Sunwater Limited

Paradise Dam Safety Improvement Project

Technical Review Panel Report No 2

Technical Review Panel

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Status **Final**

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1. Introduction

The first meeting of the TRP was held in May 2019 and comprised the following members:

- [redacted]

For this second review workshop held on the 27 and 28th August 2019 the TRP has two additional members:

- [redacted]

[redacted] attended the meeting via Skye on 28th August, and [redacted] attended the 2 day workshop and visited the dam site on 29th August with personnel from Sunwater and GHD.

The TRP is appreciative of the reports provided before the workshop and the presentations by Sunwater and GHD at the workshop that has enabled us to gain a good impression of the key issues.

The agenda for the workshop is included in Appendix A and again we express our appreciation for the presentations and opportunity for the discussions and clarifications during the workshop.

It is our understanding that under current conditions the GHD (2019) dam risk assessment indicates that dam plots well above the ANCOLD tolerable risk line for an existing dam with societal risk being unacceptable. Options have been developed for a Preliminary Business Case for a dam improvement project that has primarily reduced to two options:

1. Option 2 – retain the current dam geometry and stabilise the dam with post tension anchors. Included with this option is and extended stilling basin with high sidewalls and a chute and sidewall to form a scour protected side channel spillway for flows over the right bank dam blocks that form a secondary spillway. Floods greater than the 1 in 1000 AEP event will activate the secondary spillway operation. This option as presented by GHD reduces the societal risk but not sufficiently to be below the tolerable risk line for an existing dam.
2. Option 3 – the spillway crest and hence normal storage is to be lowered by 10 m. Under this base scenario floods greater than a 1 in 15,000 AEP event will activate the secondary spillway. This lowering or base case scenario can give an immediate risk reduction at the dam to get just below the tolerable risk line. No upgrade works on the stilling basin or on the secondary spillway are included in the interim lowering. Other elements associated with Option 3 proposed by GHD include a longer primary stilling basin with training walls, erosion protection on the right bank for the secondary spillway flow and stability improvements for extreme loads with anchoring.

The TRP report generally follows the key heading of the GHD presentations at the two day workshop and observations form a one day site visit by Panel Member [redacted].

2 Geological / Geotechnical Update

2.1 Site Visit and Current Geological Investigations

Geological studies of the bedrock foundation are currently underway at the Paradise Dam. These include evaluation of recent drill works at the damsite, laboratory testing of rock core samples and detailed geological mapping. TRP geotechnical engineer [redacted] visited the site on August 29, 2019 as part of the August 2019 TRP evaluation.

The consultant is carrying out a thorough evaluation of the bedrock conditions. At the current time the field work is focusing on delineating the distribution of the basic geology units throughout the site. This work will provide input for the 3-Dimensional model that is being produced. A program of laboratory strength testing is being carried out on selected rock core samples. Currently, however, there is no systematic program of rock mass classification to determine overall rock mass strengths. This classification would be based on either Barton's Q rating or the Hoek GSI ratings and would input rock mass parameters such as fracture spacing, condition, roughness, etc. When combined with intact rock strengths the classifications ratings can be used to estimate overall rock mass strengths compatible with the scale of the dam foundation.

Observations and comments from the site inspection on 27th August 2019 are presented in the following paragraphs.

2.2 Site Geology and Geotechnical Conditions

The principal bedrock units that affect the dam foundation stability are shown in Photo 2.1 and described as follows:

- Barambah Basalt beds in the right bank and outlet channel area and in the foundation of portions of the right bank secondary spillway. This unit generally consists of competent, blocky rock with at least one closely fractured scoriaceous layer.

- Paleo gravel at the base of the basalt. This is a 0.1 m to 1.5 m wide bed of sandy gravel which underlies part of the right bank secondary spillway structure. It is assumed to be an almost continuous relatively weak stratum at the base of the basalt. The material has been thermally altered by the overlying the basalt and appears to have weak intergranular cementation.

- Tectonically disturbed meta-arenite and meta-siltstone of the Goodnight Beds. This is the dominant rock unit of the project area. It forms the foundation of the primary spillway and a large portion of the right bank spillway and intake structures. It is exposed in widespread outcrops in the left bank and along much of the riverbed/valley floor downstream of the dam. Intact samples meta-arenite and meta-siltstone when unaltered, are strong to very strong. However, the rock mass is highly tectonized. It is variably fractured with several shear zones and large zones of crushed and/or brecciated rock. Variably sheared rock near the Paradise Fault is shown on Photo 2.2. The relative severity of fracturing, brecciation and shearing is the dominant factor in determining the overall mass strengths and other geotechnical parameters of various zones in this rock unit.

- Near-surface weathered rock: Surficial weathering has produced a surface zone of chemically altered, weakened rock throughout the site. Both the basalt and the Goodnight Beds are affected. The moderately weathered zone, which is generally in the range of 2 to 10 m thick, has significantly lower shear strength than deeper unweathered or slightly weathered rock.

The ongoing drilling and surface mapping geological investigations are appropriately focusing on the distribution and orientation of shear/brecciated zones and other weak features throughout the site. This is being done by experienced geologists who are assessing surface and subsurface information with a commendable amount of detail. This data is being used to produce a three-dimensional model using Leapfrog software.

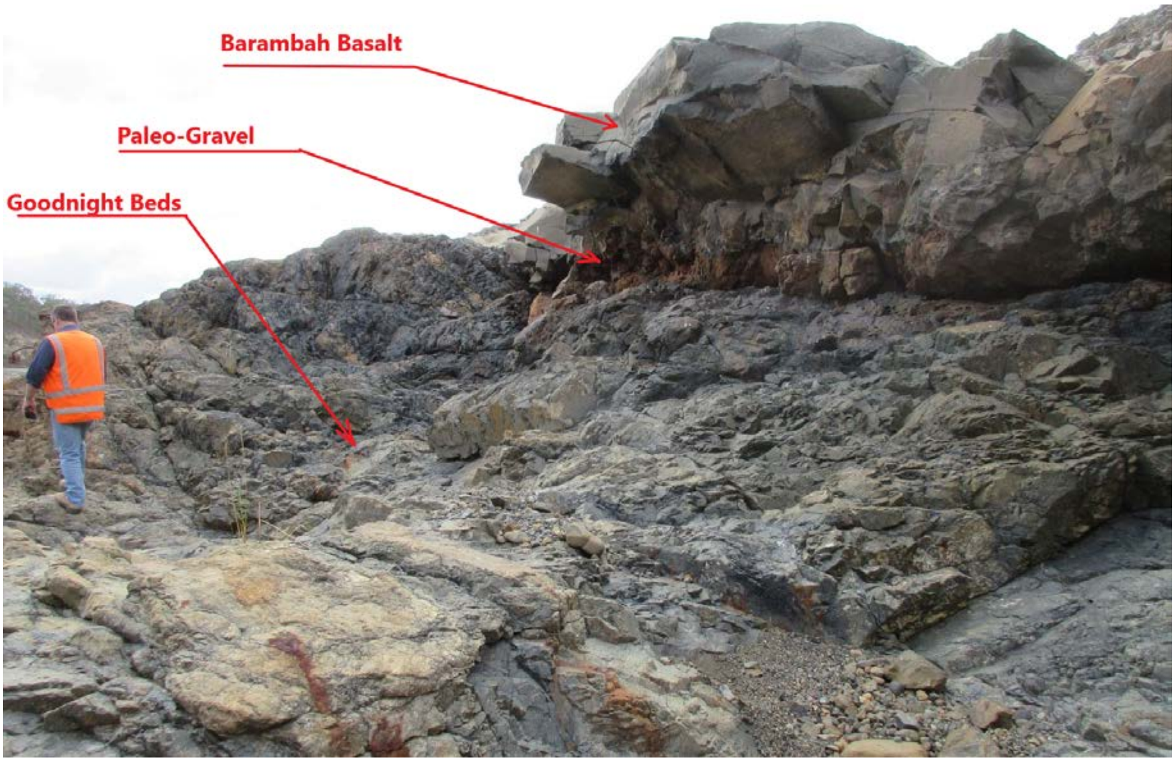


Photo 2.1: Bedrock outcrop on the right side of the river showing three of the primary geological units. A basalt bed is shown at the top of the outcrop. This underlain by the rusty colored, 20 to 30 cm thick paleo-gravel. The Goodnight Beds, with their chaotic fracture patterns, are visible at the base of the outcrop. Despite the variable fracturing of these beds are sound with fair resistance to erosion. Note the curved, sub-horizontal to gently inclined, en-echelon joints in the Goodnight Beds.

The consultant has defined several geological domains (green, orange and red) that have relatively uniform geotechnical properties. This system is based on the rock mass engineering geology of the rock mass and is very appropriate and useful. However, this is a relatively large scale classification and the current investigation program should ensure that it assesses smaller scale features that constitute stability hazards to the stability of the water retaining structures. These include:

- Presence and impact of surficially weathered and weakened material in the foundations. This is discussed in Sect 2.4 below.

- Impact of the paleo gravel under the right bank structures. This should be viewed as a relatively weak strata which could host localized foundation sliding. Sliding stability of impacted blocks needs to be assessed in this light, if not already done.

- Distribution and orientation of faults and shear features, particularly those related to the Paradise and Apron faults . This work should include the assessment of shear breccia zones and crush associated with the faults. It is important to determine the outlines and percentage of sheared/broken rock zones under the various monoliths and in the potentially erodible riverbed downstream of the dam. This work is well advanced and is a key focus of currently ongoing geological mapping.

- Distribution of subhorizontal and gently inclined discontinuities (both joints and faults) which may pass through the rock at the base of the dam. There are numerous en-echelon subhorizontal joints which might form composite bedrock sliding planes (Photo 2.1). These features should be evaluated as potential sliding planes.

- Condition and “as-built” morphology of the RCC/bedrock foundation of the dam. If not already done, this should be plotted on profiles of the dam together with assumed strength parameters to determine if RCC/rock sliding analyses as needed.

- Continue erodibility assessments of the abutments and riverbed areas downstream of the spillways

- Outline planar features associated with the Apron Fault zone. This feature underlies the dam and ramps up towards the downstream area. There are various planar zones that are roughly parallel to this zone. These kinematically viable dam foundation sliding planes for dam failure.

Photo 2.2: Variably sheared meta-arenite and meta-siltstone in the left bank near the Paradise Fault. Shear foliated dark grey carbonaceous meta-siltstone is interlayered with 10 to 20 cm wide nodules and layers of light-colored meta-arenite.

2.3 Requirements for Bedrock Foundation Geotechnical Parameters for Stability Assessments

It must be born in mind that the principal objective of the geological investigations is to provide shear strength, deformability and erodibility values that can be used by the design engineers for various analyses. It is understood that GHD is carrying out a rock testing program on samples of rock cores. It is also understood that no overall engineering rock mass classification work is currently being carried out, either for individual zones/features of the various domains.

Laboratory strength and index testing of intact samples, as currently planned, provide only a portion of the required data. Rock mass behaviour is the sum-total of rock strength together with the spacing, condition, roughness and alteration/infillings of the fractures. In a variably fractured rock mass, like that at the Paradise Dam, overall rock mass strengths will be only a fraction of the intact rock properties. The prevalence of shearing and faulting controls overall shear strength. Generally, completely disintegrated fault breccia will have soil mechanics parameters characterized by low to moderate friction angles and very low or no cohesion. Closely broken or sheared rock with tightly interlocked blocks and fragments will behave as a weak rock mass rock with moderate friction angles and varying amounts of cohesion. A blocky, competent rock mass, that is relatively unaffected by shearing, will have significant shear strength with high friction angles and high cohesion.

The determination of rock mass strength from field parameters is an inexact science and the current state-of the art practice is to use rock mass classifications that integrate rock strength and fracturing data. The Geological Strength Index (GSI) is suggested for this purpose since it is supported by widespread case histories and has been in widespread use for more than 25 years. Representative rock mass strength and deformability properties can be computed by the Hoek Brown relationships using this index. It is suggested that ranges of GSI be assessed for each domain and mappable zones within a domain.

The “intact” strength of the rock material is an important geotechnical property for foundation stability evaluation and is a major component of the Hoek Brown based rock mass strength assessment. Because of widespread disturbance of the Goodnight Beds, this not straight-forward. The following points should be kept in mind when determining “intact” strength of drill core samples:

a. **Sand/gravel sized fault gouge:** As shown on Photo 2.3, this consist of completely sheared/crushed material which has cohesionless, granular behaviour. Shear strength of this material should be evaluated by soil mechanics criteria.

b. **Weakly to moderately strongly healed breccia:** Photo 2.4 shows weakly healed breccia. Fracture infilling consist of silica and silt material which is porous and v weak. Samples are delicate and can shatter into breccia fragments when handled. This material usually cannot be accurately tested by an unconfined compressive strength (UCS) test. Testing can be done only when there is good sample confinement for the weakly bonded fractures. Carefully executed triaxial tests can give a representative strength envelope.

c. **Strongly healed broken rock**: Photo 2.5 shows a healed fractured rock that is held together by strong silica veinlets. Generally, this category of rock can be tested by conventional unconfined compressive strength tests. Strongly bonded veinlets should not be counted as fractures when doing RQD assessments.

Photo 2.3: Borehole PD-01. Typical sandy gouge with gravel size fragments

Photo 2.4: Borehole PD-01: Weakly healed breccia. Fracture infilling consist of silica and silt material. is porous and very weak. Sample breaks in breccia fragments when handled

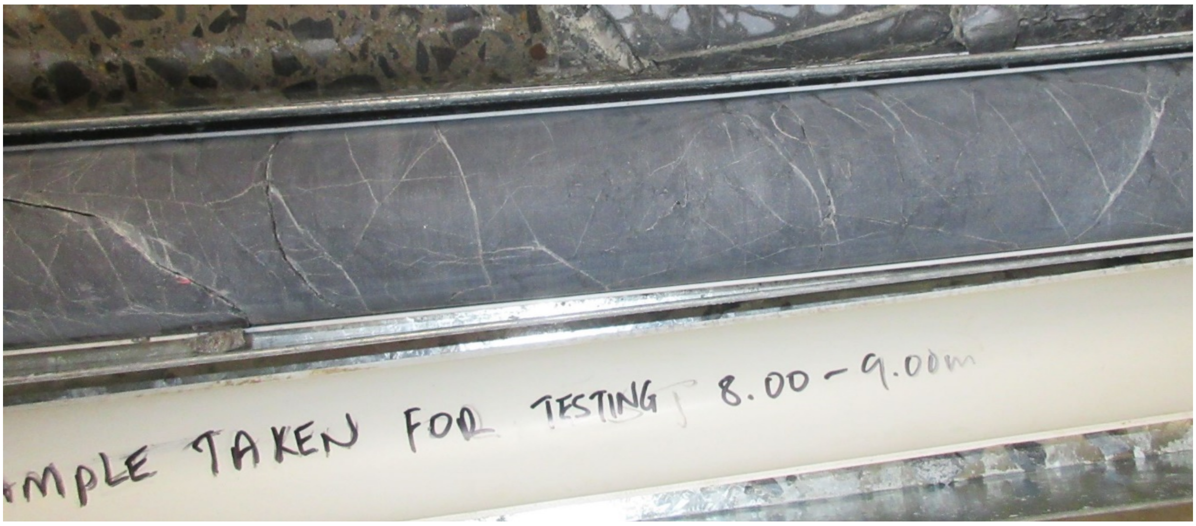


Photo 2.5: Borehole PD-01: Moderately to strongly healed meta breccia. Fractures are healed with silica veinlets.

2.4 RCC/Rock and Concrete/Rock Contact Conditions

The 2019 exploratory boreholes sampled the RCC/bedrock contact at numerous locations. The condition of this contact directly influences sliding stability of the dam foundation. Table 2.1 summarizes the depths and conditions of the RCC/concrete contacts as sampled by the boreholes.

In the secondary spillway, three (PD-04, PD-06 and PD-12) of the five boreholes of the secondary spillway showed that the base of the RCC is sitting on a poor quality moderately weathered rock. The thickness of moderately weathered rock beneath the contact varies from 2 to 6 m in these boreholes. In the remining two boreholes, PD-10 and PD-11, showed that the secondary spillway is founded on fair to good quality, slightly weathered rock. Borehole PD-05 in the spillway crest shows that the structure is founded on poor quality moderately weathered rock.

All three of the spillway apron boreholes cored a tight concrete/rock contact with good quality slightly weathered rock in the foundation.

Table 2.1: RCC/Rock contact in the 2019 boreholes as interpreted from core photographs by Stantec. The depths were scaled approximately from the photos and are subject to minor error.

Borehole Number	Location	Depth to RCC/Rock contact	Description
PD-01	Primary Spillway Apron	2.80 m	Tight contact, sound SW rock
PD-02	Primary Spillway Apron	2.06 m	Tight contact, sound SW rock
PD-03	Primary Spillway Apron	1.84 m	Tight contact, sound SW rock
PD-04	Secondary Spillway	10.06 m	Open contact on weathered rock surface. Moderately weathered from 10.06 to 12.05 m depth
PD-05	Spillway Crest	6.33 m	Tight contact with EW/HW/MW rock. Rock is moderately weathered from 6.3 m to 18.0 m depth
PD-06	Secondary Spillway	5.94 m	Tight contact with MW rock. Rock is moderately weathered from 5.94 m to 9.74 m.

Borehole Number	Location	Depth to RCC/Rock contact	Description
PD-07	Sky Slab	6.59 m	Open contact with sound SW rock
PD-08	Sky Slab	0.85 m	Open contact with poor quality HW/MW rock. Rock is moderately weathered from 0.85 m to 1.60 m
PD-09	Sky Slab	0.43 m	Open contact with good quality SW rock
PD-10	Secondary Spillway	10.0 m	Open contact on fair quality SW/MW rock
PD-11	Secondary Spillway	9.20 m	Open contact on fair quality SW rock
PD-12	Secondary Spillway	8.70 m	Open contact on poor quality, closely fractured MW rock. Rock is moderately weathered from 8.70 m to 13.0 m depth (bottom of borehole)

3 RCC Construction and Joint Shear Strength

3.1 Basic Concept of Rapid and Economical Construction of Concrete Dams

Early contributions to the concepts of RCC dam design and construction were being researched and formulated by engineers in academia, public agencies and professional societies in different parts of the world in the 1960's. A seminal meeting titled "Engineering Foundation Conference on Economical Construction of Concrete Dams," was held in Pacific Grove, California at the Asilomar Conference Grounds, May 14-18, 1972 and cosponsored by American Concrete Institute, American Society of Civil Engineers, and United States Committee on Large Dams.

Many of the basic principles of Roller Compacted Concrete (RCC) concrete dam design and construction used worldwide in practice today were discussed and debated at that meeting. Common to all the ideas expressed was the transformation of building concrete dams as individual vertical monoliths to constructing concrete dams in horizontally in successive continuous shallow lifts of 12 inches (30 cm) from abutment to abutment. A fundamental element of RCC technology regardless of the approach espoused has been the subject of RCC mix designs and the critical element of bond between successive lifts of RCC. Early in the development of today's practice, two principle concepts were espoused regarding the design of the RCC mixes, low paste, low cementitious mixes versus high paste high cementitious RCC mixes. How bond is achieved differs between the two basic approaches but is critically important to the behaviour and safety of the finally constructed concrete dam. The presence of many lifts in a RCC dam requires that every lift surface be designed to have sufficient bond and associated shear and tensile capacity to sustain the applied loads on the dam and with reserve capacity sufficient to satisfy margins of safety required in the design criteria established by governing regulatory bodies.

Methods for achieving bond between RCC lifts differ between high paste RCC mixes and a low paste RCC mixes but common to both and critically important to achieving the target bond between lifts are the specifications governing post compaction and preparation of a completed lift surface before placing next lift on top.

Post Compaction – Lift surfaces post compaction must be (1) kept clean and free of all extraneous debris, (2) water cured and remain wet until next lift is placed and (3) free water must be removed leaving the receiving lift in a saturated surface dry condition at time the next lift is placed.

Lift Preparation - The condition of the receiving lift is governed by its maturity (elapsed time and ambient temperature existing between existing and next lift in degree-hours). Detailed remedies are specified when a completed lift's maturity exceeds specified limits in order to achieve the target bond strength. These range from wire brushing to hydro-blasting in order to restore a fresh surface common in high-paste RCC construction.

Paradise dam was designed as a low paste RCC dam with a cement content of 61-65 kg/m^3 and no flyash. RCC was typically placed in 310 mm lift heights. According to the original specifications and drawings, a bedding mix was to be placed with a minimum width of 500 mm at the upstream face and at cold joints as below:

- Type 1 Joints >500 ˚H and <36 hr – 25% of the lift surface
- Type II Joints >36 hr – 20% wider than Type 1.

(refer to 2004 cold joint treatment memo written by [redacted] following a site visit in 2004).

MEMO
Cold Joint Treatment & Criteria
July-August 2004 [redacted] Site Visit

The following issue has been discussed and agreed with [redacted].

Continued accumulation of global large scale lift joint test data for RCC has shown that lift joint maturity and the quality of the lift joint surface have almost no noticeable impact on friction. They do impact cohesion.

The probable friction values for various levels of inspection and construction quality have been adjusted to reflect the latest knowledge (FileMatPropBurnettRev5 dated 9 July, 2004).

Because the Burnett River dam essentially achieves stability with current friction values alone (and it will be supplemented by some cohesion even under the poor conditions and with mature lift surfaces), the requirement for added bedding at various levels of cold joint can be relaxed. It arguably could be eliminated for essentially all conditions, except where it is otherwise required for isolated regions of higher tensile stress, added cohesion, or for watertightness.

With all things considered, the current cold joint criteria of 500 degree hours but not over 36 hours for Type I cold joint and greater than 36 hours for Type II cold joint should remain in effect. However, the amount of bedding can be reduced by about half. The new requirement is to place bedding over the upstream 10% of the lift surface for a Type I cold joint and over 15% of the surface for a Type II cold joint.

[redacted]

The TRP is aware from the Deign Report that the shear strength parameters assigned to the lift joints at Paradise dam were:

1. Good Joints c=325 kPa and Phi = 40.4˚
2. Poor Joints c =250 kPa and Phi = 35˚

The values were derived from [redacted] global database for joint strength test results and a quality rating of lift joints to establish a Lift Joint Quality Index (LJQI).

The sentence that the “dam essentially achieves stability with current friction values alone” does not state what friction angles are assumed as current values, nor does it indicate the acceptance criteria for stability, or the test data that justifies the comment. The design report is silent on any sensitivity test for sliding stability using a friction only or residual strength value on lift joints.

Internationally recognised gravity dam safety guidelines at the time allowed for a friction only or residual strength assessment for the sliding factor of safety. FERC Chapter 3 (2000) required a factor of safety of 1.5 for the worst-case static load and 1.3 for a PMF extreme load. The Canadian Dam Safety Guidelines (1999) required a residual sliding FOS of 1.5 for usual loads and 1.3 for the Inflow Design Flood. The USACE (1995) Gravity Dam Design Manual also allowed a sliding factor of 1.3 for extreme loads but noted that the sliding factors of safety are based on a comprehensive field investigation and testing programme.

Current gravity dam guidelines allow for sliding factors of safety using residual shear strengths such as:

1. Canadian Dam Association (CDA) Guidelines (2007) Friction only Usual loads ≥1.5, Unusual loads ≥1.3, Extreme Loads ≥ 1.1
2. ANCOLD Gravity Dam Guidelines (2015) Residual strengths well defined minimum strengths Usual loads 1.5, Unusual loads1.3, Extreme Loads 1.1

Initial drilling in 2015 to extract RCC core for shear testing came from hole DD600 drilled on the left abutment and Hole DD601 drilled on the right abutment. Four horizontal holes were drilled at the toe of the spillway and a sample G1B1 at CH340 was tested in shear. GHD has supplied photos of the core for these holes and a sample F1B from Block F, but it appears that no shear test sample was prepared from Block F or other spillway blocks.

More recently samples in 2019 have been obtained from seven 150 mm diameter horizontal cores in the downstream face of the secondary spillway and left abutment. Much of the core showed unbonded lift surfaces. Three samples that had bonded lift surfaces have been tested and testing of a further three samples is underway. Each sample is tested to get a single stage peak bond strength, a three-stage test to get a peak unbonded strength and a three-stage test to get a residual unbonded strength.

The combined data from 2015 samples and 2019 samples tested to date using a statistical approach to calculate a 97.5% confidence level strength that is exceeded results in a residual friction strength of 39.3˚ for the stress range of 200 -600 kPa. This stress range is applicable within the dam stability analysis.

GHD has also examined the core and logs from a hole drilled in 2006 to investigate the RCC condition in the dam after construction. The hole diameter is 146 mm and intersects approximately 108 lift surfaces with 78% logged as having no bond, and 22% logged as “good bond” or broken by drilling. The unbonded joints were typically smooth and segregation was commonly evident in the cores. GHD have not sighted any shear strength test results for this core.

Based on the vertical cored holes from 2006 and more recent drilling GHD summarised the lift joint quality as:

- There are likely to be segregated/unbonded zones extending across lifts
- Unbonded joints present a smoothed rolled layer at the top of the lower lift surface with no mechanical bonding from interlocking aggregates of the lower lift with the upper lift.

- Peak strengths from current testing are not considered representative of the lift surface as the strength comes from roughness of the joint created with the test.
- 2015 and 2019 testing show minimal difference from peak to residual strength on unbonded samples

GHD recommended the use of residual strengths to assess whether the stability on unbonded lift surfaces meets the ANCOLD criteria for gravity dams.

The TRP agrees with this approach of using residual strengths and also with comments from GHD during the presentation that the strain required to generate peak bond strength and residual strength are different and any attempt to average strengths is not meaningful as the joint will be unbonded when the residual strength is fully developed.

The TRP had the opportunity to visit the Trilabs laboratory and see samples from the direct shear testing and the horizontal core. Photo 1 below shows and example of an unbonded joint surface.

Photo 1 Unbonded RCC Lift Surface

The most recent RCC investigation helps to confirm that the sliding/shear failure along the lowest RCC joints is a credible failure mode. Core logging, cross-referenced with ATV and OTV, has confirmed a generalised unbonded condition of the RCC joints (around 80% of total) which is distributed along the full dam height. Previously a shear strength of 37° had been used for the preliminary stability analyses of the dam, but the figure was adjusted to 39.3° after considering adding the 2019 shear strength test data. With the information available to date, it is prudent, in the opinion of the TRP, to maintain the assumption of a generalised unbonded RCC lift joints with a shear strength of zero cohesion and an angle of friction in the 37°-39.3° range.

3.2 Construction Photos and Site Visit Observations of Technical Consultant During Construction

From construction records, specifications and recent communications with the original dam designers, it appears that the poor condition of the dam is not the result of using low paste RCC per se, but rather the result of poor construction practices for the treatment of the numerous cold joints in the dam, which were not rectified in spite of evidence that poor bonding and segregation were identified on the early RCC lift joints.

A review of construction photographs made available to the TRP during meeting #2 suggests that the organization of equipment, personnel and RCC placing operations had the potential of contributing to the poor lift surface bond strengths. An orderly array of equipment operating at a single front for delivery of RCC, spreading by dozer and immediate compacting is the preferred organization on the active lift surface. Placing and dozing away from forms and abutments versus spreading and dozing toward forms and abutments should be avoided. There was evidence that multiple layers had been discontinued at several different locations which created multiple shoulders susceptible to poor compaction and the potential for "rock pockets" (uncompacted RCC) to be covered over and not properly prepared to receive the next lift. There were obvious large expanses of dried out surfaces with little or often no water curing visible. There were also areas where the old lift surfaces were being contaminated by traffic, dust and debris. Whether or not these conditions were mitigated before new RCC was placed on those surfaces cannot be ascertained from photos but suggests the possibility of those areas not having been properly prepared.

It has been difficult to find evidence of bedding mix on cold joints in recent and previous drilled RCC cores. Additionally, horizontally drilled cores extracted in the plane of horizontal lifts observed during a laboratory visit on 27 August 2019 exhibited dust and/or mud on the RCC joints suggesting little or insufficient surface preparation. Based on the observed condition of recovered horizontal lift surfaces and taking into consideration that the specifications did not require thorough cleaning of previously placed RCC lift surfaces that required bedding mix, it is questionable whether or not the application of bedding mix would have improved the bond between RCC layers.

4 Review of Dam Stability

The inputs and assumptions for the stability analysis are set out in the GHD (2019) preliminary Design report and are summarised below:

a) Self-weight of dam using a density of 2,400 kg/m^3

b) Upstream reservoir load assuming a hydrostatic distribution

c) Uplift – The design assumption was a reduction on pressure to 50% of headwater downstream of the facing panels. Data indicates that this may not be conservative for some spillway dam blocks. The TRP suggests that 80% headwater may be appropriate for projecting to unusual and extreme reservoir levels.

d) Downstream tailwater - an effective tailwater pressure equal to 80% of the tailwater depth has been adopted based on CFD modelling

e) Crest pressures- negative crest pressures adopted for the 1 in 10,000 AEP flood and PMPF based on CFD modelling

f) Silt pressure – assumes silt build up to EL 38 m AHD with a unit weight of 18 kN/m^3.

The loads considered include FSL (usual), 1in 100 AEP flood (usual/unusual), 1 in 2000 AEP flood (unusual), 1 in 10,000 AEP flood (extreme) and PMPF (extreme).

GHD has presented stability results for spillway Block H for a lift surface at elevation 32.4 m AHD. The ANCOLD sliding stability assessment criteria for a well-defined residual strength is to achieve a minimum FOS for usual loads of 1.5, for unusual loads of 1.3 and for extreme loads of 1.1.

"Well-defined" means a sufficient number of tests have been done on concrete core from the dam and lift surfaces to give the strength parameters with reasonable certainty (e.g. assumed strength is exceeded by 80% of the test results from a test regime involving a significant number of tests). For new dams tests would be carried out on samples made in trial concrete mixes and confirmed by tests on core samples taken from the dam during construction.

The TRP recognises that most of the shear strength testing is clustered over towards the right bank spillway area with less testing on the left abutment and one sample from the toe of block G. The TRP is comfortable with using the residual strength parameter derived by GHD from all the test results to date to answer the question as to whether the existing dam complies with ANCOLD stability criteria and whether some immediate action should be taken to reduce the risk profile of the dam.

For any long term assessment of upgrade works to accommodate extreme floods the TRP suggests Sunwater and GHD obtain more core from the spillway, ideally from areas with and without bedding mix for shear strength testing to advance the "well-defined" residual strength parameters for the spillway blocks.

The stability analysis results are best summarised in the slide presented as Fig 1 below.

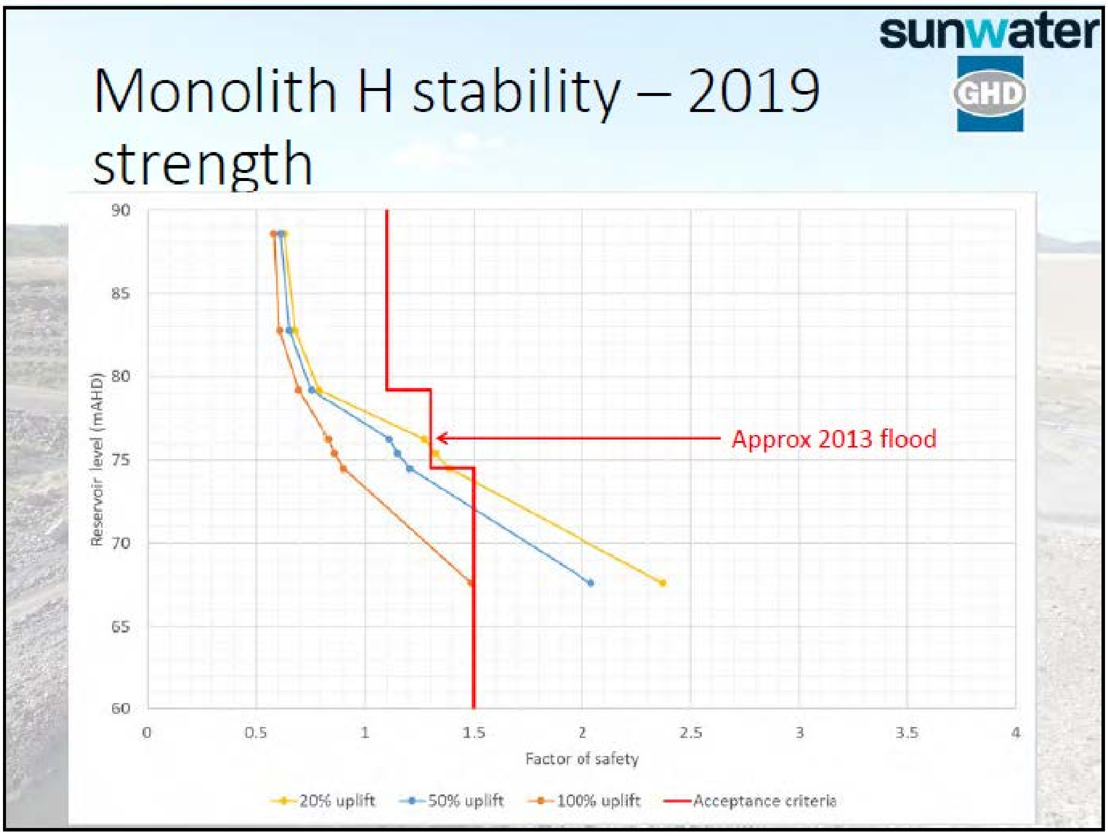


Fig 1 Stability Summary

On the basis of GHD's analysis results using the frictional strength of 39.3˚ presented in Fig 1 above the TRP notes:

a) At FSL the stability meets ANCOLD criteria with uplift in the range of 50 to 70%.

b) At the 1 in 50 AEP flood (EL 74.5 m) with uplift in the range of 50 to 100% the ANCOLD stability criteria for a usual load is not meet

c) At the 1 in 100 AEP flood (El 75.4m) with 50 to 100 % uplift the ANCOLD criteria for an unusual load is not met

d) At the 1 in 2000 AEP flood (EL 79.2 m) and beyond the ANCOLD criteria for an extreme load is not met.

At the 1 in 2000 AEP flood the friction would need to increase from 39.3˚ to approximately 50˚ to give a FOS of 1.1. Prior to the Canadian Guidelines (2007) and ANCOLD Guidelines (2011) the FERC and Canadian Guidelines required a FOS of 1.3, which would necessitate a friction angle of approximately 55˚.

The TRP requests that the extent of any base cracking that occurs in the above analysis is also noted in the stability output, along with the normal stress at the downstream toe.

A further stability memorandum from GHD was forwarded to the panel that included stability analysis for Monoliths C, N and various section levels for Monolith H.

The stability results indicate:

e) Block C will not meet ANCOLD acceptance criteria for sliding with reservoir levels above EL 85 m.

f) Block N will not meet ANCOLD acceptance criteria for sliding with reservoir levels in the range of 77 to 79 m depending on uplift assumptions

g) Monolith H base elevations in the range of RL 32.4 m to 54 m fall below the ANCOLD acceptance criteria for sliding as reservoir levels very from RL 72 m at the lowest surface to 77 m for the highest surface.

5 Update of Existing Dam Risk Assessment

The CRA currently indicates that the risk of the existing dam is unacceptable. Option 2 is also plotting above the limit of tolerability and therefore is also not acceptable at present. Option 3 is plotting less than a half order of magnitude below the limit of tolerability and therefore is also generally considered to be unacceptable based on the accuracy of the assessment. However, it is recognised that the CRA is largely still based on assumptions from the previous design phase and requires updating based on the findings of the current analyses.

A risk curve was presented for Option 3 called the “base case”. This considered lowering of the primary spillway crest only and excluded other works such as anchoring or stilling basin works. This creates a significant risk reduction relative to the existing dam risk profile and plots marginally under the tolerable risk line in a region where risks are tolerable if they satisfy the ALARP principle. The TRP suggests the risk profile is developed for the full scope of works that GHD proposes for Option 3 to enable comparison with the “base case”.

6 Option Scoping

Option 2 is reliant on large post-tensioned anchors at 3m to 4m spacing. Given the extensive areas of potentially weak and compressible materials to great depth in the foundation, there is a risk of anchors not having sufficient bond zone to take the load and interaction between anchors may occur. Staggering the bond zones and placing the anchors at an increased depth to extend the free length will help mitigate the risk of the anchors losing stress due to compression of the rock mass or interaction between anchors.

The possibility that large anchors may not be feasible indicates that buttressing options may be viable. If Option 2 is to be further explored, a review of the feasibility of the vertical anchors is required in the short term.

Comments made previously on anchoring:

- Strand comprising 7 wire, low relaxation with nominal diameter of 15.7mm and M.B.L. of 279kN are in common usage.
- The cone pull-out should be checked for cones starting at the top of the bond zone as most of the load during stressing will be taken up by the upper part of the anchor initially before distributing along its length.
- The free length may need to be increased so there is enough extension to allow the lock-off load to be easily reached during stressing. There should be at least 20mm extension for the 5% reduction of M.B.L. from the 75% test load to 70% lock-off load. (Note. 72% lock-off is also used commonly to give higher protection against relaxation over the life of the anchor from reducing below design loads). The anchor length should also have a transition length of about 2m between the free length and bond zone to achieve a safe take up of load into the bond zone.
- The number of strands in an anchor can be standardised to 65 and 91 strands to simplify construction (minimise likelihood of errors during fabrication) and provide redundancy for the future. The cost of additional strands is usually a small part of the total cost of an anchor.

The Option 3 scope proposed by GHD includes the following:

- Lowering of the primary spillway by 10 m (base case)
- 50 m long spillway stilling basin and training walls
- Post tension anchoring
    - Primary spillway (D-K) 91 strand anchors at 9 m centres
    - Monolith L 41 strands at 4.5 m centres
    - Monoliths M to R 41 strands at 9 m centres
    - Monoliths S to W 24 strand at 22.5 m centres
- Monolith strengthening for blocks D and K (already completed)
- Overlay slab for secondary spillway apron
- Other miscellaneous works

The TRP opinion is that interim lowering base case is an appropriate step to give immediate risk reduction but must be supplemented with additional works in line with the GHD scope.

The Ross River Dam Upgrade works is an Australian example where the spillway crest was lowered to give an immediate risk reduction before additional works to protect the embankment dam against piping failure modes and to restore the storage with a gated spillway were completed.

7 Preliminary Design and Trade Offs

As mentioned previously, the current designs do not show any protection where the flows captured by the secondary spillway discharges into outlet channel. Given the level of detail of some aspects of the options to date, some indication of the likely works in this area seems warranted.

A number of trade -off were presented for Option 2. The TRP notes that this is all hypothetical at this stage as Option No 2 is currently not indicating an acceptable risk profile relative to ANCOLD criteria.

The option to reduce the length of the stilling basin by using a pile cut-off to prevent back erosion (Option 2-2) appears viable and would be worth exploring in later design.

The option of downstream buttressing in lieu of anchoring is worthy of consideration as a fall-back option if there is a fatal flaw in proving the ability to use 91 strand anchors and prove a satisfactory zone of sufficient extent to bond the anchors to the underlying rock.

For Option 3 a shorter stilling basin and piled cutoff appears more costly from a preliminary assessment by GHD. The TRP considers it prudent to have a basin the extend beyond the apron fault to protect against scour (see following section)

GHD assesses that a buttress in lieu of anchors for Option 3 was not viable and recommended retaining the anchors. Due to potential limitations of the foundation rock for anchoring of post-tensioned ground anchors (still to be confirmed), it is suggested that the downstream buttressing of the spillway section of the dam be reconsidered as an upgrade alternative, in combination with or instead of, ground anchors. The use of 91 strand anchors at 9 m centres in the spillway monoliths needs to be proven to be feasible at the dam crest and within the foundation bond zone. Smaller anchors say 65 strand at 6 m centres may warrant consideration as the foundation investigations progress. The process of drilling anchor holes through the RCC, water testing and grouting creates a risk to the panels at the upstream face of the dam. Full casing of holes through the RCC may be required.

The complexity of the geology and geotechnical conditions of the foundation may result in varied rock mass and defect parameters under different areas of the dam, and possibly under particular monoliths. If such circumstances are confirmed, the final design of the upgrade option(s) may need to be tailored with rock parameters developed for each monolith individually (or for groups on monoliths founded in similar rock conditions), as it was the case for the recent analysis of Sturt River Dam in South Australia.

Similar to the point above, the upstream uplift recorded by piezometers seem to yield different drain effectiveness under different monoliths. Instead of the 50% uplift adopted in the analysis to date (which is a debatable assumption as some piezometers show uplifts up to 60% and 70% at some point in time), the uplift to be used in future design of the option(s) may need to be considered individually for each monolith or group of monoliths, using the recorded data of the closest installed piezometer. It is also suggested that the uncertainty of not having piezometric records during medium and large floods be considered in the adoption of the uplift for the future design of the option(s). The existing uplift data is likely appropriate for usual loads, however higher uplift is likely appropriate for unusual and extreme loads within the RCC. At the dam foundation interface the 100% uplift assumption is necessary due to the lack of foundation drainage.

8 Hydraulics and Scour

8.1 Summary of presented information

For the previous TRP report (No. 1), comments were provided on hydraulic modelling and scour assessment studies by GHD to date. Commentary regarding hydraulics and scour assessments are now provided in this present TRP Report (No.2) on the following additional material provided by GHD:

- GHD 2019 Paradise Dam Spillway Improvement – Preliminary Design - Response to Comments in TRP Report No 1
- GHD 2019 Paradise Dam Spillway Improvement – Preliminary Design Report Technical report for SunWater Limited July 2019.
- GHD 2019 Paradise Dam Spillway Improvement – Interim Lowering Detailed Design Report. Technical report for SunWater Limited July 2019

The Preliminary Design Report presents analyses for Option 2 (full dam upgrade) and 3 (partial spillway reduction).

For option 2, the following design hydraulic design features are included:

1. Addition of an extended spillway spilling basin.
   Based on CFD modelling, it was recommended that an extended spillway basin of 60m length would be required. Preliminary detailed design of the spilling basin slab and baffle blocks were presented.
2. Possibility of retrofitting a smooth chute to the spillway. Based on CFD modelling, the smooth chute offers some advantages, but requires further analysis
3. Construction of a secondary spillway side channel
   A side-channel wall on the secondary spillway would need to be up to 25m in height for containment of the 1:15000 AEP flood
4. Spillway stilling basin training walls to minimise risks of erosion of left and right banks

A scour assessment was presented for the Option 2 case, utilising comparative methods of Annandale (1995) and Pells et at (2016). Based upon this it was concluded that the risk of scour to exposed faulted zones or highly weathered zones was high.

For option 3, the following design hydraulic design features are included:

1. Reduction of the crest height and therefore hydraulics of the primary spillway.
   The CFD studies presented in “Interim Lowering Detailed Design report” assessed crest pressures and rating curves for the lowered crest option. It was considered infeasible to fit a classical ogee shape to the lowered crest, and the overtopping flows were found to separate from the downstream face for all discharges modelled.
2. Addition of an extended spillway spilling basin.
   Based on CFD modelling, it was recommended that spillway basin of 50m length (ie smaller than Option 2) would be required. Preliminary detailed design of the spilling basin slab and baffle blocks were presented.
3. Construction of a secondary spillway side channel
   The lowered primary spillway lowers design overtopping discharges on the side channel spillway compared to the current dam, or Option 2 designs. A side-channel wall on the secondary spillway would need to be up to 25m in height for containment of the 1:15000 AEP flood

4. Spillway stilling basin training walls to minimise risks of erosion of left and right banks. This design was the same as for Option 2, albeit with a shorter length to match the shorter stilling basin slab.

No separate scour assessment of scour for the Option 3 lower spillway case was made, but it is noted that the primary jet impinges upon the unlined rock directly downstream of the current spilling basin in some predicted floods, which is likely to be unfavourable for scour.

8.2 Review comments

GHD's "Response to Comments in TRP Report No 1" is essentially a position statement regarding the scope of works and type of scour assessments that have been undertaken and are further proposed. The TRP do not take exception to the views presented, although at this stage is seems apparent that further detailed scour modelling may not be required. The limitations of scour assessment methods were discussed in the previous TRP Report No 1, and it appears that a pragmatic response to scour risk is already evident, with respect to two arguments:

1. from studies to date it is evident that the existing spillway basin is too short
2. Geological studies and historical performance at the dam suggest that is would be prudent to not leave the Apron fault exposed.

It appears that further analyses would not alter these facts. The construction of an extended basin slab would address these facts. Further regions of scour protection, such as through rock bolting and / or dental concrete may be required in identified regions of poorer rock mass quality downstream of the extended basin may be required, and on-going monitoring of the spillway performance should be undertaken. It also seems likely that construction of an extended basin would have benefits for lowering the risk of some other identified dam failure mechanisms. It is unclear if these benefits have been examined in the analyses by GHD to date.

The CFD modelling presented in the Preliminary Design Report follows the methodology of previously presented studies and it is a suitable tool for assessing the rating curves, pressures and preliminary design of the spillway basin. While the basis for sizing of the secondary spillway channel is understood, it is questioned whether the secondary side-channel training wall is a suitable or optimal solution, as an extensive wall height is required that is largely inundated by tailwater levels, and appears to deliver large discharges onto the toe of the main structure. It is noted that an alternative design comprising cut-off piles, which attempt to limit the extents of headcutting below the secondary spillway has been mooted. This may be effective in limiting the upward (headcutting) movement of erosion, although there are two aspects of concern. Firstly, erosion that does develop against the piles, may form a linear gully against the piles, in the direction of flow back toward the dam. Various case studies have demonstrated that under such conditions, a positive feedback loop can develop which exacerbates erosion, as deeper channels attract more of the flow, causing further erosion etc (see discussion in Pells and Pells 2016, for instance). Secondly, as recognised in the TRP meeting, erosion behind the piles may result in the requirement for extensive repair work if the design flood conditions do occur. This may be an acceptable compromise, but it is questioned whether an alternative solution, to allow for dissipation and spreading of secondary spillway flows away from the structure may be available.

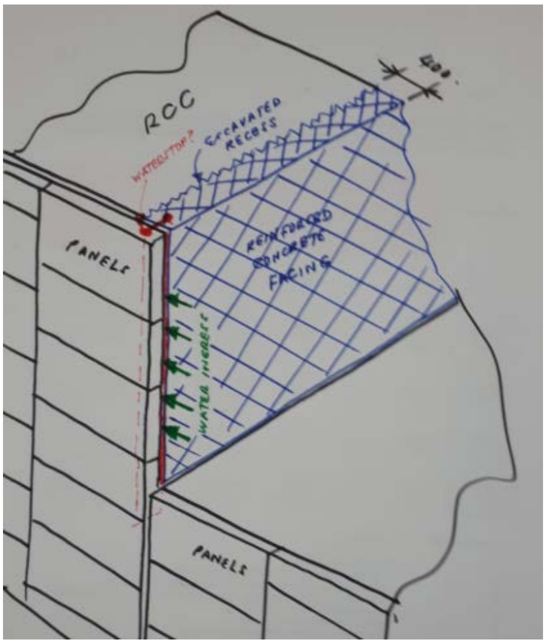
The analysis of scour presented in the GHD Preliminary Design Report utilises unit stream power dissipation as a guide for erosion risk. Various studies have argued that unit stream power dissipation is a useful indicator for this, and the maps of unit stream power dissipation presented by

GHD, which are based on the product of bed shear stress and near-surface velocity from the CFD model are valid indexes for visual analysis. However, unit stream power values derived in this way differ from the methodology used in various comparative erosion techniques, such as Annandale 1995 and Pells et al, 2016. As these erosion methods are ones of comparison, it is necessary to use a compatible method of stream power dissipation analysis, which considers dissipation of the total energy head. Values derived in this way will be significantly higher than those presented from the CFD model by GHD, but will also show little spatial variation across the spillway domain. In short, these methods would only provide enough guidance to show that it is necessary to protect the Apron Fault and other more highly weathered of fractured zones, but much of the Goodnight Beds will offer acceptable resistance to the imposed erosive power, for many design flood scenarios.

9 Interim Lowering Design

The interim risk measures involve lowering of the primary spillway by 10 m, the same reduction envisaged for Option 3, but with a flat horizontal rather than an ogee shape. The TRP questioned whether consideration was given to implementation of the ogee shape of Option 3 for the interim lowering, after which rest of Option 3 (anchoring and apron extension) could be completed at a later stage. SunWater explained that the priority of the interim lowering implementation was to minimise the exposure during construction time, which is obviously faster for a flat horizontal shape than it could be for an ogee shape spillway.

An important detail has not been considered (or has not been finalised) as part of the interim lowering. The vertical 400 mm recess on the sides of the demolished ogee, which is to be reconstructed with a reinforced concrete (RC) facing, will be located downstream of the existing face panels. However, the elaborated waterstop arrangement presented do not include vertical waterstops to prevent ingress of water at the interface between the new reinforced concrete facing, the Carpi membrane and the upstream panels. Ingress of water on this interface must be prevented (possibly using a vertical waterstop between the upstream panels and the new RC facing) as such ingress could lead to pressurisation and potential popping-up of the panels, with the subsequent loss of uplift reduction that is crucial to the stability of the dam.



TRP member [redacted] provided SunWater on 23 August 2019 a separate review of GHD's "Paradise Dam Spillway Improvement – Interim Lowering - Detailed Design Report, 4132235". The review included the observation made above and other comments and recommendations.

10 Concluding Statements

The TRP key concluding statements and opinions based on presentations by GHD are:

1. Paradise dam in its present state has a risk profile that plots well above the ANCOLD tolerable risk criteria and does not meet industry standards for gravity dam stability.
2. Option 2 dam improvement concept does not meet ANCOLD tolerable risk criteria and as such is not an acceptable dam safety improvement option.
3. Option 3 base case consisting of just a 10 m lowering provides a significant risk reduction relative to the existing dam situation. The risk profile that GHD presents is marginally under the tolerable risk criteria but the dam still remains vulnerable to other failure modes.
4. The Option 3 full scope of works proposed by GHD are considered by the TRP to both prudent and practicable risk reduction methods to complement the initial 10 m lowering.
5. GHD should produce an updated Option 3 risk profile based on the full scope of works including works associated with the primary spillway and training walls, secondary spillway scour protection works and anchoring to improve stability for extreme flood loads.

11 References

ANCOLD (2013) ANCOLD Guidelines on Design of Gravity Dams

Annandale, G.W., 1995. Erodibility. Journal of hydraulic research 33, 471–494.

CDA (1999) Dam Safety Guidelines

CDA (2007) Dam Safety Guidelines

FERC (2000) Chapter 3 Gravity Dams (Draft)

GHD (2019) Paradise Dam Spillway Improvement Project; Preliminary Design Report, July 2019

GHD (2019) Paradise Dam Spillway Improvement Project ; Paradise Dam Geotechnical Model Report, May 2019

PELLS, S., DOUGLAS, K., PELLS, P.J.N., FELL, R., PEIRSON, W.L. (2016) Rock mass erodibility. J.Hyd.Eng. ASCE. HYENG-9857 https://doi.org/10.1061/(ASCE)HY.1943-7900.0001243

PELLS, P.J.N., PELLS, S.E. and VAN SCHALKWYK, M. 2016 A tale of two spillways. Proceedings, 84th ICOLD Annual Meeting, International Committee on Large Dams 15 -20 May 2016, Johannesburg, South Africa

Sunwater (2019) Paradise dam ftp file

Exiting dam Drawings

Primary Spillway RCC photos

Right abutment foundation photos

Shear test Results

USACE (1995) EM1110-2-2200 Gravity Dam Design

Appendix A
TRP Workshop No 1 Agenda

Day 1 – 27th August 2019		
8:30-9:00	Arrive – visitor sign in	
9:00-9:15	Welcome, Safety/value share, introductions	Sunwater – [redacted]
9:15-9:45	Progress update	GHD - [redacted]
9:45-10:45	Geology update from previous TRP meeting	GHD – [redacted]
10:45-11:00	**Morning Tea**	
11:00-12:30	Dam stability incl. shear strength	GHD - [redacted]
12:30-1:00	***Lunch Break***	
1:00-2:00	Preliminary Design Options 2 & 3	GHD - [redacted]
2:00-2:30	Spillway hydraulics training wall optimisation	GHD – [redacted]
2:30-5:00	Laboratory Site visit – Located at Geebung – Please advise if you will be attending. Returning to Sunwater office	Trilabs – Geebung laboratory
Day 2 – 28th August 2019		
8:30-9:00	Arrive – visitor sign in	
9:00-10:00	RCC shear strength discussion	GHD - [redacted]
10:00-10:30	Alternative Risk Mitigation measures – Tollgate review workshop	GHD - [redacted]
10:30-11:00	**Morning Tea**	
10:30-12:30	Alternative Risk Mitigation measures - continued	GHD – [redacted]
12:30-1:00	***Lunch Break***	
1:00-2:30	Interim lowering presentation	GHD - [redacted]
2:30-3:30	TRP discussions – TRP member internal discussions	TRP members only
3:30-4:30	Close out discussions – next steps etc.	Open
4:30-4:45	Closing comment	[redacted] Sunwater