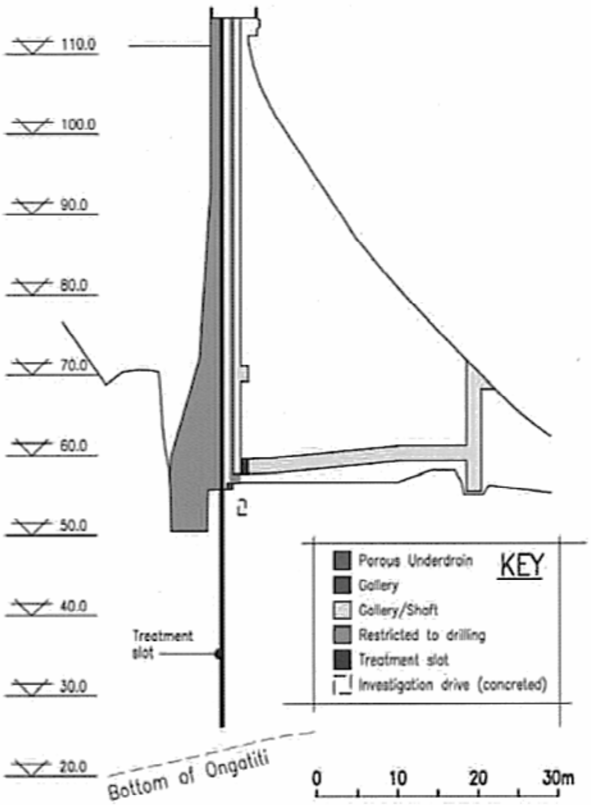
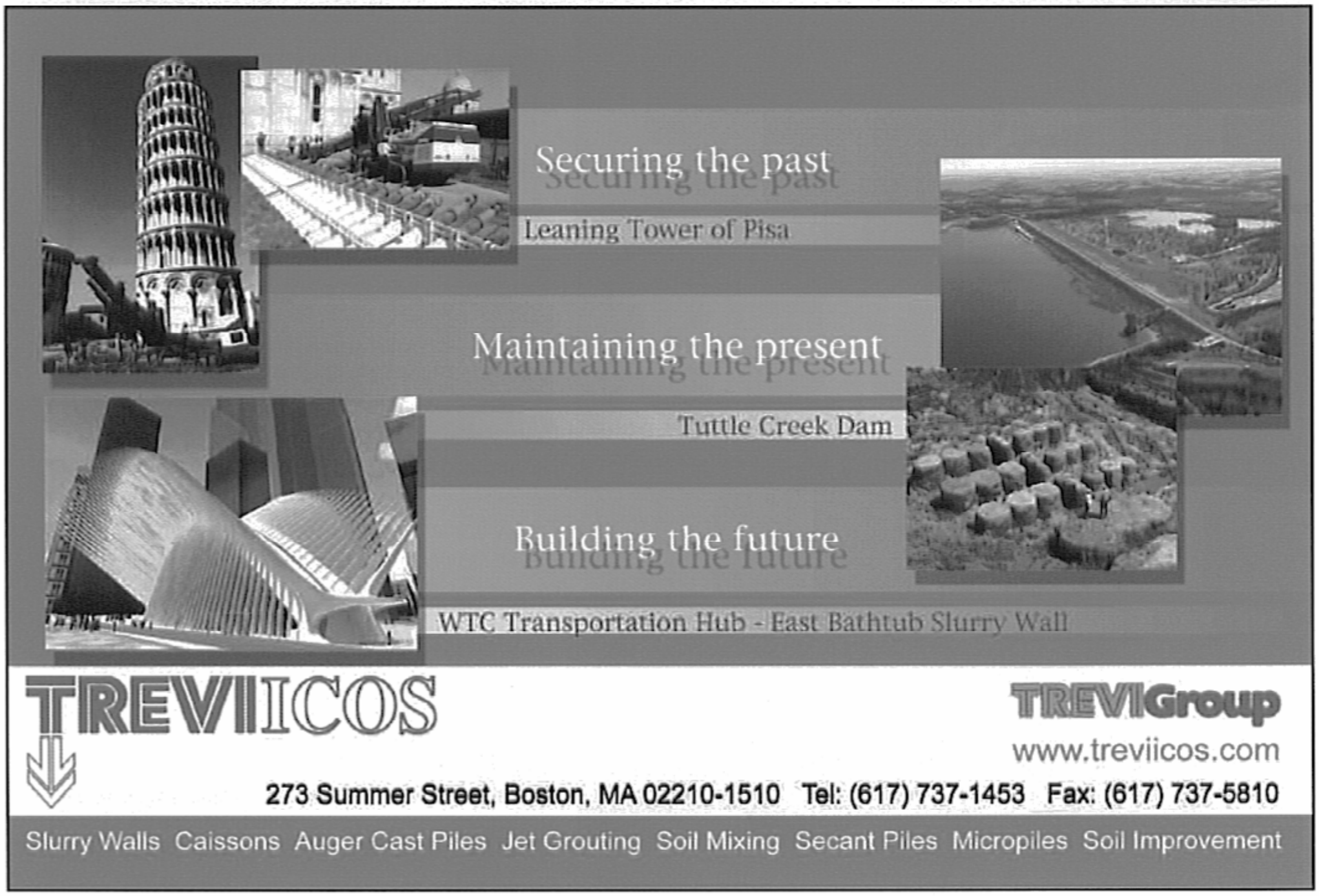


Figure 6. Typical cross section of dam at a contraction joint showing cutoff wall with shafts, gallery and underdrain.



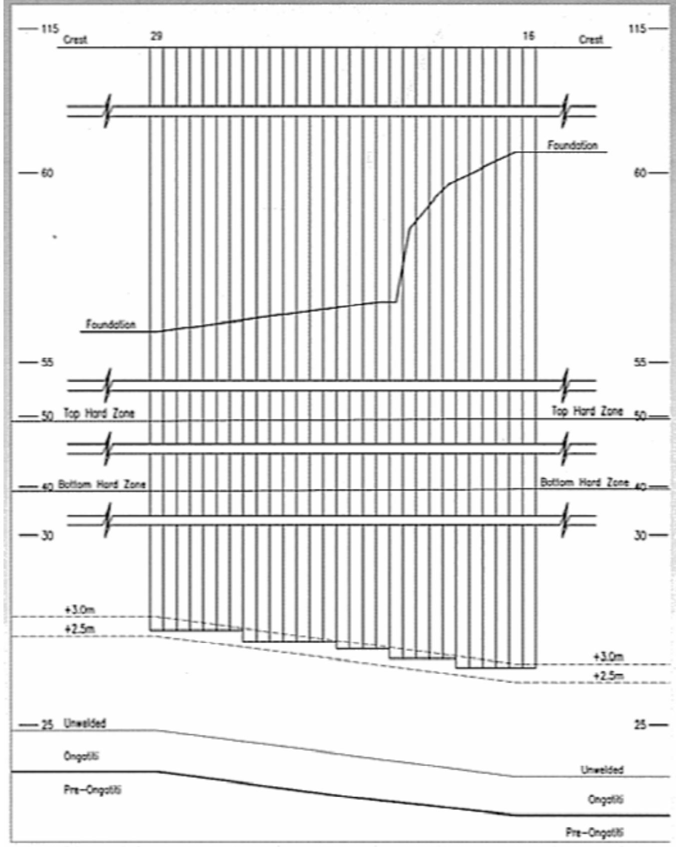


Figure 7. Typical Elevation of a Treatment Panel

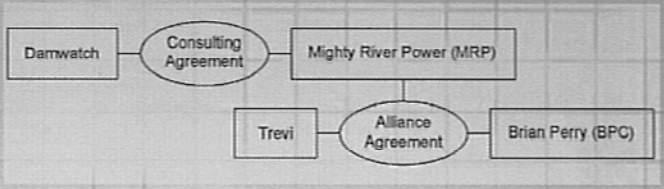


Figure 8. Contractual Relationships

An Alliance Agreement includes the following elements bound into a commercial agreement, (Carter and Bruce, 2005):

- Cost reimbursable component for direct costs
- Negotiated and agreed margin for overheads and profit
- Target outturn cost (TOC) for sharing cost savings or overruns between the commercian participants and the client
- Incentive payment for key performance indicators (KPI's) for quality performance, environmental, and stakeholder management
- A "no blame" best-for-project culture using unanimous decision making during the project
- Agreement not to sue the other parties in the Alliance

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 in the design and construction phases of this project. A consortium of two commercial participants Trevi SpA of Italy and Brian Perry Ltd of New Zealand were selected by Mighty River Power (Figure 8).

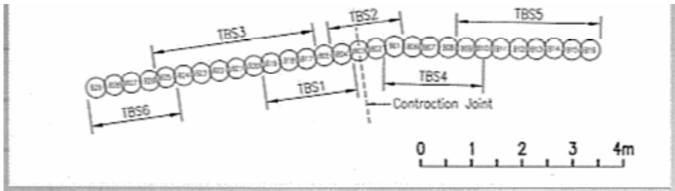


Figure 9. Plan of a treatment panel showing slot sequence

Mighty River Power 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.

OVERLAPPING PILE CUT-OFF

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

The overlapping bored pile wall at Arapuni Dam consists of 400mm (15.75in) diameter holes drilled at 350mm (13.78in) centres (Figure 9) to form the required overlap. The four discrete lengths of the wall ranged from 9.85m to 15.45m (32.3-50.69ft) long, requiring 134 piles and a total drilling depth of 11,600m (38,000ft).

Each cut-off wall was constructed in discrete segments or slots (Figure 9) to limit constructioninduced tensile stresses on the unreinforced concrete dam face up stream of the cut-off wall; and to limit the potential for weak foundation rock to collapse into the open cut-off slot before concreting. Tensile strength of the dam concrete was in the range of 2.2-3.0MPa (320-440psi) from tensile splitting tests and applying Raphael's formulae (Raphael, 1984) to the average compressive strength of the dam concrete (25MPa or 3,600psi). Construction practices, particularly slot lengths and concrete backfill rates, were restricted so that tensile stresses in dam concrete remained less than 1.0MPa (145psi).

A notable cut-off wall project for a dam foundation with full reservoir present was recently completed at Walter F George Dam in Alabama (Simpson et al., 2006) where overlapping piles and diaphragm walls were installed in karstic limestone 30m (100ft) below reservoir level at the upstream face of the dam. Small diameter (150mm (5.9in) diameter) overlapping piles have also been successfully used to form a cut-off 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 (125ft) 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 cut-offs in karst at Wolf Creek Dam, Kentucky (1975-1979) and Beaver Dam, Arkansas (1992-1994) (Bruce et al., 2006). With relatively few precedents for this type of work, and none constructed in such weak rock or to 90m (295ft) depth, the Arapuni Dam project significantly extends international overlapping/secant pile experience.

The main reasons for selecting this construction method at Arapuni were:

- The "positive" cut-off 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 the simplest to construct, which provided confidence that the treatment objectives would be met.
- The method met 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 panels provided assurance of a continuous cut-off.
- The methodology selected scored the lowest construction risk when compared to the other options considered, while not restricting construction alternatives if the methodology failed.
- The selected methodology had the lowest risks of construction cost overruns.

CONSTRUCTION OF THE OVERLAPPING PILE CUT-OFF

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.

The holes were drilled from the dam crest (i.e. above reservoir level) to minimize construction and personnel safety risks. The overlap between holes (and hence continuity within a slot) was controlled by the use of a 400mm diameter guide piece attached to the drill string but running in the adjacent completed hole.

Drilling accuracy was important to avoid features within the dam (such as drains and galleries) and to achieve the target cut-off 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 cut-off panel.

Hole accuracy was frequently measured during drilling by inclinometer measurement of drill rods. On completion of the prescribed number of holes for a slot, the continuity of overlap across the open slot was checked and if rock remained bridging across an open slot, this was removed by tools such as a steel chisel. The condition of the rock walls of the slot was usually viewed with an underwater camera and a velocity probe used to identify if any concentrated flow of groundwater was occurring through the open slot.

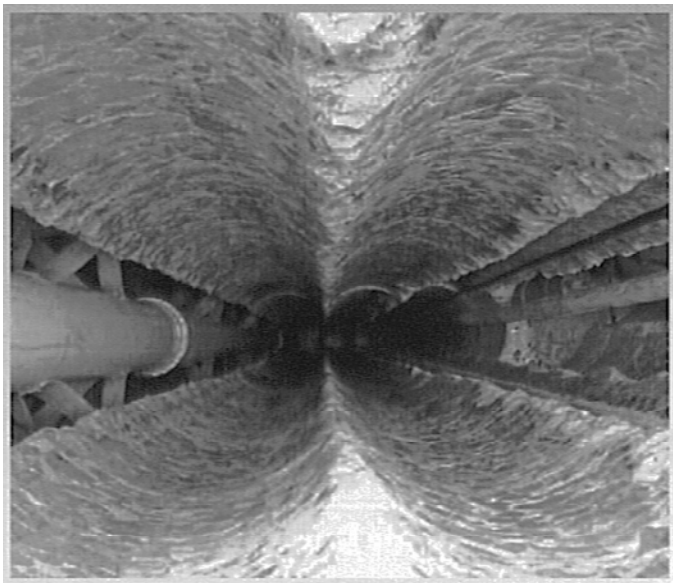
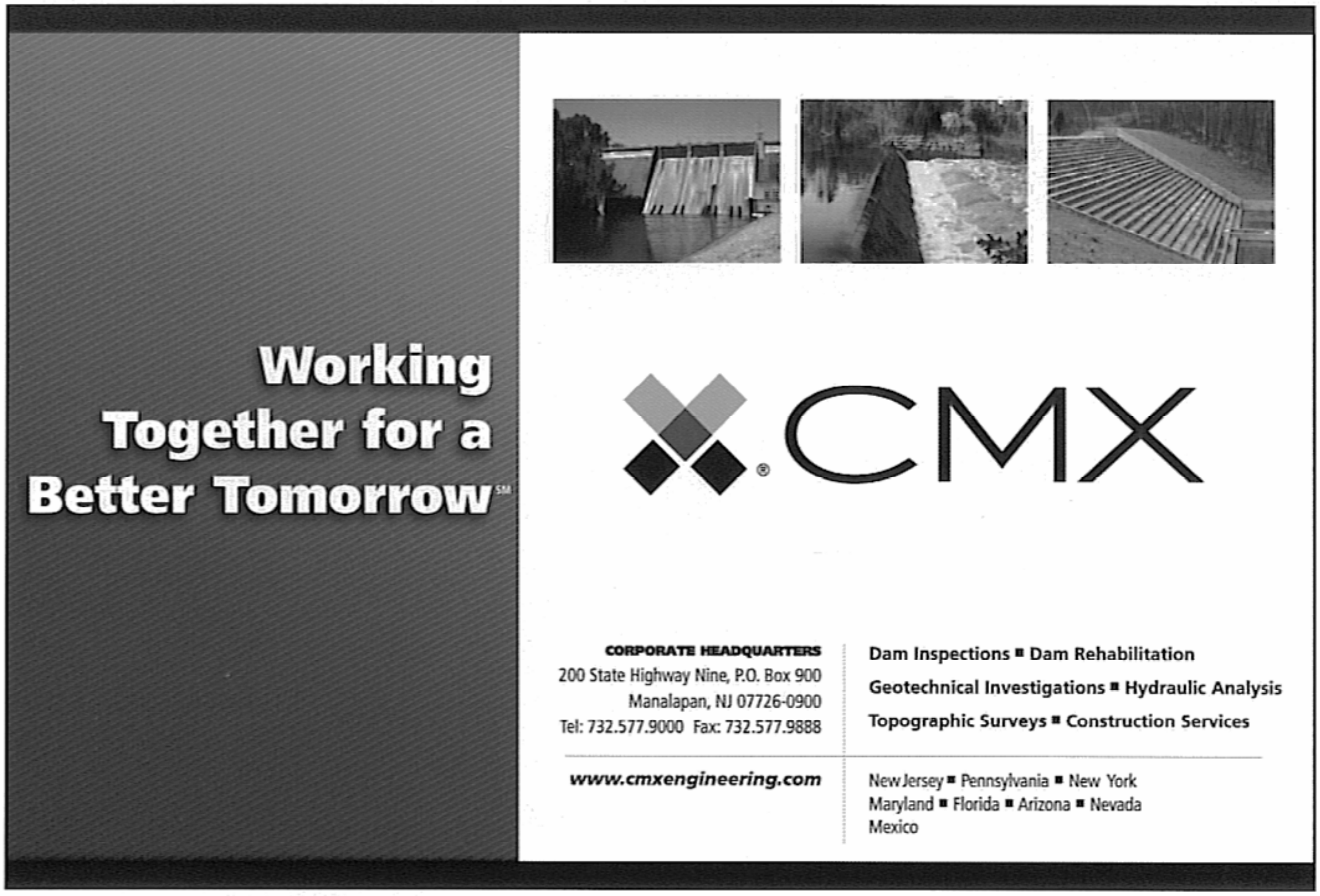


Figure 10. Downhole photograph of overlapping drill holes

The open holes were backfilled with concrete (or grout) using tremie concreting practices. Detailed finite element stress analysis of the upstream concrete face had been carried out, particularly for the net outwards load during concrete backfilling. Several factors were identified as contributing to the risk of tensile cracking of the dam concrete, especially differential temperatures between dam concrete and slot water, and heat of hydration of backfill concrete. The rate of concrete rise was strictly controlled in the upper 15m (50ft) of the dam to reduce tensile stresses in the upstream face due to lateral pressure from fresh concrete. Backfill rates depended on the number open holes in a slot and the thermal stress state of the dam body at the time of the concrete pour. 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 against net outward forces due to drilling fluid.

A conventional tremie concrete mix was used with 10mm aggregate and a nominal slump of 200mm (8in). Additives for shrinkage compensation were not used because anticipated shrinkage in the underwater environment was too small to pose a risk to bond between the new concrete and dam concrete. This was confirmed by underwater camera inspection of the bonded interface exposed when the next slot was drilled.

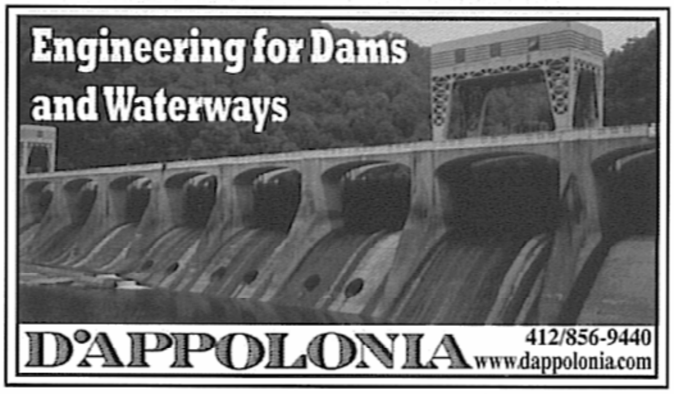
While there were no instances of water flows measured in slots by the velocity probe, downstream dam drains were also closed prior to concreting to reduce the head across the open slot, thereby reducing the risk of concrete washout. If the velocity probe had identified flowing water through the open slot, contingency measures such as anti-washout concrete additives were also available to ensure that there was no washout of fines in the backfill concrete.

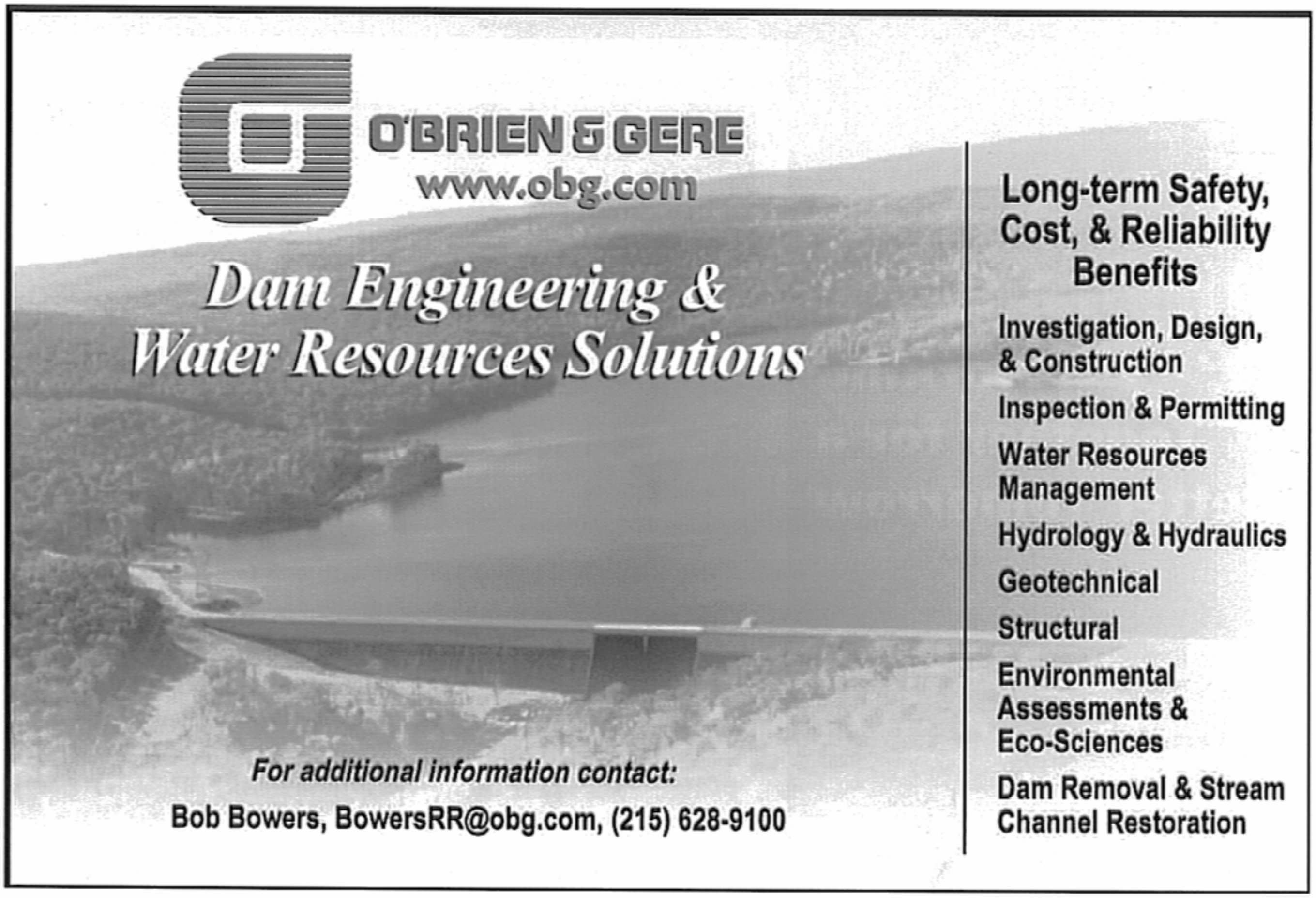


Because cut-off wall construction took place with a full reservoir present, there was an everpresent risk that the construction activities could have a detrimental effect on the fissures, potentially leading to erosion of fracture infill and the creation of a new leak under the dam. Detailed dam safety planning took place at the start of the project in conjunction with foundation coring and mapping. 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 drilled into fissures at the start of the project. These relief holes were normally shut, but were available to manage in-fissures pressures during construction in the event of an unacceptable leak. Discharge from the relief holes was measured at v-notch weirs. A total of sixty two electronic pressure and eighteen weir flow transducers were installed in the dam foundation at key locations.

A dedicated dam safety team was located on site throughout the construction period. 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. Turbidity and pH measuring transducers located in each weir box were also monitored 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 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. Contingency plans were in place to respond to a rapidly deteriorating condition in the dam foundation.

VERIFICATION OF COMPLETED PANELS

Verification of the quality and successful completion of the works took place at several stages of cut-off panel construction. Verticality, continuity and closure of the treatment zone was verified before concrete backfilling using 1) bi-axial inclinometer readings to determine hole drift, 2) underwater camera surveys of slot walls to verify fracture presence in rock face, and 3) sweeping each drilled slot with a steel frame to check that the slot was continuous.

Quality of a completed slot in the cut-off wall was verified by several means: underwater camera surveys were taken of the end of the adjacent completed slot concrete, flow meter surveys were performed to check for concentrated seepage flows in fissures that could impair the quality of the new fresh concrete, tremie concrete operations were carefully controlled, and post-concreting verification was conducted by core drilling the completed cut-off wall.

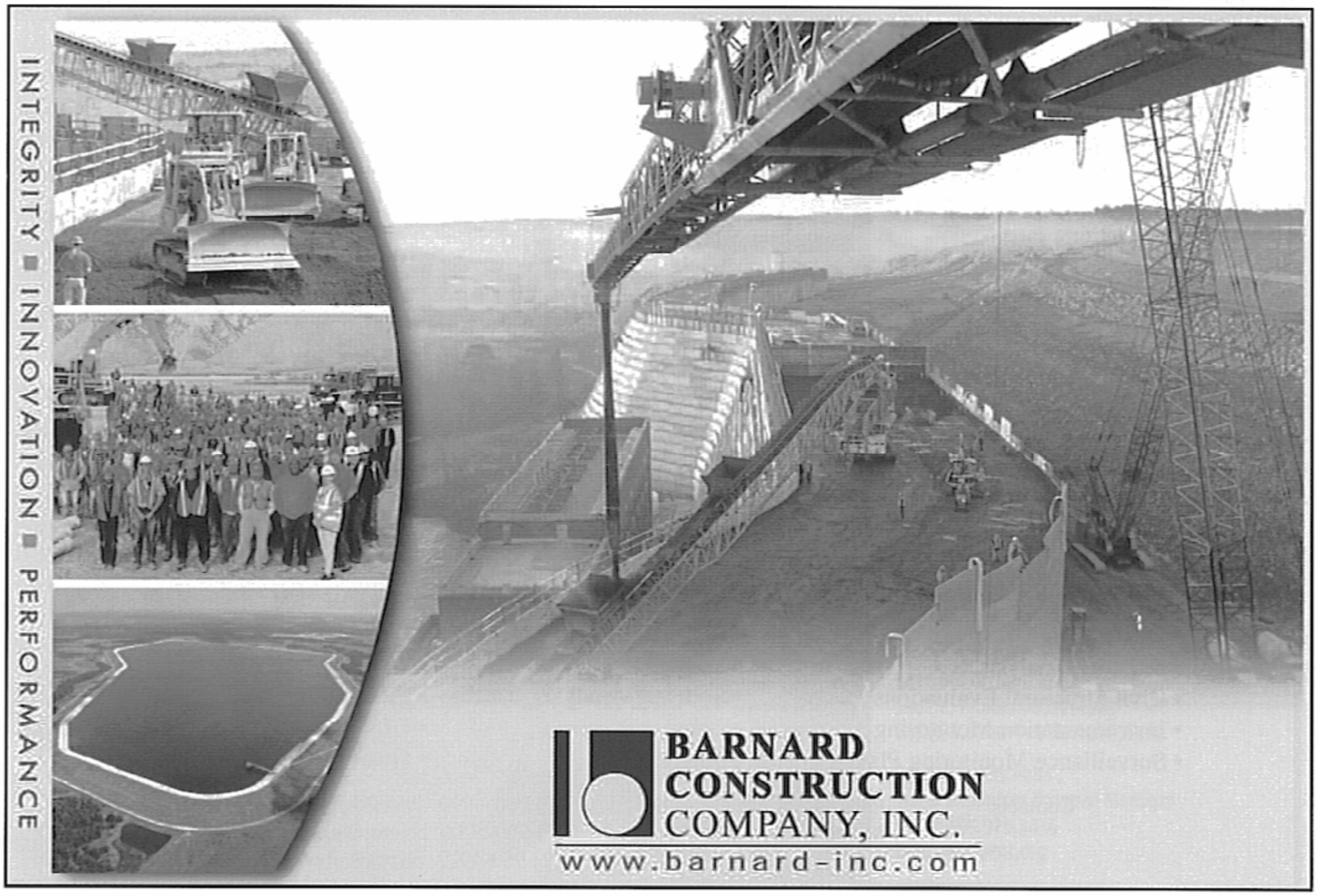
Foundation response to the completed works was assessed by post-concreting monitoring of downstream fissure pressures and drain flows and comparison with pre-construction benchmark behaviour, followed by post-construction pressure response testing of the fissure downstream of the completed panel and comparison of results with similar pre-construction tests.

RESULTS

Foundation drilling and panel construction was completed in September 2007. Inclinometer surveys and continuity checks demonstrated that the completed panel geometry covers the defined treatment area, with hole drift typically less than 500mm (20in) over the height of the cutoff walls, as evident for the panel in Figure 11.

Maintaining drill hole accuracy over 90m was generally successful, although at times a new directionally drilled starter hole was required to improve slot alignment. At two locations shown on Figure 11 this resulted in holes passing behind the adjacent slot without direct overlap. However the orientation of the slots is such that it is not possible for a concentrated leak in a foundation fracture to pass through this section of wall. In particularly soft ignimbrite the drill bit would at times drag the guide sideways, resulting in an elongated slot as seen on Figure 11 where adjacent holes appear not to overlap. However verification tests confirmed successful creation of the slot over the full geometry.

Foundation drains responded to drilling activities in some areas under the dam, but with preventative actions such as closure of



dam drains during drilling and concreting, there were no instances of concentrated seepage flow through open slots that would have affected backfill concrete quality. The quality of the completed concrete cut-off has met or has exceeded the high standards set in the specifications.

Post-construction, instrumentation demonstrated that flow through the former high pressure fissure zones into the downstream area beneath the dam has decreased by approximately 90%, while pressures behind panels in the high pressure fissure zones have decreased by approximately 14m (46ft).

CONCLUSIONS

The dam's owner, Mighty River Power Ltd, has undertaken a dam foundation enhancement project to construct concrete cut-off 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.

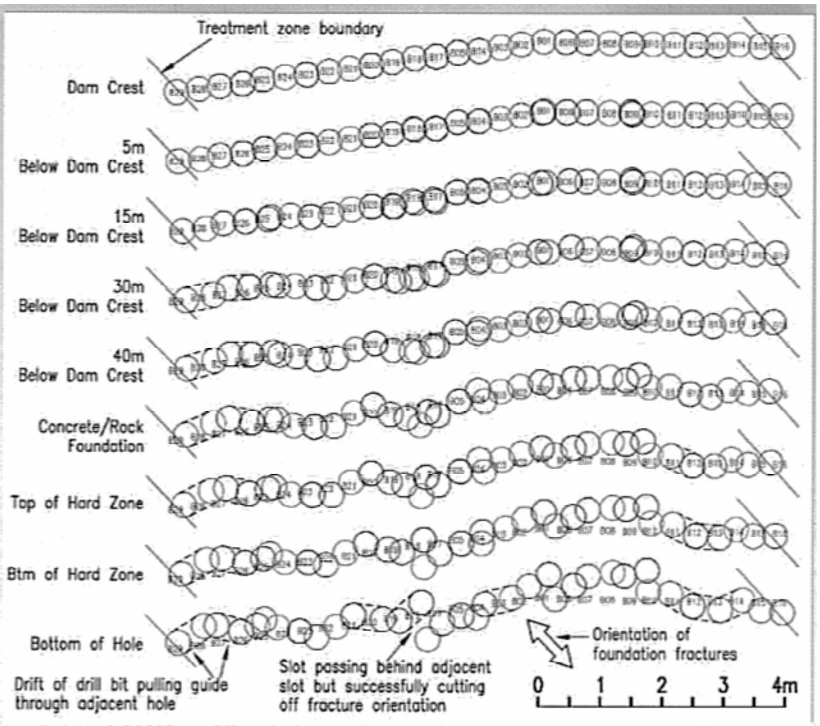


Figure 11. Sections at key depths of Treatment Panel B, showing the relationship between inclinometer readings for each drill hole

The cut-off walls consist of overlapping 400mm (15.75in) diameter holes drilled through the dam and underlying ignimbrite sheet with a full reservoir present. The construction technique required 90m (295ft) deep cut-off walls, which significantly extends international overlapping/secant pile experience and 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, thereby ensuring that the dam's safety was not compromised. The construction works were successfully completed with no damage to the concrete dam, no dam safety incidents requiring intervention, and no impact on power station operation. Pressure and seepage under the dam has been successfully controlled. Furthermore, at the end of the project the underdrain remains serviceable, even though cut-off construction intersected the underdrain several times.

The outcome of this project 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 and equitable risk share and reward. All parties (owner, contractors and designer) concluded that the collaborative design and alliance delivery model contributed to the outstanding delivery of the solution and construction works, whereas the same final cost or degree of success could not be guaranteed had more traditional contractual systems been used.

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