Geological Inputs into the Waipori Dams Stability Review

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ABSTRACT

Three concrete arch dams (Mahinerangi Dam, Waipori Nos. 2 and 4) and one gravity weir (Waipori No.3) of varying ages provide the impoundments for the Waipori Hydro-electric Scheme. Findings of a comprehensive dam safety review carried out in 2012 indicated some knowledge shortcomings when assessing the stability of the No.2 and No.4 structures. It was proposed to fill these knowledge gaps by carrying out a multi-staged assessment of the two structures comprising a comprehensive desk study and gap analysis; fieldwork comprising detailed engineering geological mapping and intrusive geological investigation; determination of analysis parameters, and assessment of kinematically feasible failure modes to inform a Finite Element Method model build and analysis.

Whilst regulatory requirements are entirely different today to when the dams were initially constructed (1914 and 1956) and given that records of construction are minimal, this project demonstrates the importance of carrying out thorough reviews of historical records where they exist, prior to launching into deep and expensive site investigation. The reviews turned up several important documents which reduced site investigation inputs later and aided in building a clearer understanding of the history of work at the sites.

Geological mapping was a key element to understanding risk at the sites in a geologically complex setting, more so than intrusive investigation, which was carried out primarily to gain samples for strength testing. Geological mapping and traditional geological analysis tools such as aerial photo interpretation, stereonets and assessment of rock quality from boreholes have been critical for provision of rock mass properties and determination of potential kinematically feasible failure mechanisms of these ageing assets. The first stages of work have provided valuable inputs into the forthcoming finite element analysis (FEA).

1 INTRODUCTION

In the late 1990s, Trustpower's generation portfolio grew significantly through the purchase of a range of hydro generation facilities across New Zealand. These facilities had a diverse range of historical ownership and asset management methodologies. Trustpower's current dam portfolio contains 47 large dams (as defined by the New Zealand Building Act), with a variety of dam types and an age range from less than 10 to over 100 years. The dams are located in a wide variety of geological, seismological and hydrological settings. This level of diversity has always presented a challenge for dam safety and deficiency management.

Over the years dam safety regulations have been progressively promoted in New Zealand, however, there remains no specific dam safety regulatory framework in New Zealand. Rather dam safety is often addressed indirectly through the Resource Management Act (RMA 1991). In the absence of a specific regulatory framework, dam safety management is largely industry led

with guidance provided via the New Zealand Society on Large Dams (NZSOLD) Dam Safety Guidelines 2015.

1.1 Dam Safety Deficiency Management Programme

Since the late 1990s, Trustpower has needed to adopt a range of methodologies to manage dam safety and associated deficiencies. Trustpower developed its current approach in late 2014, which uses a Maturity Matrix approach. All structures and processes are broken down into manageable sub-categories and rated on a scale from Rating 1 ("Desirable") to Rating 7 ("Deficient"). This includes known deficiencies and areas of uncertainty. Two of those structures which were identified as having uncertainty regarding stability were the Waipori No 2 Dam and Waipori No 4 Dam, which are the subject of this paper.

1.2 The Project

Trustpower's 2012 Comprehensive Dam Safety Review identified uncertainties for both No. 2 and No. 4 dams with regard to their stability under earthquake loads. Trustpower initiated a project to establish if there is a deficiency. In order to confirm this, a modern day Finite Element Analysis (FEA) needs to be conducted including analysis for static, flood and seismic loadings. This paper discusses the steps that needed to be carried out in order to gain relevant information on the ageing dams prior to creating the model with a focus on the geological inputs required.

2 BACKGROUND

The Waipori Scheme comprises four dam or weir structures of varying ages. This paper focuses on Dam Nos. 2 and 4.

2.1 No.2 Dam

No.2 Dam is an 18.6 m high, concrete gravity arch weir. Having no spillway, the dam was designed to overtop and has a crest length of 52 m. Construction of the original dam is believed to have been completed in 1914. Following intrusive investigation into concrete strength carried out from 1969 to 1971 the dam was subsequently raised by 0.9m in 1963 and a further 1.2m in 1970 and strengthened with grouting and installation of anchors within the dam from 1973 to 1988.

2.2 No.4 Dam

Built in 1956, the No.4 dam is a concrete arch structure with a gravity spillway. It is 18.6 m high and has a crest length of 76.2 m. During a flood event in 1977 the right wing-wall of the spillway collapsed and was subsequently replaced. Otherwise, the dam stands as it was originally constructed.

2.3 Existing Information and Knowledge Gaps

It was recognised early in the project that information on the dams themselves was scarce and not collated in one location. This led to the first stage of the project being an assessment of existing records to determine knowledge gaps. Information was sourced from a variety of locations including the Waipori station, Dunedin City Council and Calibre Consultants Ltd in Dunedin (formally Duffill, Watts and King (DWK) the consultants for the scheme throughout the 1970s to 1990s).

The desk study phase of the project collated known information, but also uncovered useful documents in archived locations. These documents included historical photographs from construction, earlier geological mapping, in particular detail at No.4 dam, plans and sections for the dams, and an early Finite Element Analysis of No.2 and No.4 Dams carried out in 1992 by DWK.

While formerly unknown drawings had been found providing additional information on the foundation level of No.4, it was recognised that detailed as-built information was still missing for both structures, as well as any site observations recorded during construction. DWK carried out

an assessment of concrete strength of No.2 dam for the 1970s to 1980s strengthening programme but present day assessment of concrete strength had been based on visual assessments only since that time for No.2 Dam, and only ever based on visual assessments for No.4 Dam. Additionally, there was no information at all on the founding material of No.2 Dam. While a 'geological appreciation study' was carried out in 1986 for No.4 Dam there was no determination of rock mass characteristics or intrusive geotechnical investigation at either structure.

In order for a modern day FEA to be carried out these knowledge gaps were required to be addressed.

3 SITE INVESTIGATION

3.1 Geological Appreciation

Overview

Bedrock in the area is known to comprise Haast Schist (Bishop & Turnbull 1996), exposed in mid-Cretaceous times by uplift on north to north-west trending faults. Subsequent erosion has resulted in the formation of a peneplain, which created gentle topography that has since been altered by post-peneplain faulting and folding. Haast schist in the region had been mapped as Textural Zone IIIA (Bishop and Turnbull, 1996). From site mapping it was determined that characteristics of the bedrock included well foliated quartzo-feldspathic schist with prominent quartz segregations parallel to foliation. Quartz veins that cross-cut foliation are notable by their absence. Pelitic (fine-grained) schist is dominant throughout the area, individual mica grains are fine sand sized discernible through hand-lens. Lineation in the form of rodding and bedding foliation intersections are evident. Using a re-appraisal of the Haast Schist Textural Zones (Turnbull et al, 2001), Haast Schist in the Waipori Hydro-Scheme area is considered to be Textural Zone III, defined on the basis of mica grain size and foliation penetration. Engineering projects throughout the region carried out within Haast Schist have faced difficulties as this material is known for well-defined foliation parallel shears that can result in large complex deep seated landslides and cause problems for slopes, cuts and structure foundations. The best known of these projects is likely to be the Clyde Dam construction.

Surficial deposits on gentler slopes comprise colluvium with schist derived quartz rich alluvial sediments found in some areas within the valley floor.

Two prominent faults are identified in the wider Waipori area (Bishop & Turnbull 1996). Both faults are classed as inactive by GNS Science (Bishop & Turnbull, 1996). The North Eastern Fault trends broadly north to south, traversing the eastern end of the Mahinerangi Reservoir, east of No.2 Dam and crossing the Waipori River approximately 400 m downstream of No.2 Dam (Bishop & Turnbull 1996). The McNamara Fault trends east to west along the Waipori Valley mapped to be passing 200 m south of the right abutment of No.4 Dam. Overall, the seismic risk in the area is relatively low in the New Zealand context.

No.2 Dam Geological Mapping

The No.2 dam is accessed via a gravel road that drops from the peneplain into the gorge. The road cut, the platform for the No.1 water supply tunnel and construction of the dam itself have resulted in excellent exposure of schist bedrock on both abutments down to foundation level. For geological mapping purposes the left abutment was accessed using safety ropes.

Exposed rock was consistent, noted to be moderately weathered, moderately strong with a fracture preference parallel to foliation. Foliation is well-developed and predominantly south dipping 30 to 40 degrees. Quartz segregations are typically 10 to 30 mm thick and comprise 20 to 30 % of the rock mass.

Surfaces are controlled by foliation and jointing. In addition to the joint set parallel to foliation there are three more dominant joint sets identified consistently in the area of the No.2 Dam.

No.4 Dam Geological Mapping
An earlier geological map (MacFarlane, 1986) uncovered in the desk study phase formed the basis of geological mapping at No.4 Dam.

On the left abutment of the dam, the schist is of good quality, moderately weathered, moderately strong with quartz segregations up to 50 mm thick, though typically less than 20 mm. Foliation attitude on the left abutment dips 20 to 30 degrees to east to north-east. By contrast, the right abutment is highly disturbed with numerous shear zones present, varying in character but with distinctly fissile rock mass sheared to fine to coarse, platy, angular gravel. Foliation orientation within shear zones is variable though broadly parallel to sub-parallel with the orientation of the McNamara Fault, with micro-folds (Figure 1) evident within some zones. The disturbed nature of bedrock on the right abutment is attributed to the proximity of the McNamara Fault which was mapped (MacFarlane 1986) as passing approximately 200 m to the south of the dam. Recently a new track has been cut on the river right of the No.4 reservoir, which exposes the fault. The zone of deformation is approximately 70 m in width at this location (consistent with the earlier DSIR report), highly disturbed, with a lack of distinct fabric. Material in the fault zone is predominantly extremely weak and when logged as a soil material is assessed to be moderately dense, light greyish brown sandy angular gravel. There are zones (up to 2 m diameter) of dark bluish-grey, highly plastic silty clay within localised pockets within the overall fault zone. Shear and crushed zones were noted up to 300 m on either side of the 70 m wide fault zone, including on the right abutment of the No.4 Dam. The fault appears to be high angle and downthrown on the southern side, indicative of a reverse thrust fault.

Figure 1: Micro-folds within shear zones in Haast Schist at No.4 Dam

3.2 Intrusive Investigation

No.2 Dam
Due to the consistency observed in outcrops around No.2 Dam, the intrusive investigation was limited to one borehole to assess the impact of weathering on the exposure and to obtain a sample for core testing. The borehole was drilled near the right abutment of the dam to a depth of 27 m. A distinct weathering profile was observed where at a depth of 7.1 m a clear colour change was noted and the rock changed from slightly weathered to fresh. Rock Quality Designation (RQD) in recovered core was 80 % consistent with geological mapping.

No.4 Dam
From mapping, a distinct change in rock mass quality was noted from the right abutment to the left. Previous mapping (MacFarlane, 1986) hypothesised the presence of a fault or shear zone within the dam foundation due to a change in foliation attitude from the left to right abutment and

indeed the presence of the gorge in this location. However, the mapping carried out as part of this project (and the original maps of MacFarlane) did not support a change in overall foliation attitude and rather indicate that the outcrop on the right abutment was highly disturbed likely due to the proximity of the McNamara Fault.

Although mapping provided more information there was still some uncertainty on the condition of the dam foundation at depth. Therefore, four boreholes were carried out to assess the change in rock mass and material properties across the footprint of the dam. As Trustpower wished to continue generation as much as possible throughout this time, the three vertical bores (BH4-2 to BH4-4) were drilled (20 m into the foundation) from a floating barge adjacent to the upstream face of the dam. The fourth borehole (BH4-1) was drilled from scaffolding on the upstream side of the right abutment, and inclined at 60 degrees into the right abutment using PQ triple tube coring to improve recovery of the fractured rock mass.

Other than the upper 10 m of BH4-1, core recovery was generally good, exceeding 90 %. The upper 10 m of BH4-1 had total core recovery of only 61 %. There was no apparent correlation of RQD other than that generally the upper 10 m of each borehole were more fractured than the remainder of the borehole. Results are skewed across the dam footprint by a change in core diameter as the size was reduced to HQ in better quality rock. An improvement of rock mass quality was also noted from right to left across the dam. The weathering profile is deeper on the dam abutments (up to 13 m) than in the reservoir base where only the upper 2 m to 3 m of rock was weathered. Dark orange brown staining is evident on many joints within the weathered zone indicative of water flow.

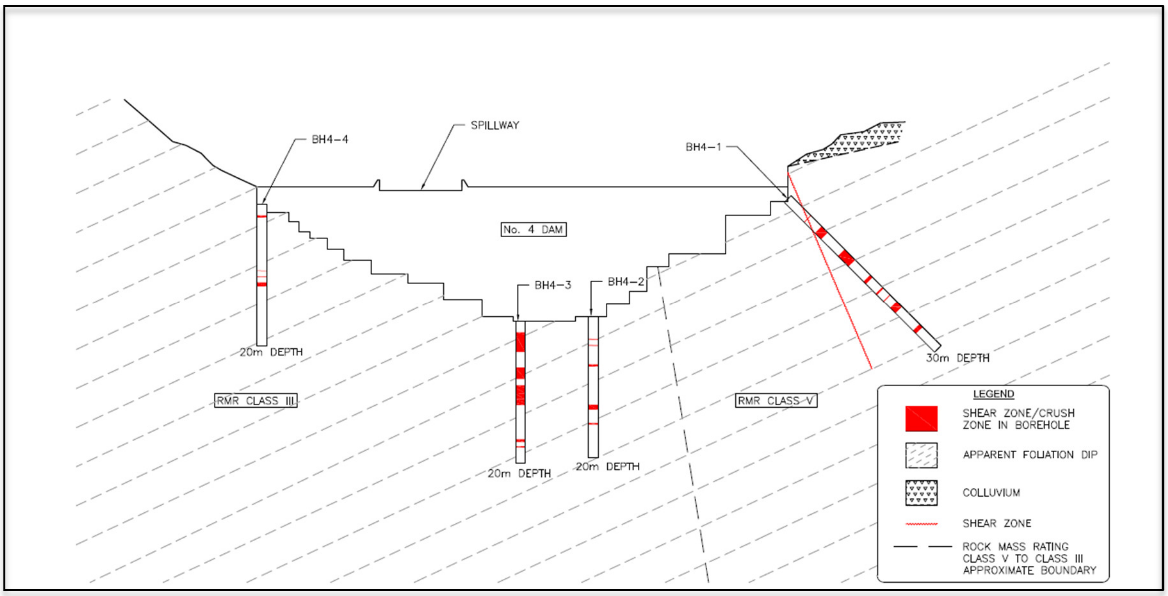

Figure 3: Cross Section of No.4 Dam showing encountered shear/crush zones.

Many crushed and sheared zones were noted on the borehole logs as shown in Figure 3. While it is possible that shear zones could daylight in the dam foundation (as proposed by MacFarlane 1986), in light of the recent investigation it was considered unlikely that one large crush or fault zone exists within the dam foundation, rather a number of smaller shears likely attributed to the proximity of the McNamara Fault.

<u>Concrete Coring</u>

An additional knowledge gap identified in the review relates to the strength of the concrete. Therefore, ten 140 mm diameter concrete core samples were obtained from the downstream face of each dam. Core samples were gathered by horizontally drilling into the downstream face of the dams using rope access contractors. Samples were obtained from intact concrete and across lift joints, as well as in areas known to have been grouted in the No.2 Dam strengthening works.

3.3 Laboratory Testing

Typical laboratory characterisation tests were carried out on both foundation rock and concrete cored from the dams. Additionally, elastic modulus testing and splitting tensile tests were conducted on the concrete, including tensile strength across construction lift joints.

Concrete Testing

Compressive strength, density and elastic modulus results were consistent with international studies on concrete strengths in dams greater than 30 years of age. All results showed concrete used in the older No.2 dam is generally of better quality than that tested from No.4 dam.

Splitting tensile strengths were converted to direct tensile strength using two methods, which gave consistent results of 1.5 MPa for No.2 Dam and 1.3 MPa for No.4 Dam. Test results over lift joints, where core was received at the laboratory intact and able to be tested, gave low tensile strengths. Due to the amount of broken sample a tensile strength of 0 MPa was used for lift joints.

Bedrock Testing

Density of the tested schist was consistent across the two dams with 27 kN/m^3 at No.2 Dam and 28 kN/m^3 at No.4 Dam. Unconfined Compressive Tests for intact rock strength ranged from 39 MPa to 60 MPa, with the mean being 51 MPa. The lower values correlated with rock sampled from the right abutment of No.4 Dam.

4 ASSIGNMENT OF MATERAL PROPERTIES FOR FEA

Material properties for the concrete within the dams were derived from site specific laboratory testing. For bedrock, however, it is not as simple as using the rock material parameters from laboratory testing, as testing was only undertaken on intact core samples and the results are not representative of the rock mass.

Hoek Brown failure criterion were derived for the right abutment of No.4 dam, however, this methodology was not considered appropriate for the remainder of the sites where foliation is dominant and foliation parallel joints sets tend to occur. Therefore, the Rock Mass Rating (RMR) System (Bieniawski, 1989) was utilised to take into account rock mass characteristics including defect spacing and orientation, Table 1 shows results.

Table 1: Rock Mass Rating (after Bieniawski, 1989)

Site	Strength	Drill Core Quality	Defect Spacing	Defect Condition	Groundwater	Orientation Adjustment	Rock Class
No.2 Left Abutment	7	17	15	25	15	-50	Class IV Poor
No.2 Foundation	7	17	15	25	7	-7	Class II Good
No.2 Right Abutment	7	17	15	25	4	0	Class II Good
No.4 Left Abutment	7	3	8	25	10	-5	Class III Fair
No.4 Foundation	7	3	5	30	10	-7	Class III Fair
No.4 Right Abutment	4	3	5	20	4	-25	Class V Very Poor

Using this system, the main areas of concern were highlighted as being the right abutment of No.4 Dam and the left abutment of No.2 dam.

5 FAILURE MODE ASSESSMENT

The investigations carried out at both dams have allowed the geological model to be verified, along with characterising the dam foundations and the dams themselves. Another input required

was to determine kinematically feasible potential failure modes at the dams to determine if there are any potential failure planes or interfaces that needed to be specially considered as part of the FEA. A check on failure modes was carried out per NZSOLD Guidelines Section 6.92.

Failure via basal shear was discounted at both structures. Arch dams transfer load predominantly into the dam abutments and therefore basal sliding was not seen as a likely failure mechanism. Additionally, there is an absence of low angle defects to act as a slide plane at both sites.

Stereograph interpretation and use of the programme SWEDGE by Rocscience were used to determine wedge failure potential at the dams. This was also determined to be a low risk at both sites, generally due to quite favourably orientated foliation and joint sets.

A potential sliding failure mechanism was identified for the left abutment of the No.2 Dam. Historical photos of No.2 dam shows there does not appear to have been movement on this abutment since completion of construction (Figure 4a), however, it is noted that since raising the dam, water does now flow over the left abutment during dam overtopping events (Figure 4b). FEA analysis should assess the risk of block failure in this location and any impact this might have on abutment and dam stability.

Figure 4a: Historical photo of No.2 Dam, showing potential slide block, arrow indicates foliation plane and dashed line indicates potential block of instability (date unknown).

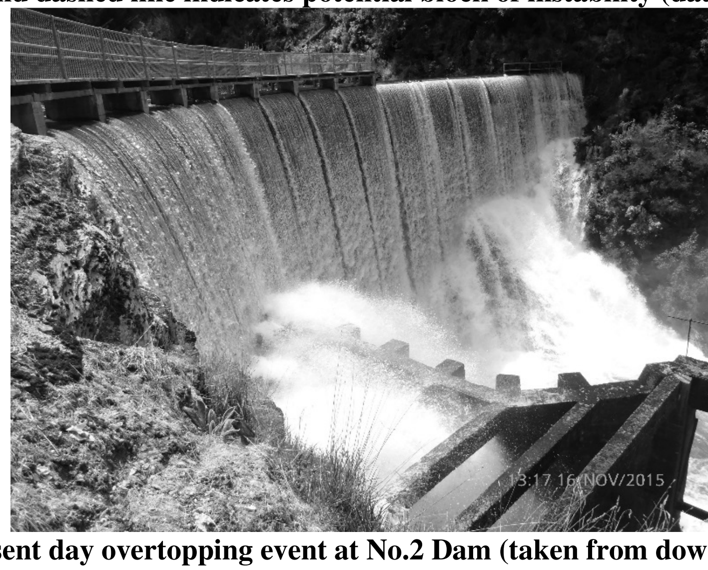

Figure 4b: Present day overtopping event at No.2 Dam (taken from downstream of right abutment, November 2015).

For the No.4 dam, the rock mass on the right abutment is highly jointed and sheared, and it is kinematically possible that a failure could develop through the jointed rock mass.

6 CONCLUSION

It can be a difficult and costly exercise to obtain the necessary inputs for modern day Finite Element Analysis in order to determine if ageing infrastructure meets today's standards for safety and stability. In the Waipori No.2 and No.4 Dam assessments, a review of available historical documentation and detailed geological mapping were both critical to carrying out an efficient and targeted intrusive geotechnical investigation. While investigation was still required to gain knowledge of rock and concrete strengths the main benefit of the intrusive investigation was providing ground truthing to the geological model in areas that could not be mapped (for example the reservoir foundation).

The definition of rock mass properties as a whole rather than just assessing intact rock strength will lead to more realistic inputs into the FEA. An assessment of kinematically possible failure modes using simple stereonet analysis allows advanced modellers to know which areas are of concern and which parts of their models they need to take a step further.

This study confirms that basic geological mapping and geological tools such as aerial photo interpretation, stereonet analysis and review of historical photos are still an important consideration in the world of 3D laser scans and complex modern day FEA. The outputs from advanced models will only be as good as the inputs gathered from basic site characterisation using traditional techniques.

7 ACKNOWLEDGEMENTS

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8 REFERENCES

Bienawski, Z., (1989) *Engineering Rock Mass Classifications: A Complete Manual for Engineers and Geologists in Mining, Civil and Petroleum Engineering.* New York: Wiley-Interscience.

Bishop, D.G., Turnbull, I.M. (compilers) (1996) *Geology of the Dunedin Area.* Institute of Geological and Nuclear Sciences, 1:250,000 geological map 21. Lower Hutt, New Zealand.

MacFarlane, D. (1986) *Geology of the Waipori No.4 Dam and Site Area.* Department of Scientific and Industrial Research.

Turnbull, I.M., Mortimer, N. and Craw, D. (2001) *Textural zones in the Haast Schist – a reappraisel*, New Zealand Journal of Geology and Geophysics, 44:1, 171-183.