Construction Implications from Geological Modelling at the Enlarged Cotter Dam site, ACT, Australia

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ABSTRACT

The Enlarged Cotter Dam (ECD) Project comprises the construction of a new 80m high roller compacted concrete (RCC) water supply dam, located 120m downstream from an existing 31m high, concrete gravity Cotter Dam, 18km west of Canberra, in the ACT, Australia. The dam site is located in a steeply-sided valley, with overall abutment slopes in the order of 45°, that have been eroded through a geological sequence of Early to Late Silurian Age, comprised predominantly of porphyritic rhyolite and lapilli tuffs of the Walker Volcanics. The geology and topography of the dam site constitute a distinctive environment for the construction of the ECD. An understanding of the geological model of a dam site can inform both design considerations and construction planning aspects. This paper focuses on implications for construction that have been identified through a review of the site's geological model. Implications drawn from geological modelling at the ECD site have been identified as applicable to the following key construction issues:

- Required depth of abutment stripping
- Useability of stripped abutment spoil
- Foundation excavation stability
- Forecasting stabilisation types and quantities.

Keywords: dam, roller compacted concrete, geotechnical investigation, geological model, foundations

1 INTRODUCTION

The Bulk Water Alliance (BWA) was formed in May 2008 to design and construct the Enlarged Cotter Dam (ECD). The Alliance partners are ACTEW Corporation (owners), GHD Pty Ltd (designers), and an Abigroup John Holland Joint Venture (constructors).

The ECD project consists of the following key components:

- An RCC dam, 80m high with a crest length of 335m;
- A 70m wide central uncontrolled weir spillway tapering to 45m;
- A fully lined stilling basin;
- A reinforced concrete intake tower on the upstream face of the dam;
- A steel outlet conduit connecting to the existing Cotter pump station;
- Two saddle dams (to 21m in height), constructed of earth and rockfill; and
- A reservoir with a capacity of approximately 78GL.

At the time of writing in 2012 the ECD is under construction, with RCC being placed.

2 REGIONAL GEOLOGY

A review of the published Canberra 1:250,000 and Brindabella 1:100,000 Geological Series Sheets, indicates that the ECD is located within an area of Late Silurian Walker Volcanics. The Walker Volcanics is described as predominantly comprising green to purple dacite, ignimbrite and bedded tuff. The Tarpaulin Creek Ashstone, which directly overlies the Walker Volcanics, is the lowest member of the Uriarra Volcanics. Structurally, the Winslade Fault is located about 800m to the south of the ECD site and trends approximately north east. The Pig Hill Fault trends approximately north south and is located approximately 4km north west of the ECD.

3 GEOTECHNICAL INVESTIGATIONS

Comprehensive geotechnical design investigations for the ECD were carried out between April 2008 and September 2009 and they included:

- 19 cored boreholes in the main dam site, totalling 1,102m of rock core;
- 243 in-situ permeability tests (packer tests);
- Installation of piezometers for groundwater level monitoring;
- 7 Seismic refraction surveys along and perpendicular to the ECD axis;
- Engineering geological mapping and statistical fracture surveying of abutments; and
- Geotechnical laboratory testing of rock cores.

4 GEOLOGICAL MODEL

4.1 General

A geological model has been developed for the ECD site using data obtained from core logging, permeability testing, rock strength testing, seismic refraction, geological mapping and statistical fracture surveys of both inclined oriented rock core and dam site rock exposures. Each model component is described below.

4.2 Soil and Rock Types

The valley floor at the dam site is characterised by alluvium comprising sand, gravel, cobbles and boulders with a depth or thickness in the order of 3m. The alluvium is underlain by moderately weathered bedrock.

The left abutment of the ECD site is virtually devoid of soil cover, with the exception of local deposits of colluvium comprising sandy gravel and cobbles. The right abutment has a semi-continuous colluvial mantle, with scattered bedrock outcrops.

The bedrock at the dam site has been subject to petrographic analyses, the results of which when reviewed with site exposures have suggested to the authors that the prevailing crystal structure and the 25% to 30% K-feldspar groundmass warrants the name porphyritic rhyolite.

4.3 Weathering and Strength Conditions

The Walker Volcanics has shown a clear relationship between degree of weathering and intact rock strength. Uniaxial compression strength (UCS) tests have been carried out on 52 core samples of rhyolite from the site. Figure 1 presents a summary of results. It can be seen that with the exception of highly weathered (HW) and only 3 moderately weathered (MW) materials, all samples tested below a depth of 4m had a UCS greater than 50 MPa. This value was adopted as a minimum dam foundation strength parameter, in moderately weathered, slightly weathered or fresh bedrock.

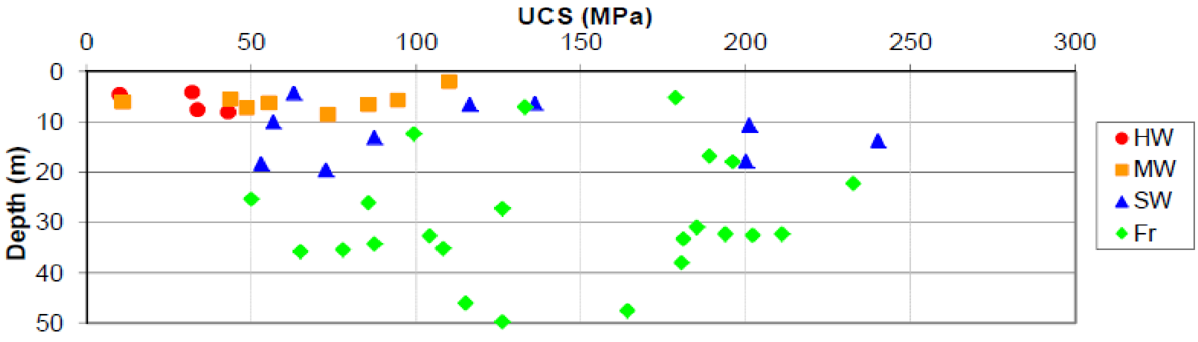

Figure 1. Uniaxial Compressive Strength versus Depth (mBGL) and Weathering

4.4 Geological Structure

Geological mapping at the dam site and a review of rock cores have identified the occurrence of three predominant rock mass fracture types in the site area, being joints, sheared surfaces/seams, and crushed seams/zones. The BWA has adopted seam and zone thicknesses as being less than or greater than 250mm respectively.

The orientations of all defects measured during geological mapping were consolidated into left and right abutment data sets. Defect data has been plotted as stereographic projections of poles to planes, which were subsequently contoured for defect set delineation (see Figure 2).

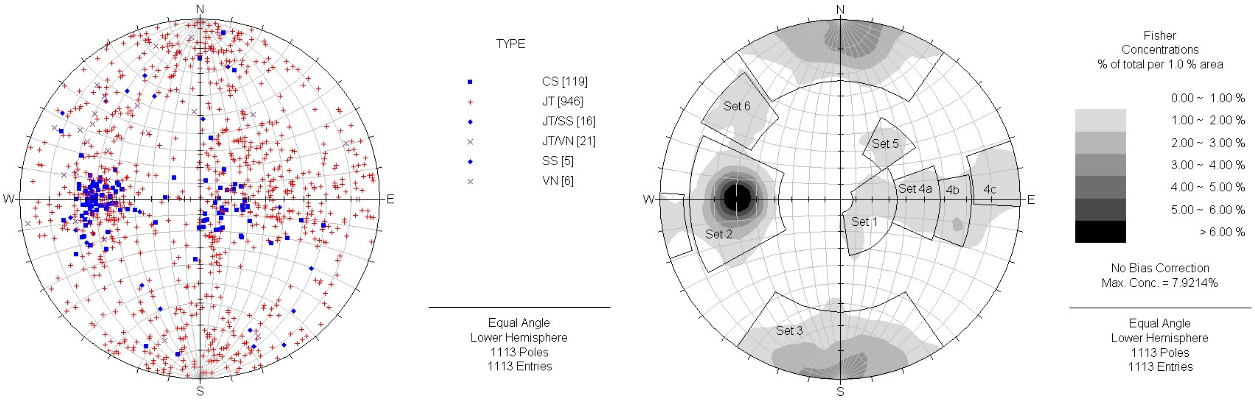

Figure 2. Stereographic projections of poles to planes (left abutment)

Analyses of data have enabled the delineation of seven defect sets, present on the ECD abutments, three of which were found to comprise prominent vertically and horizontally persistent crushed or sheared seams and zones, as summarised in Table 1.

Table 1: Left Abutment – Most Prominent Defect Sets and Orientations

Defect Set	**Mean Orientation (Dip° / Dip Direction°)**	**General Description**
Set 1	19 / 291	Major defect set that predominantly consists of joints and horizontally persistent crushed seams
Set 2	60 / 088	Major defect set that predominantly consists of joints and vertically / horizontally persistent crushed seams
Set 3	89 / 360	Major defect set that predominantly consists of joints with some vertically / horizontally persistent crushed seams

These three defect sets define the valley forming topography and prominent geological structure of both abutment slopes. It was apparent that the presence and intersections of these three defect sets would dominate the shape and treatment requirements of ECD excavations.

4.5 Groundwater and Permeability

Five piezometers were installed at the ECD site. Water levels were monitored regularly from June 2008 to establish an inferred phreatic groundwater surface.

Lugeon permeability testing was carried out over selected depth intervals in each of the cored boreholes at the site. A total of 243 permeability tests were carried out, the results of which have been grouped into the categories presented in Table 2.

Table 2: Permeability Categories

Lugeon Range	**General Rock Mass Conditions**
< 5	Relatively tight rock mass
5 – 10	Minor open fractures
10 – 50	More numerous or more widely open fractures(s)
> 50	Many open fractures, or prominent Set 1, 2 or 3 intersection(s)

Of the 243 permeability tests undertaken at the ECD, only 50 tests yielded a permeability of greater than 5 Lugeons. Rock core analyses confirmed a correlation between higher permeabilities (up to 194 Lugeon units) and relatively shallow intersections of prominent Set 1, Set 2 and Set 3 defects.

4.6 Geological Model Compilation

Graphic summary borehole logs delineating the geological and geotechnical conditions and features described above were plotted onto dam abutment sections, as were seismic velocity contours. Good correlations could be found between degree of weathering, rock mass strength, seismic velocity, prominent crushed and sheared seams and permeability. This technique was interpolated and extrapolated across the full ECD site area to compile the geological model, an example section of which from the left abutment is shown as Figure 3.

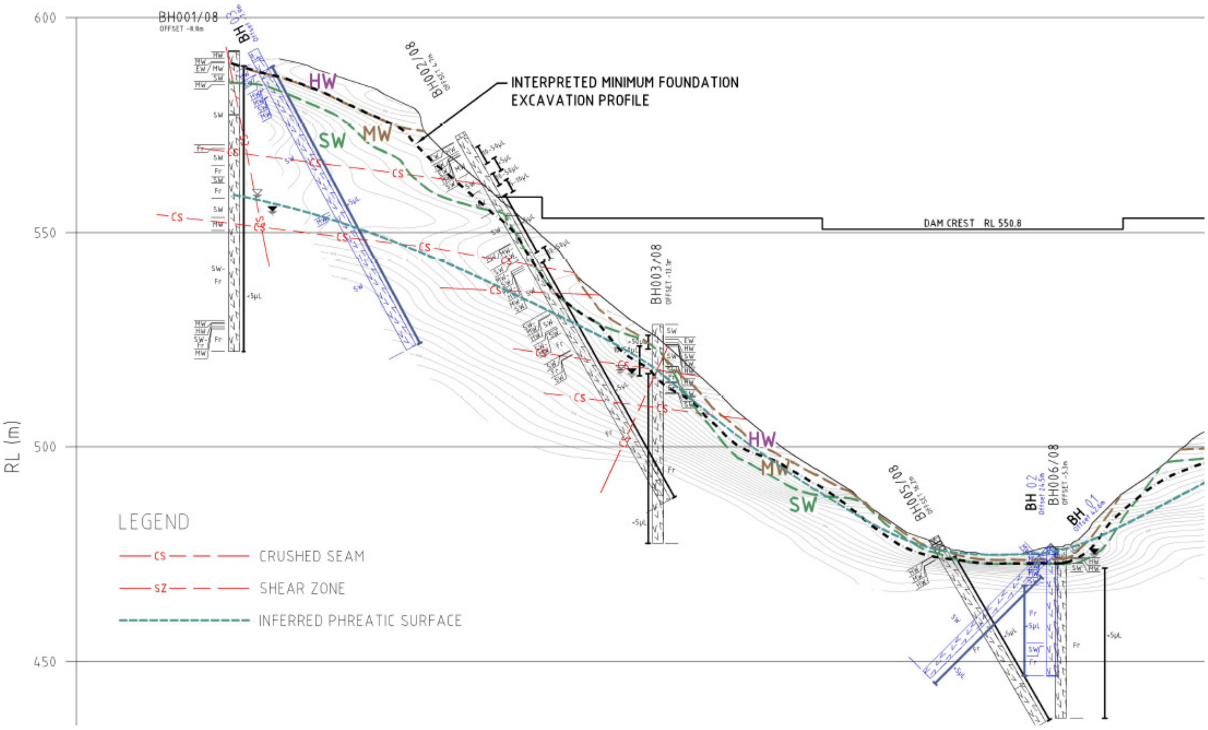

Figure 3. Left Abutment Geological Model

5 DESIGN CONSIDERATIONS AND CONSTRUCTION IMPLICATIONS

5.1 General

The geological model was in turn interrogated to enable decisions to be made on a number of key design and construction issues, as described below.

5.2 Required Depth of Abutment Stripping

Abutment stripping to achieve an acceptable dam foundation needed to satisfy both geotechnical design and construction practicality criteria, as follows:

- **Geotechnical Design Criteria** – to provide a kinematically stable foundation with suitable rock mass strength, stiffness and permeability characteristics on which to construct an 80m high concrete gravity dam; and
- **Construction Practicality Criteria** – to provide adequate access for plant and personnel to work safely and effectively on temporary excavation benches.

Abutment stripping sought to achieve the geotechnical design criteria listed in Table 3.

Table 3: ECD Foundation – Geotechnical Design Criteria

Parameter	**Expected Rock Conditions**
Degree of rock mass weathering	Generally moderately to slightly weathered
Intact rock strength	50 MPa or greater
Intact rock modulus	40 GPa
Nature and distribution of natural rock mass fractures	Minimal intersecting defects that will produce detached rock masses requiring either removal or stabilisation
Permeability	Typically less than 5 Lugeons – some local seams and zones that yield a higher permeability may require treatment
Seismic velocity	Ranging generally from 2000 to 2200m/s

An analysis of the geological model for the design criteria in Table 3 enabled the delineation of an interpreted "*minimum foundation excavation profile*", as shown on Figure 3. This profile defined the minimum depth and quantity of abutment stripping. It also became the basis for excavation design to satisfy construction methodology criteria, which focused on a minimum required bench width of 10.5m for a Caterpillar 330 excavator operating in its standard forward facing position. The resultant cut and fill design of temporary benches was referenced to the *"minimum foundation excavation profile".*

5.3 Reuse of Stripped Abutment Excavation Spoil

Due to the steepness of both ECD abutments, a series of access tracks were required to gain access to the dam foundation excavation. A total of 286,000m^3 of excavation spoil was to be generated.

The geological model was interrogated to estimate the percentage of excavation spoil that could be reused for general fill, rip rap, or crushed and used for concrete aggregate (Table 4).

Table 4: Potential Reuse of Abutment Excavation Spoil

Degree of Weathering for Excavated Materials	**Volume (m^3)**	**Percentage of Total Spoil**	**Potential Use of Abutment Spoil**
Soil to highly weathered rhyolite	123,100	43	General fill
Moderately weathered rhyolite	91,600	32	Concrete aggregate
Slightly weathered to fresh rhyolite	71,500	25	Concrete aggregate or rip rap

Beneficial reuse of abutment excavation spoil provided a significant cost saving for the project. In particular the volume of spoil used for concrete aggregate and rip rap products reduced the volume of rock required from onsite quarrying (Table 5).

Table 5: Beneficial Reuse of Abutment Excavation Spoil

Reuse of Abutment Spoil	**Volume (m^3)**	**Percentage of Total Spoil**
Waste spoil	50,000	17
General fill for roads and construction pads	72,000	25
Concrete Aggregate	145,200	51
Rip Rap	19,000	7

5.4 Foundation Excavation Stability

The geological model incorporates the seven rock mass defect sets mapped on the abutment slopes. The three most prominent defect sets in particular, intersect to create fracture-bound rock masses with a range of stability conditions and magnitudes.

Kinematic instability mechanisms were identified for each of the abutment excavation batters. Valley stress relief, rock mass weathering and construction blasting were all expected to contribute to loosening and increased instability, particularly near the crest of excavation batters. It was anticipated that fracture-bound failure masses up to 100m^3 would require either removal or stabilisation in place.

Batter angles and orientations were adjusted where possible to reduce the likelihood of sliding, wedge and toppling failures. Intermediate benches were incorporated into the excavation design to provide rockfall catch capacity. Rockfall modelling was used to refine the batter height to bench width ratio and identify critical zones where higher densities of rock support would be required.

The prominent Set 1, Set 2 and Set 3 defects were mapped to be persistent over distances of up to 130m and the potential for their intersections to form large kinematically admissible failure mechanisms in the dam foundation was also recognised, thus requiring modelling of foundation stability. Statistical fracture data from the geological model enabled the engineering characterisation of each defect set for the purpose of geotechnical stability analysis. The results of laboratory strength and shear box testing (both saw cut and natural defects) were used to determine defect strength parameters for various stress ranges. Acceptable factors of safety against sliding and overturning under a range of operational conditions were confirmed.

5.5 Forecasting Stabilisation Types and Quantities

Utilising the geological model, together with the results of kinematic analyses and rockfall modelling, a 3D excavation model was developed to forecast stabilisation types and quantities. The 3D model (Figure 4) was zoned according to criteria including geological weathering, strength and fracturing conditions; excavation methodology – blasting or mechanical; purpose – access track or dam foundation; and design life – temporary or permanent.

Provisional rock support types and quantities were estimated for each zone. Final installed rock support quantities were marginally lower than those forecast (Table 6). The utilisation of the geological model to accurately forecast support types and quantities facilitated construction planning and cost budgeting.

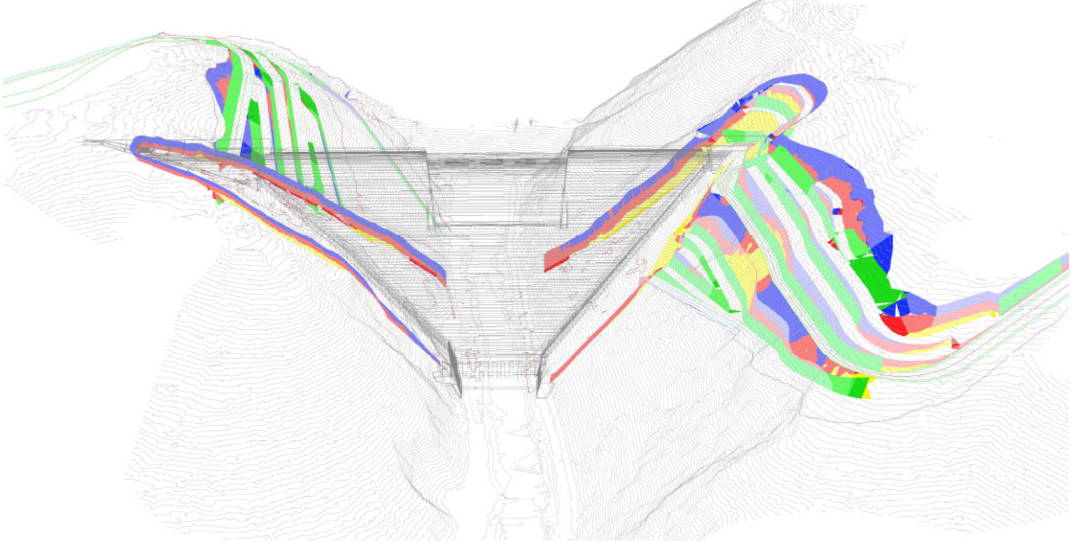

Figure 4. Perspective View of Dam Abutments from 3D model, looking upstream

Table 6: Forecast and Actual Rock Support Quantities

Rock Support Type	Forecast Quantity	Actual Quantity	Difference
Rock Bolts	2257 (No.)	1845 (No.)	- 18%
Structural Shotcrete	9503 m^2	8974 m^2	- 6%

6 CONCLUSION

The authors consider the Enlarged Cotter Dam project to be a notable case study that demonstrates the cost effective benefits that can be gained from the results of a rigorous geotechnical investigation and a thorough understanding of the geological model of a construction site. Teamwork by designers and constructors has enabled the optimisation of both excavation and foundation stabilisation components of this landmark project.

7 ACKNOWLEDGEMENTS

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