PARADISE DAM COMMISSION OF INQUIRY

Commissions of Inquiry Act 1950

STATEMENT OF RICHARD IAN HERWEYNEN

Name of witness:	**Richard Ian Herweynen**
Date of birth:	[redacted]
Current address	[redacted]
Occupation	**Civil Engineer (Principal Consultant)**
Contact details (phone/email):	[redacted]
Statement taken by:	Shana Webster - Lawyer

I, **Richard Ian Herweynen**, state as follows:

1 I am employed by Hydro-Electric Corporation trading as Hydro Tasmania (**Hydro Tasmania**) in the position of Principal Consultant, Civil Engineering, with Entura. Entura is the consulting division of Hydro Tasmania and was known as Hydro Tasmania Consulting during the design construction period of the Burnett River Dam (**Paradise Dam**).

Previous statement

2 I have previously prepared a witness statement dated 12 March 2020 for the purposes of the Paradise Dam Commission of Inquiry. I do not intend to repeat any matters contained in that statement unless necessary for me to do so.

Richard Ian Herweynen

Taken by

Corrs Chambers Westgarth
Level 42, 111 Eagle Street
Brisbane Qld 4000
Tel 07 3228 9377
Fax 07 3228 9444
Ref: MJK:9152525

3 I participated in an interview with Counsel Assisting where I first discussed my views on the technical reports that have been made available to the public. A transcript of my comments is **TRA.510.007.0001**.

Observations on the reports made available to the public

4 I have been provided with a series of technical reports that have been made available for the Paradise Dam Commission of Inquiry. I would like to make some observations regarding some of these reports and provide some context to the events surrounding the issues discussed in the reports. I will respond to further documents as they are received.

5 Any other comments I make are limited to matters which I consider may be errors, short-comings in reasoning, or (in my view) over-conservatism in assumptions and/or conclusions reached in these reports, providing where possible, documents that support this point of view.

Paradise Dam – Flood Event of January to March 2013 - Review of Dam Safety Management Actions, Report for the Office of Water Regulation, Report No. DC 13127, 22 August 2013.

6 It is necessary to appreciate the background to the flood event in 2013. Since December 2010 the spillway flowed continuously until September 2012 apart from 3 days in November 2011 and 2 days in January 2012. That is a period of 21 months with virtually continuous flow (refer to Section 6.2 of the Report). My comment on this is that time period is a very long duration of spillway operation.

7 Even following this 21 months of virtually continuous operation it is noted in Section 5.2 of this report: "There was no significant scour of rock downstream from the dissipator apron in the 2010/2011 flood event and the end sill of the dissipator, though abraded by gravel to expose the steel, remained intact." This indicates damage to the end sill and damage to the slab near the end sill, which may have impacted its structural integrity for the 2013 flood.

8 Based on hydraulic calculations undertaken by Mike Wallis at the time of the design, the end wall clearly impacted the length of the required dissipator apron as demonstrated by comparing the results of a USBR Type I and Type II dissipator **HYT.006.004.5331**. This is supported by some of the statements given in the URS Independent Technical Review by URS, October 2014

Richard Ian Herweynen

Taken by

(**SWA.512.001.0578**)

9 Page **SWA.512.001.0587**, Point 5 – "CFD modelling shows that without the end sill structure in place, the incoming, high energy jet follows the river bed profile and the downstream river bed is subjected to increased hydraulic energy across a broader area of the foundation".

10 In Appendix D of this report (starting from **SWA.512.001.0781**), which was the erodibility assessment by Dr George Annandale (from Golder, and the author of the Annandale Method). In the Executive Summary of this report by Dr Annandale it states "The failure of the end sill of the spillway further exacerbated the situation. Its failure resulted in significantly higher stream power downstream of the dissipator, and therefore significantly greater scour....The failure of the end sill increased the scour extent".

11 Based on the timeline given in Section 10.2 of this Report, the first observation documented during the flood was the failure of the end sill on the left portion of the primary spillway on the 8 February 2013.

12 I would suggest that 21 months of virtually continuous spillway flow, followed by what is estimated to be a 1:200 AEP flood event (peak inflow of 16,500 m^3/s), is a combined event far more extreme than a 1:200 AEP event. It is important to put this combined period from December 2010 to March 2013 into perspective, and it would be good to provide an overall probability of this combined event (i.e. 21 months of almost continued spillway operation, followed by a 1:200 AEP event).

13 A spillway is not necessarily designed to experience no damage during extreme flood events, as long as the dam remains stable. This is recognised in the ANCOLD Guidelines on Selection of Acceptable Flood Capacity for Dams – March 2000 (Section 4.1). This concept is also supported by Dr Steven Pells' Statement, paragraph 10 (**STP.001.0001** at **STP.001.0002**).

14 This overall concept formed the basis of an ANCOLD paper that Collen Stratford (from SMEC at the time) and myself wrote relating to the secondary spillway at Wyaralong Dam, entitled "A unique and holistic approach to the erodibility assessment of dam foundation" (ANCOLD Conference, 2010). This was the

Richard Ian Herweynen

Taken by

method used at Wyaralong and was accepted by the Independent Review Panel. It does not design for zero erosion but sets out to ask the following questions – Is the flow contained?, Does erosion occur?, What is the duration of erosion during an event?, What is the ultimate depth of erosion?, Does the dam remain stable?. So it is important to consider the full progression of the potential failure mechanism.

15 The determination of the downstream erosion protection designed at the toe of the dam requires some engineering judgement based on consideration of information available at the time from both the hydraulics (the energy of the flow) and the erosion resistance of the geology (based on the erodibility index, which is dominated by the identified defects in the rock mass).

16 The combination of this was undertaken by Golders using the Annandale Method, which compares the energy in the flow with the erosion thresholds determined for the rock mass. The only areas of erosion concern highlighted by Golder was downstream of the secondary spillway and the left abutment, which were treated as per their recommendations.

17 There is no doubt the final decision made by the Design Team in relation to the dissipator apron was heavily influenced by the high tailwater level, the physical hydraulic model study undertaken and the observations made, and the assessment of Golders that the rock downstream of the primary spillway was considered to be erosion resistant based on the Annandale Method.

18 It is important to note that using the same raw data two geotechnical engineers could determine two different Erodibility Index as highlighted in paragraph 15 of Dr Steven Pells' Statement (**STP.001.0001** at **STP.001.0004**).

19 It is also important to note that the primary spillway is 315m long and it appears that significant erosion immediately downstream of the apron occurred in two localised locations, indicating that a vast portion of the foundation proved to be erosion resistant for the 2013 flood.

GHD Memo dated 5 September 2019 regarding Shear Strength Test Data for RCC Lift Joints (**GHD.004.0001**)

20 I have a few clarifications to statements made in this report:

Richard Ian Herweynen

Taken by

(a) The Alliance, as per Detail Design Report – Section 5 – Dam, adopted the following shear strength parameters in the stability analysis based on 'Good' lift joints (adopted values) of 325 kPa cohesion and 40.4° friction, however, a sensitivity analysis was undertaken for 'Poor' lift joints with 250 kPa cohesion and 35° friction. This is different from that implied in Section 2.1 of this report.

(b) It is correct that no testing was undertaken on shear strength, which is not unusual for a dam of the height of Paradise Dam where stresses are relatively low. Based on the extensive RCC trial mix program for Paradise Dam, and Ernie Schrader's extensive database on RCC mixes and shear testing, Ernie Schrader provided what he considered to be a best estimate set of shear strength parameters and factors to reduce these best estimates for various ages of RCC and lift joint quality. Even for an excellent lift joint, the best estimate shear strength parameters were reduced by 20% at the recommendation of Dr Schrader to have some degree of conservatism in the shear strength parameters. This was provided to the Alliance via a memo from Dr Ernie Schrader.

(c) It appears Francisco Lopez from the current Technical Review Panel accepted that these design parameters were in fact conservative, as presented in Appendix D of the 1st Technical Review Panel Report (**SUN.009.003.063** at **SUN.009.003.0652**). In this document, he obtained (ICOLD source) data from shear strengths of RCC lift joints in Brazil, USA and Vietnam and plotted them as shear strength against normal load. From the data points associated with low paste RCC with no bedding mix, he provides a line of best fit. This line of best fit shows an "apparent cohesion" of around 1MPa (this is the intercept of the line on the y-axis). With bedding mix it is even higher.

Richard Ian Herweynen

Taken by

(d) Based on this line of best fit, our design cohesion does not seem low. In the graph he also plotted our design parameters, and based on this comparison he stated the following: "*All the adopted values without bedding mix are lower than the lowest tested joint in the database. In principal, the values adopted for no bedding mix appear conservative*". Based on this observation, I cannot understand why the parameters adopted by GHD have not been more fully challenged by the TRP, before action was taken based on these parameters.

21 The key concerns I have with this GHD document are as follows, all of which would lead to lower assessed shear strength parameters for Paradise Dam then the likely actual shear strength across the <u>entire</u> RCC lift joint:

(a) Given that Paradise Dam is a lean RCC mix, the size and method of sampling is critical if lift joint shear strength testing is to be representative of the actual lift joint shear strength in the dam. The drilling method and size of drill core for the vertical drill holes was 83mm, with maximum aggregate size of 51mm, this is too small. Even the larger horizontal core is only 142mm, which is still marginal. This would lead to higher number of debonded lift joints recorded, due to breaking at lift joints due to the drilling process, and lower shear strength when testing, then if larger samples were obtained with a more careful method of obtaining the samples. Even comparing the down hole camera imagining obtained against the core photo (refer to Attachment 4 of this report) shows that the coring process has caused damage to the lift joints. This damage to the lift joint is also shown on the core photos for samples tested in Figures 3.2 & 3.3 (e.g. DD600/12.2 and DD601/23.3).

(b) The number of test samples is in my opinion too small a sample to be confident that any sort of probabilistic assessment is resulting in reasonable shear strength parameters. This is particularly true given that GHD are adopting shear strength parameters that are exceeded by 80% of the tests. If the sample size is too small then this will mean that the shear strength parameters that are exceeded by 80% of the tests is dominated by the lower shear strengths obtained from the small data set (i.e. the poorest samples tested), which could be based on a very

Richard Ian Herweynen

Taken by

localised issue in that core.

(c) The samples obtained are not representative of the shear strength distribution that would be expected on the actual RCC lift surface. As per the Specification – Section 11.0 – RCC Dam Construction, each lift surface was given a Lift Joint Quality Index (LJQI) and if this index was too low, it was improved by the placement of bedding mix on the lift surface prior to the placement of the next RCC lift. This bedding mix was placed in the upstream portion of the lift surface, and therefore the upstream portion would generally have higher shear strengths than the downstream portion of any lift. To date all of the larger sample testing has been undertaken on samples obtained from the downstream portion of the lift joints.

(d) This procedure is supported by the Quality Reports, with the final Quality Control Report for August & September 2005 (Section 7 and 8) indicating a total of 8,526 m^3 of bedding mix, with an average bedding mix to RCC volume ratio of 2.14% (the bedding mix to RCC ratio varied between 5.7% to 1.54% for the entire project). Based on the Specification, bedding mix was placed in 25mm thickness on the lift surface, and RCC layers were 330mm thick), therefore the average width of the lift surface that was covered in bedding mix was approximately 28%, and based on the minimum bedding mix to RCC ratio of 1.54% would equate to approximately 20% of the lift surface (which is a significant portion of the section covered in bedding mix). Therefore it is critical that there is sufficient large samples obtained from the upstream 20% of the dam lift surface to ensure that appropriate representation of lift joint treatment with bedding mix is obtained, a significant proportion of these samples should be taken from the critical primary spillway section (to date there has been absolutely no sampling from the upstream zone of the primary spillway).

(e) A poor or bad lift joint shown in a borehole does not mean that the entire lift joint is poor, which appears to be the conservative approach or logic taken in this GHD report. If this logic was used, then this would demonstrate a lack of understanding of RCC construction methodology

Richard Ian Herweynen

Taken by

and the quality control undertaken. In fact, a poor or bad lift joint shown in a borehole, could in fact be very localised, and not representative of the entire lift surface. Therefore, it is important to get a good representation of variability in shear strength across any potential lift surface by testing poor, good, and excellent lift joint samples (as based on the LJQI measured on each lift surface, as shown in the quality records, all of these were present in the Paradise Dam - recognising that poorer lift joints would have had the upstream portion of the lift joint treated with bedding mix).

(f) GHD's assessment does not take into account the actual design intent and construction specification, which states that if the LJQI is too low that it is improved with the placement of bedding mix in the upstream zone, which is also the treatment of cold joints. This same cold joint could be debonded or have poor bonding in the downstream zone, but would have good bonding in the upstream portion. To ignore the benefits of the bonding and corresponding shear strength of the upstream zone, would be ignoring the fundamental method used to improve the shear strength of lift joints at Paradise Dam. Therefore, in my opinion it would be more reasonable to have two parameters for lift joints, one for the debonded portion of the lift joint (which is not necessarily the residual strength), and a second for the bonded portion of the lift joint where bedding mix was placed, or good bonding was achieved. But to assume that the entire lift surface is debonded is, in my opinion, just too conservative an assumption, given the full understanding of the design, specification and construction methodology. Support for this concept was indicated by Francisco Lopez in his Statement (**LOF.001.0001** at **LOF.001.0014** page 14, paragraph 52) where he states that for the shear strength across the entire lift joint you should take into account both bonded area and unbonded area.

(g) "ANCOLD approach" of 20th percentile shear strength (i.e. 80% of the shear strength measurements were higher than this value) for a typical lift joint at Paradise Dam, if adopted, would require the good lift joints to be tested, as well as poor lift joints, so that the analysis is not biased towards the poor portion of the borehole sample in any lift joint. This is

Richard Ian Herweynen

Taken by

demonstrated by the test data given in Table 4.3 of GHD report, where a much higher shear strength was obtained for the "peak bonded" and "peak unbonded" tests.

(h) No allowance has been made for the "asperity" or waviness of the lift surface. In earlier stability assessments between 2014 and 2016 a value of 2 to 3 degree was added to the shear friction value due to "asperity" of the lift surface, based on recommendations of Graeme Bell, a member of the TRP at that point in time, in report **IGE.020.0001** (at **IGE.020.0052**). This is also supported by Francisco Lopez in his Statement (**LOF.001.0001** at **LOF.001.0015**, page 15, paragraph 53) and also Glen Tarbox mentions this on page 32 of his recent transcript from his questions in front of the Commission (**TRA.500.007.0001**). GHD currently ignore this, they have assumed zero contribution due to "asperity" of the lift surface as they believe it is negligible.

(i) In addition to this, as indicated in the Section 3.1.2 (**GHD.004.0001** at **GHD.004.0004**), residual strength is "the lowest strength achieved at large displacements" and therefore a high degree of displacement would be required before these low shear strength parameters are achieved. The shear strength of a debonded lift joint does not equal residual shear strength, this is due to the fact that you will require some dilation of the lift surface in order to ride over the interlocked aggregate (which will show up as apparent cohesion, or adopting a higher friction angle), and therefore even with a debonded surface you will have a higher peak shear strength then these residual values. Therefore factors of safety quoted using "residual strength" values, are not the current "actual" factor of safety of the dam. In my view, the only way this can be properly demonstrated is with large scale testing.

Additional Comments on Updated Version of GHD's RCC Shear Strength Report **(SUN.009.004.0037)**

20 It is important to again note that the original design adopted strength parameters using both friction and cohesion, and therefore it is incorrect to compare these parameters with "residual strength" parameters, as this is not comparing like with like.

Richard Ian Herweynen

Taken by

21 Referring to Figure 6.2 of this updated GHD report - this shows the scatter of the test results for peak shear strengths, and they have also plotted the BDA design strength on this line. There are almost as many points above the BDA design strength line, as there are points above the BDA design strength line. Therefore the BDA Design Strength line is similar to the Median peak shear strengths tested, even given the extensive criticism various RCC Experts have made on the current testing. Which is evidence that the shear strength parameters adopted by the Alliance at the time of the design are not outrageous.

22 Referring to Table 6.3 of this updated GHD report - is an important Table when looking at what the GHD testing tells us about "Actual" shear strength of lift joints. Let us just forget for a moment the ANCOLD Factor of Safety criteria for using "residual strengths" and the need for 80% of tests to exceed the adopted parameters – which, in my opinion, is an absolute lower bound assessment of the FoS, it is not a reflection of what we think the "Actual" FoS is at this time.

23 In Table 6.3 GHD also provide the Peak unbonded, friction only value (friction only, with no cohesion) – which is probably the closest estimate (based on current testing data) of the actual shear strength.

24 This indicates that 80% of the test results exceeded a peak unbonded, friction only shear strength value of around 45 degrees. But it should be remembered that this still had 80% of the tests being equal or greater than this value – so it is not the median peak strength (where there is equal test data points above as below). The median value for the peak unbonded, friction only shear strength would be even higher than this. This is an important point, and taking this into account would lead to a much higher "Actual" FoS, which, in my opinion, is what should be used in the Comprehensive Risk Assessment to demonstrate the current risk position of the dam.

GHD Memo dated 5 September 2019 – Dam Stability Analysis (**DNR.001.2344**)

25 My comments on this report are based on this summary memo of the stability analysis, as I have not been given the actual stability calculations. They are as follows:

(a) In Section 2.3 it indicates that a shear strength of zero cohesion and 39.3° friction angle was adopted, which was based on the statistical

Richard Ian Herweynen

Taken by

approach for the residual strength of a debonded surface. As already discussed in my comments on the previous GHD report in this statement above there are many reasons why this, in my opinion, is overly conservative and not representative of the shear strength across an entire lift surface. In Section 6 of the GHD report it indicates that "*the key reason for the stability of the dam as currently assessed, is the significantly lower shear strength of the lift joints then assumed in the design and the 2016 dam safety review*".

(b) In Section 3.2 of this report it states that "BDA (2005) lists the density of the RCC based on the construction records and this lists a density as low as 2,484 kg/m^3". However, a density of 2,400 kg/m^3 was adopted in the analysis. My view is that where actual records data is available it should be used in the analysis. Using 2484 kg/m^3 is a 3.5% increase in the weight, which would have a direct 3.5% improvement in the stability results. This appears to have been highlighted in TRP Report No. 1 by Francisco Lopez as presented in his Statement (LOF.001, page 17, paragraph 61).

(c) In Section 3.4 of the report they discuss uplift. It is important to note that in the foundation and at the dam / foundation interface, the uplift assumption in the design was 100% headwater pressure at the upstream face to full tailwater at the downstream face. In the dam body the uplift assumption was 50% headwater pressure at the upstream face to tailwater pressure at the downstream face. Based on the Initial Inspection Report of Burnett Dam, November 2005, which I authored, following initial filling to EL 51.5m, I made the following statement: "*A review of the piezometer data indicates that the current uplift pressures are all less than the design assumptions.*" I held, and still hold the view, that the 50% assumption (based on the plots given in this GHD report) is reasonable for the higher primary spillway blocks. There is no reason why the 50% reduction would not also be relevant for flood loading as the same membrane, drainage and upstream bedding mix system is in place and would not be impacted by flooding. Therefore the same uplift assumption as per the original design appears to be reasonable based on available data, and any scenario looking at increased uplift should be

Richard Ian Herweynen

Taken by

treated as a conservative sensitivity analysis and not a reported FoS against ANCOLD guidelines.

(d) In Section 3.4 in relation to uplift, GHD have also indicated the following - "*The analysis has included the effect the downstream apron/stilling basin has on increasing the uplift. Where there is a downstream apron, the uplift reduces to tailwater at the downstream end of the apron rather than the downstream toe of the dam. The uplift pressure is only applied under the dam and the apron is not considered to form part of the structure for the stability assessment*". First, there was extensive drainage holes installed in the apron/stilling basin to relieve any potential uplift pressure under the apron and to ensure that it was equal to the tailwater pressure measured in the apron from the physical hydraulic model. Secondly, if excessive uplift pressures occurred under the apron then cracking is likely to occur (as this is a relatively thin structural element) and this in turn would relieve any pressures. As a result, the original design assumption was that tailwater pressure was at the toe of the dam section not at the downstream end of the apron as assumed by GHD (this design uplift assumption was verified at the time of first filling using the piezometric data, as discussed above). This is exactly the same as what is shown in Figure 3.1 of the ANCOLD 2013 Guideline, and in Section 3.4 of this Guideline it states "Drainage sytems under or through downstream aprons will reduce seepage path length and may be taken into account". The original design assumption is also the same assumption that were accepted by the dam safety reviewers undertaking the review in 2016 (this is highlighted in Section 5.4 of the GHD report).

(e) Downstream tailwater load given in Section 3.5. The original design used the actual pressures measured in the physical hydraulic model study and adopted the triangular shape of the tailwater profile shown in Figures 3.6 & 3.7 in the GHD report. We then undertook various sensitivity analysis based on HECRAS modelling, SunWater tailwater modelling, tailwater undertaken by URS in their Stage 2 submission. It is unclear from Section 3.5 whether GHD adopted the actual downstream pressure profile (i.e. triangular shape of increase tailwater as you move downstream of the downstream face), which is shown by both the

Richard Ian Herweynen

Taken by

physical model and GHD's CFD modelling, or whether they just adopted 83% reduction in the tailwater level. Again, if a straight 83% reduction in tailwater level was adopted then this again is a conservative assumption, as this is both not shown by the physical hydraulic model and their own CFD modelling. Based on the original design stability results it was clear that lower tailwater levels generated a lower factor of safety, therefore this is a critical assumption.

26 It is the sum of these conservative assumptions that, in my opinion, creates a compounding issue. The combination of all of these individual conservative assumptions, including using "residual shear" strengths parameters, means that the conclusion given in Section 5.1 that the actual factor of safety (FoS) was only 1.11 at the time of the January 2013 flood is, in my opinion not an accurate statement. The actual factor of safety quoted and presented for the 2013 flood should be based on the most likely parameters for each of these items discussed above.

27 These same factors of safety using residual shear strength and these conservative assumptions were used in the current comprehensive risk assessment to map FoS against probability of dam failure. In my opinion, if such a mapping is used, then it should be based on actual peak shear strength parameters and most likely assumptions, rather than overly conservative assumptions and residual shear strengths. By not doing so, has resulted in my opinion, to an overly pessimistic view point on failure probabilities due to sliding in the risk assessment, which is the basis of key decisions by Sunwater (further discussion on this is made later).

Technical Review Panel Report No. 2, Paradise Dam Improvement Project, 23 September 2019

28 I recognised that the TRP can only arrive at conclusions based on the knowledge and information that is provided to them. However, in reading this document there are a number of statements that I do not consider to be factually correct and/or I do not agree with. They are as follows:

(a) I have read Section 2.2 (page 3 to page 5) of the TRP report. My comment in response to this section is that foundation excavation removed all of the basalt material (including any paleo gravels) from

Richard Ian Herweynen

Taken by

under the foundation of the dam, excavating this material to the Goodnight beds. The only area of the basalt that was left in place was referred to as the "Pimple" and extensive investigations were undertaken to demonstrate that there was no paleo gravels under the pimple (which is what was initial thought as the original geological investigation which was presented in a 3D geological model by Golders indicated that the Paleo Gravels pinched out as they got closer to the river channel). This was reported in the Golder's Geotechnical Design Report, the corresponding 3-dimensional geological model, and the additional investigation undertaken on the interface between the basalt "pimple" and the Goodnight bed rock (refer to the ITP associated with this, **DNR.001.2038**). This was reviewed by Patrick McGregor, as an independent reviewer (**DNR.010.0929** at **DNR.010.0945**). His comments were incorporated into the final design as indicated in the Alliance's Final Design Report.

(b) In Section 2.4, it states that, "in the secondary spillway, three of the five boreholes of the secondary spillway showed that the base of the RCC is sitting on a poor quality moderately weather rock". This is not surprising as the acceptable foundation level and material properties was significantly lower for the secondary spillway as highlighted in the Specification – Section 3.0 – Surface Excavation and Earthworks. It was recognised that it was pointless excavating foundation rock and replacing it with a lean RCC mix section, if the strengths were not required at these lower height sections of the dam. Significant interaction with Golder's geotechnical team and the Design team occurred to develop the various flow charts given at the back of this Specification. These flow charts clearly indicated the design objectives, the construction process, the geotechnical inspections, and the HOLD POINTS and sign-offs required by both the on-site geological mapping team and the on-site dam engineer. This procedure was adopted on site during construction. The foundation excavation and preparation was split into a number of Inspection & Test Plans (ITPs), with quality records showing these sign-offs. Given that the acceptance of the foundation is a critical issue, this process was independently reviewed and foundation preparation was

Richard Ian Herweynen

Taken by

inspected by Brian Shannon, as an independent reviewer (**DNR.010.0918**). His comments were incorporated into the final design as indicated in the Alliance's Final Design Report.

(c) As stated in Section 3.1, "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". This was recognised by the Specification with clear requirements for time from batching to compaction, the requirements for lift joint clean up, treatment of any localised segregation, the requirements of bedding mix if the Lift Joint Quality Index (LJQI) was inadequate, and the treatment of cold joints. Based on my observations and knowledge, this specification was generally followed.

(d) I have, in my witness statement of 12 March 2020, set out the responsibilities of Jose Lopez and Robert Montalvo, who had key duties around the placement of RCC.

(e) All cold lift joints, to my knowledge, were treated as per the requirements of the Specification with the clean-up of the lift joint and the placement of bedding mix over the upstream portion of the dam (as per Specification, Section 11, Clause S11.10.2). This is reflected in the RCC Quality Control Reports prepared by the international expatriate, RCC quality Control Engineers. Also to my knowledge, the Cold Joint Treatment and Criteria as indicated in Ernie Schrader's memo during his July-August 2004 Site Visit was not taken up and the Specification was never relaxed. Any changes to the design, including the specification, required sign-off by the Lead Dam Engineer. What was indicated in this previous discussions was that there was more cold joints then originally planned for, due to both some teething issues with the placement method in the early stage of RCC placement, warmer weather and greater number of rainfall events due to delays in the construction schedule resulting in the main RCC placement occurring in the hotter, wetter months. But in all cases cold joints were treated as per the specification.

(f) Section 3.2 also implies that the original designers were involved in this

Richard Ian Herweynen

Taken by

assessment. This is not the case. The original designers have not been involved in any evaluation and assessment since around 2010, and have not been engaged in anyway in relation to this work undertaken since 2016. The only interaction that occurred is following a request from SunWater in June 2019 to have a teleconference call to discuss a few items. This occurred on the 1 August 2019. Based on this teleconference call it was clear that SunWater did not have all of the documentation, and as such I helped source some of this information from key Alliance members – which included a Quality Assurance Report from August and September 2005, which outlined, amongst other things, the amount of bedding mix placed on the dam to-date (which I discuss further above). The fact that SunWater did not appear to have all of the design and construction documentation was concerning and also surprising, given that all of the Alliance documentation and the Alliance server with electronic copies of all design and construction files was released to SunWater at the end of construction. It appears that far more of this documentation has now been sourced through this Commission of Inquiry process.

Tatro Hinds Report, Paradise Dam - Shear Strength Evaluation Comments, 25 November 2019 (**TAT.001.0001**)

29 I would like to make the following comments about this report:

(a) I agree with all of these issues/criticisms given in this report on the sampling, sample size and testing method. In my opinion these shortcomings reflect that those undertaking this investigation and testing are not familiar with lean RCC mix. This report identities, and I agree with, a view that an insufficient number of tests have been conducted given the high consequence associated with a conclusion that lift joints are unbonded. I support the recommendation that many more shear tests be conducted in order to more accurately determine the strength condition of the RCC. The current number is too few for such an assessment. I believe it is important that the Tatro Hinds report is considered in detail.

(b) This concerns raised by Steven Tatro in this report are again reflected in

Richard Ian Herweynen

Taken by

his Statement provided to the Commission (**TAT.002.0001**). I interacted with Steven Tatro on the Wyaralong Dam project, where he provided RCC expertise to the Independent Technical Review Panel. I know he has significant experience in lean RCC and testing of RCC, and believe that his comments should be appropriately considered on this subject.

(c) It is my opinion, that if the issues identified in the Tatro Hinds report were rectified in a repeated investigation and test program, that higher and more representative shear strengths of the actual lift surface would most likely be obtained.

Paradise Dam Preparedness Review, Report 1: 2019 – 2020, 19 December 2019 **(IGE.084.0001)**

30 I would like to make the following comments about this report:

(a) I observed that, on page 26 of this report, that in 2012 Paradise Dam was considered one of Queensland's safest dams. This was based on a Comprehensive Risk Assessment conducted in 2009 and then updated in 2012. Therefore, up until the 2013 flood, I note that SunWater considered Paradise Dam to be a very safe dam. It is expected that this Comprehensive Risk Assessment undertaken would have used all of the available design and construction records for the dam, as this is the standard approach with this sort of study.

(b) It also interesting to note that even following the flood damage in 2013, the risk position of the dam based on the 2016 assessment was still acceptable (refer to Figure 1 in the report), and that the stability of the dam was safe for the Acceptable Flood Capacity required of the 1:15,000 AEP flood. It was not until 2018/2019 that the probability of failure significantly increased in the assessments undertaken. This was all based on investigations and testing around the shear strength of the RCC lift joints, which I have made comments about that in my statement above.

(c) It is clear from the table given on page 27 of this report that the risk assessment has indicated that the most likely cause of failure of

Richard Ian Herweynen

Taken by

Paradise Dam is sliding along the primary spillway monoliths (51%). This is driven by the low shear strength parameters that have been adopted in the stability analysis, in my opinion too low, and some of the conservative assumptions highlighted above.

(d) The second most likely cause of failure identified is undermining of the primary spillway due to future scouring. It is unclear what the difference is in the table on page 27 between:

i. Undermining of Primary Spillway monoliths due to overflow scour, below apron, and/or

ii. Undermining of Primary Spillway monoliths due to scour at the toe, through the apron,

as combined probability of these two is 39%.

(e) Finally, I think it is important to highlight two items related to comments made in the section of the report noted as 'Appendix F: Timeline':

i. Under October 2003 it states "*A trial embankment was built and tested during construction of the dam; this tested acceptable. SunWater have subsequently advised normal practice would be to do this prior to construction so test results can influence design*".

My comment on that is although the trial embankment forms the final monolith block on the far right of the dam (where stresses are very low), it was constructed prior to any RCC placement in the main dam. This trial embankment was used to test the final RCC mix, test various extreme scenarios that may occur during construction and the impact on these, and to test and refine the RCC deliver and placement methodology. I have outlined this process in detail in my witness statement dated 12 March 2020. This trial embankment followed two stages of trial mix program, the first program was to trial various mixes to determine the appropriate mix design for Paradise Dam, while the second program was to test the final mix design and undertake some sensitivities on this. All of

Richard Ian Herweynen

Taken by

this is well documented in the design and construction records.

(f) Under January 2006 it states that "*Sunwater advised they were unaware of the 2006 core testing until July 2019*". It is difficult to understand this statement given that:

i. All design and construction documentation was given to SunWater at the conclusion of the Project;

ii. as of 16 December 2005, SunWater became responsible for the ongoing management and operation of the dam. Following this, in order to enter the site for any reason required personnel to sign onto a SunWater Job Safety Assessment (JSA). Therefore drilling would not have occurred without their knowledge. I personally experienced the requirements for access, including the signing of a SunWater JSA on several occasions in certain site visits; and

iii. there is email correspondence from SunWater requesting the results of the core samples.

ANCOLD Guidelines and their application to the assessment of 'stability issues' with Paradise Dam

31 I have reviewed the transcripts of evidence given before the Commission on 3 and 4 March 2020 by Mr James Willey and Mr Peter Foster, particularly in relation to the approach taken to GHD's assessment of Paradise Dam's current stability by the application of the ANCOLD Guidelines on Design Criteria for Concrete Gravity Dams (September 2013) (**ANCOLD 2013 Design Criteria Guidelines**) to the dam.

32 I believe it is appropriate and may assist the Commission with its task of investigating the current 'stability issues' with Paradise Dam to provide some observations about the application of the ANCOLD 2013 Design Criteria Guidelines to Paradise Dam.

33 I was a member of the working group which authored the ANCOLD 2013 Design Criteria Guidelines.

Richard Ian Herweynen

Taken by

34 In 2003, at the commencement of the design of Paradise Dam, the then current ANCOLD Guideline for Concrete Gravity Dam Criteria was the 1991 version. It is important to recognise that there are some significant differences between the 2013 ANCOLD Guideline and the 1991 ANCOLD Guideline.

35 In addition to the 1991 ANCOLD Guideline, the Alliance nominated to also use the USACE Gravity Dam Design – Engineering Manual (EM 1110-2-2200), 30 June 1995 (**USACE Design Guidelines**).

36 Adopting the 1991 ANCOLD Guidelines and the USACE Design Guidelines as the basis of design of Paradise Dam was a decision that was accepted by the client (Burnett Water) at the time. These guidelines permitted the use of both friction and cohesion (even if it was 'apparent' cohesion) to achieve a satisfactory sliding stability of a dam, and neither of them contained any requirement to meet a friction only or residual shear strength design case (it is also important to note that 'friction only' criteria used in some current guidelines internationally is not necessarily equivalent to a 'residual strength' criteria given in the 2013 ANCOLD Guideline).

37 From the evidence I have seen put before the Commission by GHD and others, current enquiries into the stability of Paradise Dam has been focused on residual shear strength to determine acceptability or otherwise of residual strength factors of safety outlined in the ANCOLD 2013 Concrete Gravity Dam Guidelines.

38 The need to achieve acceptable residual shear strength sliding stability under the ANCOLD 2013 Design Criteria Guidelines appears to be being used as justification for not properly testing for the cohesion we (who designed the dam) know was required in our design and the placement of bedding mix at the upstream face region of the dam was an integral part of ensuring adequate shear strength on any lift joint with a lower LJQI.

39 There are suggestions that the cohesion of the bedding mix cannot be adopted anyway, due to the fact that there will be a compatibility issue with the remainder of the lift joint. It is my personal opinion that this argument does not hold true.

40 Firstly, the strain we are talking about to mobilise friction across the lift surface is

Richard Ian Herweynen

Taken by

very small and therefore I cannot believe that the bond of the bedding mix would be broken with these very small strains. I recall that all of the products used in the dam were designed, based on laboratory testing, to be compatible from a stress-strain (i.e. stiffness) perspective, whether this was foundation concrete or bedding mix.

41 Secondly, assuming the compatibility argument were true (even though I do not believe it is), that the bedding mix would create a stiffer product then the rest of the dam and would attract the horizontal load initially, then equally well it would attract the vertical load. If it attracted the vertical load as well then that portion of the dam would also then provide most of the friction resistance (as the equation is sum of vertical load multiplied by the shear friction factor). Lastly, I am aware of this same process of adopting an aerial average of the shear strength parameters being used on foundations where there is two different rocks across the foundation footprint, which have different shear strength parameters.

42 Residual strength testing ignores any actual or apparent cohesion of the lift joints currently in-situ at Paradise Dam and the design intent of the dam.

43 Aside from the fact that the residual strength friction angles being reported by GHD are unrealistic and suggestive of testing error arising from improper testing method for lean RCC shear strength testing, and not representative of the entire lift joint (not some isolated core sample of one point of a lift joint), do not strictly meet the residual sliding strength factors of safety in the 2013 ANCOLD Guidelines acceptance criteria, this does not necessarily mean that Paradise Dam is currently unstable and therefore unsafe.

44 What it would mean is that the dam does not currently meet revised design criteria guidelines released by ANCOLD more than 15 years after the dam was designed. It would then become an issue for the asset owner to assess, and determine if any response is required, as part of its regular dam safety reviews and routine dam upgrade works programme to upgrade to the current standard or something less, if the risk position is appropriate.

45 For this purpose, ANCOLD has published Guidelines on Risk Assessment (2003) to guide dam owners and regulators on how to assess the safety of dams and upgrade works requirement.

Richard Ian Herweynen

Taken by

46 The focus at this point though should be in properly assessing the stability of the dam not just against the 'residual strength' ANCOLD 2013 Design criteria, but also determining the "Actual" factor of safety taking into account the peak values of shear strength. It would also be important to assess the safety of the design against the design criteria the dam was designed to, in order to properly inform an assessment on what is actually contributing to the GHD conclusions on the stability of the dam, is it purely the low shear strength parameters adopted or some of the other assumptions they have adopted, or increases in flood and seismic loading that has occurred since the original design and construction of the dam.

Risk Based Decision Making (a Key Point in Relation to the Comprehensive Risk Assessment Reports)

47 In dam engineering within Australia we often talk about both "Standards Based" assessments and "Risk Based" assessments. Not all existing dams within Australia meet the current ANCOLD Guidelines from a "Standards Based" assessment – that is a fact (this is even highlighted in Appendix A of the current 2013 ANCOLD Guideline). If an existing dam does not meet the Standards based assessment, but meets the ANCOLD tolerable risk requirements, then an assessment is made to determine if any upgrades can be justified, using the ALARP principal (referred to in Jonathon Reid's Statement, JTR.001.001.0001).

48 In order to make this Risk based decision, it is important to determine whether the existing dam, in its existing state meets the ANCOLD tolerable risk guidelines.

49 This requires an assessment to be made on the Likelihood (or Probability) of Failure for various identified failure modes, and the consequence should failure occur. This is not an easy and/or exact science, and requires considerable amount of engineering judgement, which was indicated in the Statement by Jonathon Reid (**JTR.001.001.0001**).

50 One of the potential failure modes for a concrete gravity dam is failure due to sliding, whether this be sliding along a lift joint, at the foundation interface or within the foundation. Therefore in order to determine the likelihood of failure due to sliding, so that it can be used in the risk assessment, the factor of safety

Richard Ian Herweynen

Taken by

needs to be converted to a likelihood (or probability) of failure.

51 There is no database of concrete dam failures (as concrete dams are considered to be one of the safest dams) and therefore no correlation has been developed between factor of safety and probability of failure, which was highlighted in Jonathon Reid's Statement (JTR.001.001.0001). As a result, they have used a conversion from FoS to probability of failure in the Comprehensive Risk Assessment Report using a correlation that has been developed for earth embankment and slope failures. This is the best they had available, but is questionable.

52 However, for me the key issue is not this conversion but the Factor of Safety (FoS) used in this conversion. They have used the "Residual Strength" factors of safety and not "Peak Strength" factors of safety in the conversion from FoS to probability of failure in the Comprehensive Risk Assessment. Using "Residual Strength" in this conversion to probability of failure is in my opinion not correct. I agree that the "Residual Shear Strength" criteria exists for the 2013 ANCOLD Guidelines, but this is not the "Actual" FoS along a lift joint at Paradise Dam. By definition (as per the ANCOLD Guideline), the "Residual Shear Strength" is reached following large displacements, which has definitely not occurred at Paradise Dam. The "Actual" FoS should be based on "Peak Strength" parameters for the entire lift surface, this needs to include both the peak strength of the unbonded portion of the lift joint, and the impact of bedding mix on the upstream portion of the lift joint. Determining the "Actual" FoS along a lift joint, and converting this to a probability of failure will result, in my opinion, to a different picture for the risk position of Paradise Dam against ANCOLD Risk Guidelines, and the contribution to risk due to sliding along lift joints. Therefore there is a need for determining the "Actual" FoS, given that the basis for the need (or otherwise) of upgrades at Paradise Dam is based on the ANCOLD Risk Guidelines.

Richard Ian Herweynen

Taken by

OATHS ACT 1867 (DECLARATION)

I, Richard Ian Herweynen, do solemnly and sincerely declare that:

1 This written statement by me dated 12 March 2020 is true to the best of my knowledge and belief; and

2 I make this statement knowing that if it were admitted as evidence, I may be liable to prosecution for stating in it anything I know to be false.

And I make this solemn declaration conscientiously believing the same to be true and by virtue of the provisions of the *Oaths Act 1867*.

_____ Signature

Taken and declared before me at Brisbane this 12 day of March 2020.

Taken by _____ Shana Webster

~~Justice of the Peace~~ / Commissioner for Declarations / Lawyer