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SEEPAGE EVALUATION AND REMEDIATION UNDER EXISTING DAMS

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1. INTRODUCTION

Unacceptably high seepage volumes and/or pressures may occur under a dam at any point in its service life. It is common to find these conditions existing upon first impoundment, and appropriate remedial measures are undertaken expeditiously by the contractor, willing to have a satisfactorily completed and functioning project and keen to collect the full contract amount invoiced. It is usually easier and cheaper to conduct such works at that time – appropriate resources are still readily available and there remains amongst the various parties comprehensive collective knowledge of the many site and construction factors that may have contributed to the situation. In many cases, the option followed has been simply to monitor carefully, and be prepared to act only if the situation deteriorates to such a point that a dam safety issue develops, or the loss of water becomes significant from an economic, environmental, or recreational viewpoint. This is a path often followed where the dam has been built on foundations known to contain material which may be eroded (e.g., karst) or dissolved (e.g., gypsum) under sustained differential head.

Thereafter, it is typical that the details of the project are still easily accessible via the minds of the active participants – even as much as 20 years on – and in the project records, which in all probably can still be physically located and retrieved. Thus there is still a comprehensive data base from which to design a remedial solution, supplemented of course, by the historical records maintained since first impounding, and what usually amounts to a fairly limited additional program of site investigation. Recent major works of this type described by the authors and their colleagues in the U.S. include the successful seepage remediations at Jocassee Dam, S.C. (Bruce et al., 1993), Dworshak

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Évaluation de l’écoulement et revue des measures de redressement sous de barrages existants

Dam, ID (Smoak et al., 1998), Tims Ford Dam, TN (Hamby and Bruce, 2000), and Patoka Dam, IN (Flaherty et al., 2002).

Occasionally, however, a situation arises when a dam that has been functioning satisfactorily (or, often times the case – not found to be functioning unsatisfactorily) for a long period, suddenly exhibits a potentially very serious seepage condition. Such structures are invariably an integral part of the regional or local social and economic fabric, and so must be remediated. Few, if any, of the engineers who worked on them survive or can be located, while changes in dam operation management personnel and practices may have resulted in the loss of the contemporary construction records. Historical seepage monitoring data are typically incomplete or inaccurate or have been made at instrumentation points no longer functional. Vegetation growth immediately downstream of the dam may have obscured actual seepage loci and characteristics.

The successive steps in a dam remediation process begin with a proper understanding of the factors which have created the condition, the paths followed by the seepage, and the rate and pressure of the seepage. This baseline of knowledge is typically less reliable and complete the older the dam, for the reasons outlined above: the engineer responsible for designing the remediation is therefore faced with a daunting task which can however constitute a truly fascinating research project involving many parallel investigatory tracks.

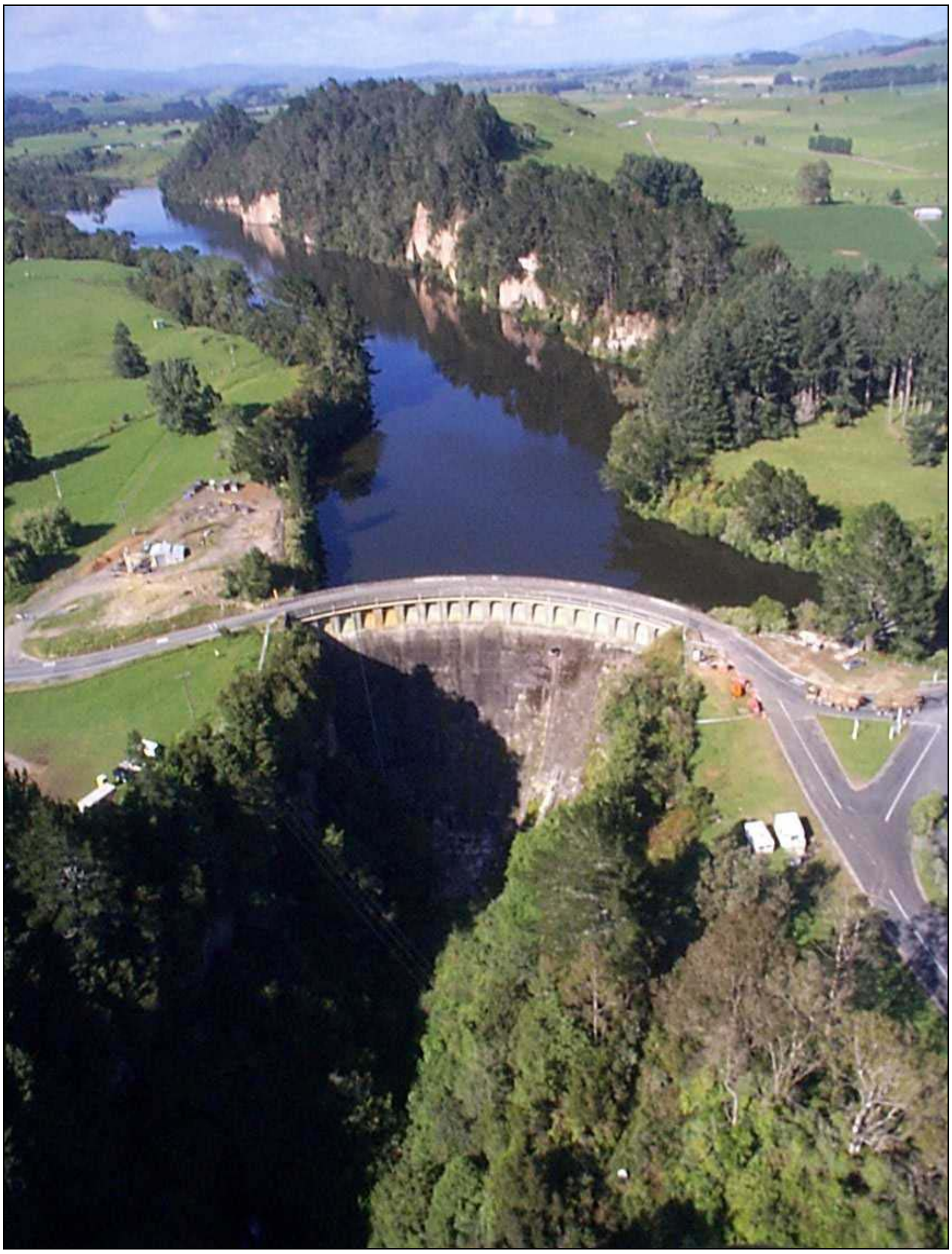
The authors are currently involved in the study and remediation of an old concrete dam in New Zealand, which, for reasons that become clear, "suddenly" was found to exhibit a seepage condition, that, while not yet threatening the dam's safety, provided grounds for considerable present concern. The purpose of this paper is, by using this case history, to illustrate the research path that has been followed in such a way that the technical and financial goals of all the stakeholders have been satisfied.

Following a brief appreciation of the project itself, and its problem, the successive steps in evaluating the baseline are illustrated

- The "event".
- Geological and history research.
- Supplementary investigations.
- Evolution of a working hypothesis.
- Short term remediation, permitting consideration of long term options.

2. SETTING OF ARAPUNI DAM, NEW ZEALAND

Arapuni Dam forms the reservoir for the 186 MW Arapuni Power Station located on the Waikato River, 55 km upstream of Hamilton City in the North Island of New Zealand. Construction was started in 1924 and lake filling was completed in 1928. The Arapuni Power Station is owned and operated by Mighty River Power Ltd., a State owned electricity generation company. The dam is a 64 m high curved concrete gravity dam (Photograph 1) with a crest length of 94 m. Original concrete cut-off walls extend 20 m and 33 m into the left and right abutments respectively. On the left abutment (Figure 1) a headrace channel takes water to the power station intakes. A diversion tunnel runs through the right abutment, south of the cut off wall and has two gate shafts, both upstream of the cut off wall. The dam is founded on relatively new volcanic eject amenta.



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

Barrage Arapuni, Nouvelle-Zélande, vue vers l’ouest.

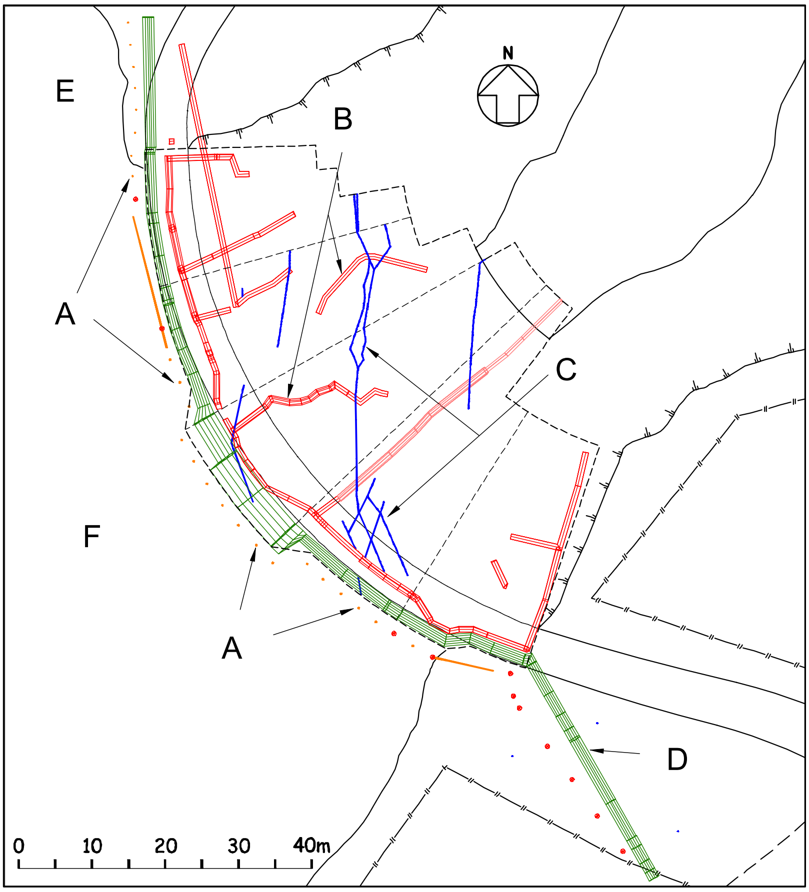


Figure 1. Plan view of Arapuni Dam, New Zealand. The position of the foundation defects ("the feature") as determined during construction is shown. The diversion tunnel curves to the south of the cut off wall.

Bue en plan du Barrage Arapuni, Nouvelle—Zélande. La position des défauts de fondation tel qu'évalué durant la constructin est indiqué. Le tunnel de diversion courbe vers le sud de la barriére d'étanchéité.

A. Grout Curtain	A. E'cran d' injection
B. Porous Drains	B. Drainage
C. Foundation Defects	C. Défauts de la Fondation
D. Cut-off Wall	D. Drainage
E. Headrace	E. Ouvrage D-Amenée
F. Reservoir	F. Retenue

3. THE "EVENT": THE DETECTION OF HIGH PRESSURE SEEPAGE

In 1995, the dam toe area was cleaned up and seepage monitoring arrangements rejuvenated. As part of this program, eight inclined non-core holes were drilled from the downstream dam toe through the dam concrete and into the foundation. The purpose of the holes was to investigate groundwater conditions under the dam. Two of the holes, referred to as OP05 and OP06, intersected a zone of high water pressure and flowed at several hundred liters per minute after drilling. The zone of high pressure coincided with a structural feature mapped on the foundation drawings (Figure 1). The other six holes encountered low ground water pressures consistent with normal design assumptions and indicative of satisfactory conditions. Following this drilling, the flows from the dam drains increased indicating a connection with the feature.

Hole OP05 was subsequently used to measure the pressure in the feature and hole OP06 was used as a relief well. With OP06 flow shutoff, the pressure in the feature in 1995 was about RL 97 m, i.e., 14 m below reservoir level. With OP06 flowing at about 380 liters/min, the feature pressure dropped to Reservoir Level 87 m. OP05 and OP06 pressure and flow were included in the monthly dam surveillance monitoring program thereafter.

By September 2000, pressures and drainage flows were assessed and found to be rising relatively rapidly. This indicated a deteriorating foundation condition with a consequently increasing risk of leakage occurring from the fissure where it daylighted downstream from the dam. The deterioration in seepage conditions was considered to be due to the erosion of fissure infill material. The seepage from OP06 was throttled to reduce flow velocities in the feature in the expectation that this would reduce the rate of deterioration while further investigations were carried out. Another immediate response was to install telemetry on the key seepage monitoring instruments with a 24 hour alarm warning capability.

4. GEOLOGICAL AND HISTORICAL RESEARCH

Close cooperation with local geological specialists helped to supplement the data available from published sources. The dam is located in a region with extensive ignimbrite (welded tuff) flows which erupted during the last 2 million years. At the dam site (Figure 2), three principal ignimbrite flows are recognized. The dam foundation is located on the Ongatiti Ignimbrite dated at 0.9- 1.1 million years. In the abutments the Ongatiti Ignimbrite is overlain by Powerhouse Sediments, a mixture of alluvial and airfall tephra deposits, the Ahuroa Ignimbrite and the Manunui Ignimbrite.

Hydraulically, the Powerhouse Sediments act as an aquiclude with seepage in the Ongatiti Ignimbrite isolated from the overlying units. The dam cutoff walls extend down through the overlying units into the Ongatiti Ignimbrite appear very effective in controlling abutment seepage through the upper units, and the foundation seepage in the Ongatiti Ignimbrite is therefore largely isolated from the abutment seepage even though the valley is very narrow.

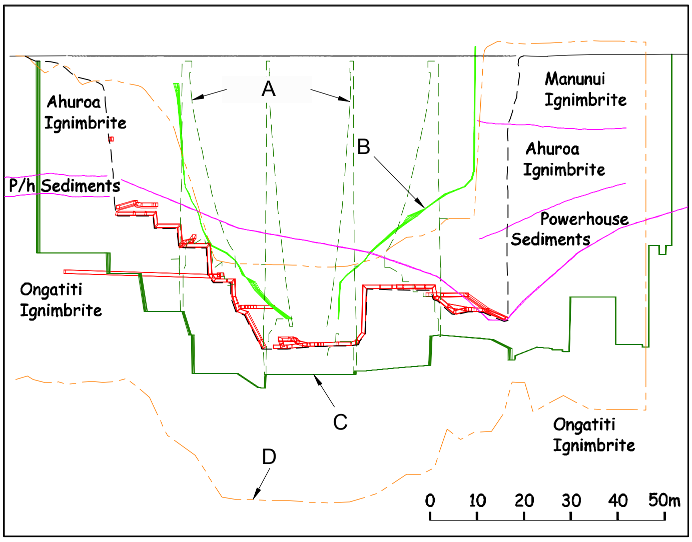


Figure 2. Elevation of Arapuni Dam, New Zealand.

Vue en élévation du Barrage Arapuni, Nouvelle-Zélande.

A. Dam Joints	A. Joints de barrage
B. Upstream Ground Line	B. Massif amont
C. Cut Off Wall	C. Parafouille
D. Grout Curtain	D. E'cran d' injection

The Ongatiti Ignimbrite in the dam foundation is a very weak, coarse pumice breccia with an unconfined compressive strength of between 1 and 5 MPa. Joints are rare, subvertical where present, and spaced at greater than 6m. A set of three sub-parallel cracks or fissures (Figure 1) were recorded on the dam foundation as-built drawings forming a feature that runs obliquely across the foundation from the upstream right abutment to the downstream left abutment.

The dam is founded directly on the Ongatiti Ignimbrite with a cutoff trench located at the upstream face of the dam and a circumferential porous concrete drain located downstream from the cutoff. Some additional, similar contact drains are located along or across the fissures. A series of radial drains conduct drainage flows to the downstream face of the dam where the flows are monitored by weirs (Figure 3).

Fortunately, in this case, the Engineer was able to locate an invaluable collection of contemporary records, photographs, and drawings, from which the following key events were noted.

Following lake filling and in the first 2 years of operation, there was considerable leakage from the reservoir both from the dam drains and from springs in the downstream rock. Flows typically varied between 2200 liters/min and 4200 liters/min.

In May 1929, a large crack opened in the headrace channel due to tilting of the left bank cliff face. The diversion tunnel was re-opened and the lake lowered. The lake was not refilled until April 1932 while the headrace was lined. During

this time a single row cement grout curtain was constructed along the full length of the dam and both abutment cutoff walls.
After refilling the reservoir in 1932, leakage flows had been reduced to 420 liters/min. In the period 1932 to 1943, the records indicate that there were several instances of sudden flow increases and a number of holes on the right abutment were injected with hot bitumen grout. The bitumen grouting had no long-term influence on the leakage flows.

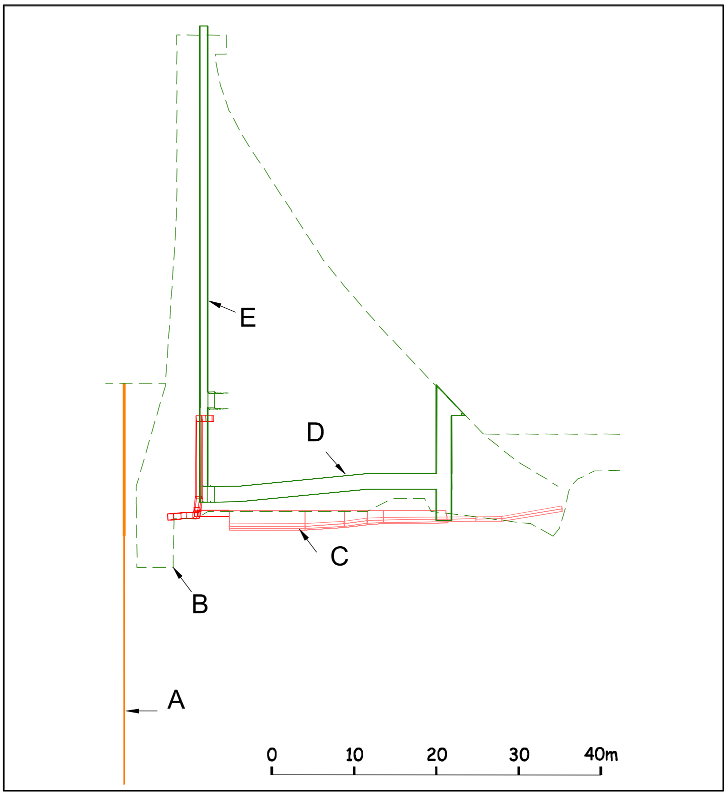


Figure 3. Cross section of Arapuni Dam, New Zealand.

Vue en coupe of Barrage Arapuni, Nouvelle-Zélande.

A. Grout Curtain	A. E'cran d' injection
B. Cut Off Wall	B. Parafouille
C. Porous Drain/Gallery	C. Drainage/Galerie
D. Inspection Gallery	D. Galarie de Visite
E. Inspection Shaft	E. Puits de Visite

From 1943 to 1950 leakage was typically about 750 liters/min but this reportedly declined to about 75 liters/min by 1950. Leakage flows of about 75 liters/min were typical through the period 1950 to 1995.

There was no blanket grouting, curtain grouting or drainage curtain in the dam foundation as originally constructed. The grout curtain constructed during the 1929-1932 lake lowering was a vertical single row curtain with holes at 3m centers. It was constructed just upstream from the dam and cutoff walls, and was not structurally connected to them (Figures 1, 2, and 3). In the valley bottom, a fill platform for drilling was constructed to above the river level and the grout curtain was constructed through the fill. In the steep gorge walls a bitumen plug was constructed between the dam and the gorge wall interface. Grouting was by descending stages with grout injection undertaken at points of lost drill water return. Injection pressures and grout takes were modest with an average take of about 50 bags per 100 feet, although far higher takes were locally recorded coinciding with elevations of lost flush return. The grout curtain does not continue for the full depth of the Ongatiti Ignimbrite sheet. In general the grouting methodologies reflected contemporary practice in the United States.

During the construction of the grout curtain it was found that several holes drilled in the upstream right abutment area and at the downstream end of the left bank cutoff wall had good hydraulic connections with the dam porous drains. Also, following the later construction of a second diversion tunnel gate shaft and the operation of that gate, it was found that that flows from the dam drains increased markedly when the tunnel between the two shafts was dewatered. This was also confirmed in tunnel dewatering in the 1980s and in 1999. This is an unusual observation.

5. SUPPLEMENTARY SITE INVESTIGATIONS

These seepage investigations were initiated with the broad objective- “to safely and economically establish acceptable, long term, stable seepage conditions at the Arapuni Dam.” The seepage investigation therefore looked wider than the immediate vicinity of the feature. Investigation activities included:

- Installation of an external filter on OP06 relief well flows
- Drilling and piezometer installation from the dam galleries to establish the area of foundation adjacent to and within the feature subject to the high pressures observed at OP05 and OP06
- Drilling, Lugeon testing and instrumentation from the two abutments to assess rock properties and seepage conditions adjacent to the grout curtain, particularly in the area of the mapped foundation defects.
- Remote Operated Vehicle inspection and mapping of the lake bed in front of the dam.
- Dye testing from the lake and within boreholes and the diversion tunnel.
- Temperature testing and groundwater sampling from seepage flows and boreholes.
- A final series of non-core holes drilled from the downstream toe into the OP05/OP06 feature to establish relief wells and grout injection points (Figure 4).

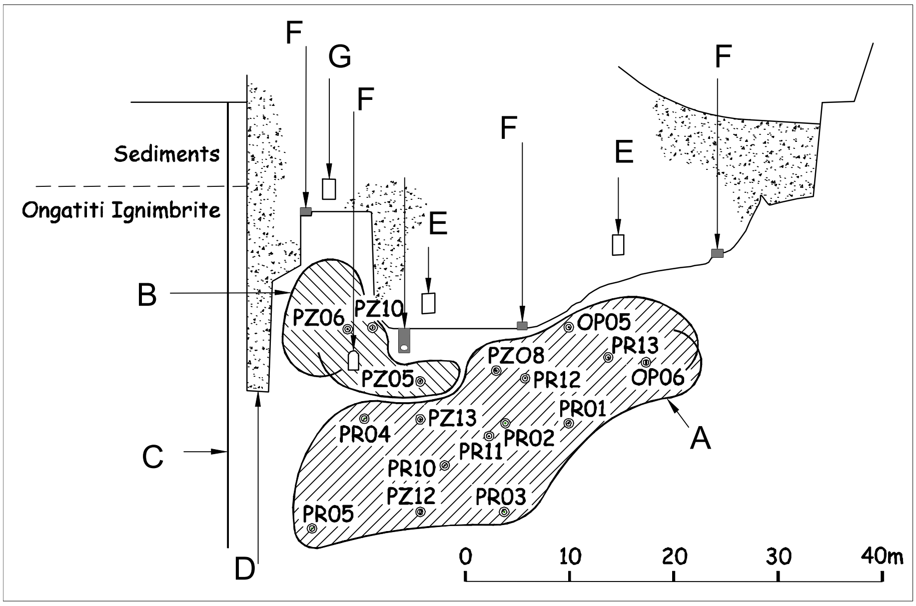


Figure 4. Elevation of defect showing intersections by investigation and grout holes drilled from the galleries and the downstream face.

Élévation des défauts de foundation illustrant les intersections définies par les trous d'investigation et d'injection forés à partir des galleries et de la face aval.

A. High pressure zone	A. Pression Elevée
B. Low Pressure zone	B. Pression basse
C. Grout Curtain	C. E'cran d' injection
D. Cut Off Wall	D. Parafouille
E. Gallery	E. Galarie
F. Porous Drain	F. Drainage
G. Circumferential Gallery	G. Galarie de Circonférence

6. EVOLUTION OF A WORKING HYPOTHESIS

The wealth of historical and contemporary data was reviewed in a series of workshops, involving the Owner, the Engineer, and various Specialty Consultants and Peer Reviewers. This proved to be a very stimulating, efficient, and effective process. This team noted the following key findings:

- High piezometric pressures were only observed in some parts of the feature (Figure 4) and had little effect on the adjacent foundation. They were not present in the other mapped foundation cracks.
- OP06 flows were carrying bitumen fragments, clay fragments from the fissure, snails (small), and lake biota.
- Dye traces were observed in drainage flows from releases in the lake, the headrace channel, right abutment boreholes and the diversion tunnel. Average dye velocities in the rock foundation were typically between 0.8 and 2.0 m/minute. Special attention was devoted to analyzing and rationalizing

the data from the dye testing, using instrumentation capable of detecting dye concentrations of a few parts per billion.

- No flow entry points were observed in the lake bed, the cliff/dam face bitumen plugs were in good external condition, and the steep uneven lake bed terrain was unsuitable for constructing an upstream blanket.
- The nature of the infill materials (nontronite – a clay mineral derived from the weathering of airborne material deposited almost contemporaneously into shrinkage cracks in the ignimbrite) observed in cores from the lower part of the Ongatiti Ignimbrite was indicative of discontinuities existing through the whole depth of the sheet.
- There was evidence that groundwater in the Ongatiti Ignimbrite was hydraulically isolated from that in the underlying pre-Ongatiti Ignimbrite.
- There was a particularly strong flow connection between the flooded portion of the diversion tunnel and the defect.
- The high pressure area of the defect averaged about 80 mm in width.
- The hydraulic connection between the dam drains and the feature was probably limited to a porous drain contact near OP05 (Figure 4).

The team concluded that the seepage sources included both the lake bed and the diversion tunnel, and that the existing grout curtain was clearly not effective in influencing the present seepage conditions. The flow paths were felt to be most likely within the Ongatiti Ignimbrite, and focused within a discrete subvertical feature from which the nontronoite had been eroded over the years of service. There was considerable concern that the possibility existed of a sudden blow out of nontronite leading to a direct and major pipe under the dam which, acting under full reservoir head, would cause destabilization of the downstream left abutment. This would immediately elevate the situation to one of dam safety being threatened.

The choice of a definitive long term solution lay between installing a new intense grout curtain (to prevent seepage) or building a large downstream buttress (to ensure seepage would not cause structural instability). Given the technical challenges and cost inherent in these options, particularly since foreign expertise would most probably be involved, the final choice demanded lengthy and detailed study. In the interim, therefore, the concept of a smaller scale remediation was agreed, as a holding action. The nature of this operation, described in Section 7 below, would itself provide further information on the nature of the feature, and so would provide a further test of the working hypothesis.

7. SHORT TERM REMEDIATION

When sharply rising pressures were identified in OP05 in September 2000, a possible mitigation measure was to attempt to grout the feature using holes OP05 and OP06. There were two main concerns with this concept. First, little was then known about the nature of the flow paths within the foundation and so the grouting operation and its effectiveness would be very uncertain. Also, there was concern that the high grouting pressures necessary to inject grout through OP05 and OP06 could blow out the infill and so significantly increase flow rates.

It was therefore concluded that while fissure pressures remained within previously observed limits, investigations and preparations necessary for a high quality, planned feature grouting operation should be completed. Grouting equipment and materials were assembled at the site to enable feature grouting to be carried out at short notice during the investigation period if conditions warranted.

Grouting was planned and initiated once the primary seepage investigations were complete and additional relief wells and grouting holes were drilled into the high pressure area of the fissure (Figure 4). The grouting plan required that upstream relief wells would be used during the grouting operation to lower fissure water pressures such that the added pressure of the grout injection would not exceed previously observed pressures in the fissure. This minimized the risk of infill blowout and increased downstream leakage.

Before the grouting operation, the seepage properties of the foundation were baselined to enable improvements to be determined following the grouting and any subsequent improvement works. Water and dye tracer was also pumped into each of the grouting holes prior to grouting to give an indication of likely flow paths and assist in determining the likely sequence of grouting.

To protect the drainage function of the porous concrete foundation drains during grouting, two fundamental precautions were taken. First, plumbing was installed to enable flushing of the drain in such a way that the drain would not backflow into the feature and so disrupt the setting grout. Secondly, small wood chips were injected into the feature close to the contact with the foundation drain to help prevent grout entry. The drain pressure was lowered to increase the flow and draw the wood chips onto the drain/feature contact. These precautions proved very successful, and the cement based grout did not subsequently enter the drain.

The grouting operation was managed and coordinated by Mighty River Power's engineering staff utilizing very detailed preprepared procedures and checklists. The grout mixing and injection was undertaken using local labor and equipment under the direction of foreign grouting supervisors. Key dam safety parameters were monitored and the relief wells operated by the Owner's dam safety consultants. The project was a classic example of practical and effective partnering in action.

The grout mix was designed to be placed in either static or flowing water conditions in a fissure conceived to average 80 mm wide. It incorporated anti-washout and dispersant additives, and a water/cement ratio (by weight) of 0.8, plus 9% of bentonite (by weight). The mix had been designed and experimented with to ensure it would be stable, durable, and possessed of appropriate rheological and hydration properties, reflecting the best contemporary practice.

Grouting took 12.5 hours during which 11.5 cubic meters of grout was placed. At the start of grouting, relief well discharge was transferred to the most upstream well so that grout being injected at the downstream end was in still water. After about 2.5 hours, grout was detected at the relief well. The relief well was closed after 6 hours and a further 4.4 cubic meters of grout was injected to refusal at 8 bar. Minor modifications were made to the mix during injection in response to the field observations.

It was recorded that the quantity of grout injected was about one third the quantity estimated assuming that fissure infill had been totally removed in the high pressure area. This piece of information, plus an analysis of the observations made during grouting strongly suggested there were considerable areas of intact nontronite infill remaining within the feature.

8. VERIFICATION OF SHORT TERM REMEDIATION

The immediate response to the grouting was that drainage flows from the dam drains dropped from a total of 600 liters/min to 50 liters/min. Piezometric pressures in other parts of the foundation and in the other mapped cracks either dropped or remained constant. The only pressure increase under the dam foundation was in a deep piezometer, which indicated no effect on dam stability.

Four holes drilled into the grouted area retrieved drill core showing three natural infill zones and one grouted zone. The grouted zone had good quality grout across the fissure with only minor surface traces of clay infill against the fissure sides. This drilling, combined with the other data and observations, led to the development of a model for envisioning the treated feature (Figure 5).

9. FINAL REMARKS

Further verification work is planned using dye tracing, temperature measurements and dewatering of the diversion tunnel to determine if dam foundation flows and pressures increase as they have in the past. This will give comfort in the period prior to the implementation of the final solution that the situation is not deteriorating significantly. This case history illustrates several basic factors which the authors believe should be contemplated by engineers involved in similar projects. The authors also believe that these factors transcend developments in measuring and monitoring technology, and that they are consistent with the fundamental aims of dam safety programs internationally. These factors include:

1. Access to detailed historical dam construction, and performance data and the provision of resources to analyze them in the light of contemporary developments.
2. Assimilation of all available regional, local, and site specific geological data especially with regard to lithogenesis and long term performance under sustained hydraulic head.
3. The commitment to conduct focused contemporary site investigations in support of working hypotheses.
4. The provision of a professional forum in which the situation can be comprehensively reviewed and remediation plans logically evolved.
5. A comprehensive, reliable, and routine dam instrumentation, data monitoring, and interpretation program.
6. The execution of the requisite remedial measures by properly qualified human resources using state of practice means, methods, and materials.
7. Appropriate verification of remediation performance and long term effectiveness.

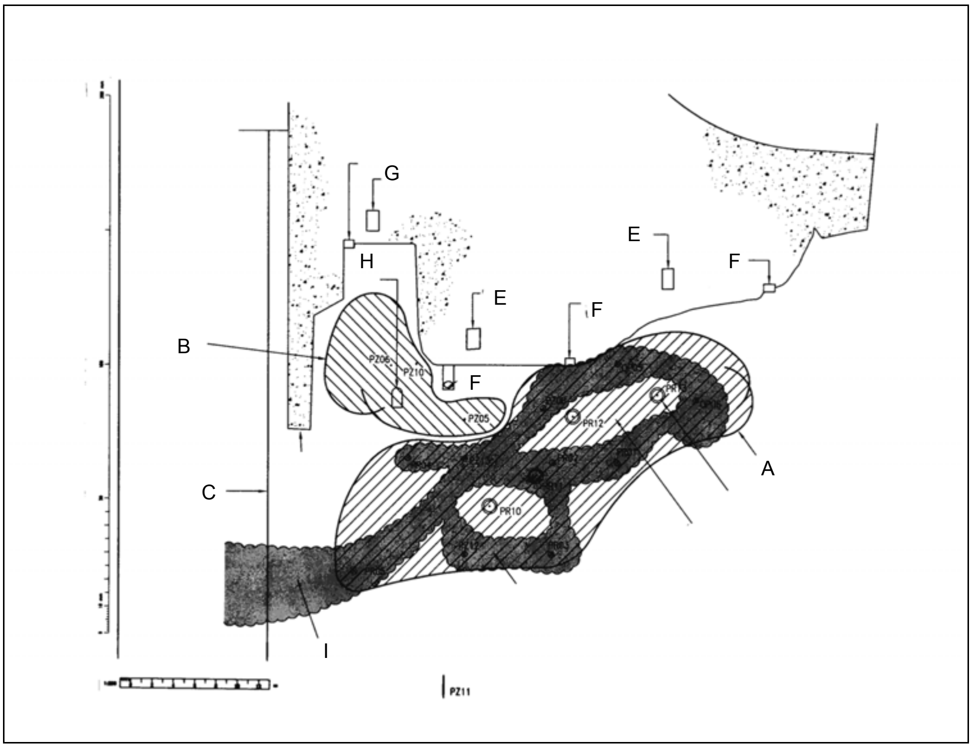


Figure 5. Conceptual interpretation of the grouted feature conditions after treatment.
Interprétation conceptuelle des conditions des défauts de foundation suite au traitement par injection de coulis.

A. High pressure zone	A. Pression Elevée
B. Low Pressure zone	B. Pression basse
C. Grout Curtain	C. E'cran d' injection
D. Cut Off Wall	D. Parafouille
E. Gallery	E. Galarie
F. Porous Drain	F. Drainage
G. Circumferential Gallery	G. Galarie de Circonférence
H. Investigation Drive	H. Galarie de Visite
I. Grout in fissure replacing nontronite	I. Clavage de Joint

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SUMMARY

Unacceptably high seepage volumes and/or pressures may occur under a dam at any point in its service life. When such events occur long after the dam has been completed, the difficulty of collecting contemporary construction data and memories is a severe challenge to the goal of trying to understand the cause of the problem. The amount of new field investigatory work to be conducted to help understand the cause and characteristics of the problem is correspondingly greater. This paper presents the case history of seepage evaluation and remediation at a 74-year-old concrete dam in New Zealand. The example is used to illustrate the basic steps that should be followed when a “condition” is recognized to exist which may threaten dam safety. These steps apply regardless of the degree of sophistication of the instrumentation and monitoring equipment.

Resume:

Des debits de fuites et/ ou des pressions d’un niveau inacceptable sous la base d’un barrage peuvent etre observes a tout moment de la vie d’un barrage. Quand de tels phenomenes se developpent longtemps après la construction du barrage, la difficulte de rassembler les donnees et souvenirs datant de la construction est un obstacle severe a la determination des causes du/des problemes. Le volume de travaux de reconnaissance supplementaries a realiser pour aider a la comprehension de la cause et des caracteristiques du probleme, est en consequence plus important. Cet article traite d’un cas de fuites et des measures de reparation sur un barrage de Nouvelle Zeland, vieux de 74 ans. Cet exemple est utilize pour illustrer les measures de base a prendre lorsqu’une “condition” qui peut mettre en danger l’integrite du barrage, est mise a jour. Ces measures sont applicables quelque soit le degree de sophistication de l’instrumentation et des equipements de controle.