PARADISE DAM COMMISSION OF INQUIRY

Commissions of Inquiry Act 1950
Section 5(1)

STATEMENT OF JAMES CLAYTON WILLEY

Name of Witness:	**James Clayton Willey**
Date of birth:	[redacted]
Current address:	**C/- GHD Pty Ltd** **Level 9, 145 Ann Street** **BRISBANE QLD 4000**
Occupation:	**Engineer**
Contact details (phone/email):	[redacted] [redacted]
Statement taken by:	**Jonathan Horton QC and Jane Menzies**

I, **James Clayton Willey**, Engineer, make oath and state as follows:

My background

1. I am a Senior Technical Director – Dams and have been employed by GHD Pty Ltd (**GHD**) since 5th March 2001.

2. I hold a Bachelor of Engineering from the University of Southern Queensland.

3. A copy of my curriculum vitae is document [[**WJY.001.002.0001**]].

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4. My engineering experience includes 24 years in embankment and concrete dam design, spillway design, investigation and design of dam and spillway upgrades and remedial works, and dam safety and surveillance reviews.

5. I am a member of GHD's project team working with Sunwater on various projects related to the Paradise Dam (**Dam**). My work has included assessment of the existing dam structure. I am aware of structural and stability issues with the Dam. I was the key author of two memoranda on this subject, dated 5 September 2019 being:

   (a) a memorandum which presented the findings of a review of the existing shear strength test data for the Dam's roller-compacted concrete (**RCC**) lift joints, provided recommendations for design shear strengths to be adopted for the stability analyses of the Dam, and presented recommendations on further work to be done to achieve greater confidence in the results **(Shear Strength Memorandum) [[DNR.001.2363]]**; and

   (b) a memorandum which provided an update on the review of the stability of the Dam monoliths with respect to failure through the RCC on lift joints under flood loading for the existing Dam arrangement and for a potential interim lowering of the primary spillway crest **(Stability Memorandum) [[GHD.005.0001]]**,

   together, the Memoranda.

6. As the key author of the Memoranda, I was responsible for documenting GHD's work. The content of the Memoranda was discussed with other GHD employees. My background is broadly in dams. I do not have extensive specific experience in RCC. On this project I have engaged with Mr Jon Williams from our team who has been involved in many RCC projects in both design and construction roles. Specifically, I

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recall consulting with Mr Williams in relation to the laboratory testing of the RCC, including development of the testing methodology, and the test results. Structural and stability assessments of dams is one of the areas I am experienced in, but I have relied on input from others, such as Mr Williams, in relation to the RCC.

7. The interpretation and the findings presented in the Memoranda represent the outcome of the assessment undertaken by GHD for Sunwater. As the key author, I confirm the Memoranda represent my professional view at the time of writing them.

My first involvement with the Dam – 2014

8. The first project in which I was involved in relation to the Dam was in 2014. That project was to undertake a failure impact assessment, and my input into that project was limited to advising the GHD team on dam breach characteristics, and providing input into the downstream consequence assessment.

9. The focus of the failure impact assessment and related work in 2014 was to assess the consequences should the Dam fail (hence failure impact). GHD was not instructed to assess the condition of the Dam or the likelihood of failure as part of the failure impact assessment.

10. I have been involved in three other projects since then related to the Dam:

    (a) design of minor strengthening works from 2015 to 2017;

    (b) review of improvement options for the Dam as part of the development of the preliminary business case from 2017 to 2018; and

    (c) preliminary design of improvement works from late 2018, noting this project is ongoing.

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11. These projects are discussed further below.

Minor strengthening works – 2015 to 2017

12. In 2015 to 2017, GHD was engaged to undertake the design of minor strengthening works on the Dam (**minor strengthening works**). The need for these works was identified by Sunwater and arose from Sunwater's Comprehensive Risk Assessment (**CRA**) of the Dam. The most recent version of the Sunwater CRA is dated June 2016 **[]IGE.068.0001]]**. I was the lead designer for the minor strengthening works and was also involved in providing construction phase support to Sunwater.

13. The minor strengthening works involved design of structural modifications to address one of the potential failure modes that Sunwater had identified in the CRA arising from the 2013 flood.

14. At the downstream toe of the Dam at Monoliths D and K (the monoliths at the left and right sides, respectively, of the primary spillway), the foundation for the Dam monoliths is higher than the level of the spillway apron at the downstream toe. The foundation above the apron level was protected by anchored reinforced concrete steps in these locations. The Sunwater CRA identified that, if these anchored reinforced concrete steps were to fail during a flood overtopping event, the foundation would be exposed and potentially eroded, thereby undermining the Dam monoliths and leading to failure of the monoliths and the Dam.

15. The focus of GHD's involvement was the design of anchored reinforced concrete blocks (monolith strengthening blocks) to strengthen this area and reduce the likelihood of failure in this area. For clarity, these minor strengthening works were designed to address the issue identified at paragraph 14 above, and not the main structural instability issues which are the subject of the Memoranda.

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16. As part of the minor strengthening works, GHD also undertook the design of other works, including an access crossing downstream of the Dam and abutment protection works, but the monolith strengthening blocks were the main focus of the project. Design reports were produced for this work, including a 'Detailed Design Report (Final)' dated September 2017, which was an updated design report following construction. That report includes the as built information for the monolith strengthening blocks and the other works.

Preliminary business case – 2017 to 2018

17. In the Sunwater CRA, a range of improvement works options were considered to address the identified failure modes. GHD was engaged by Sunwater to undertake a review of the options selected by Sunwater and to develop those options as part of the preliminary business case. I was the lead dams engineer on that project.

18. As part of this process, additional options were identified by GHD and Sunwater, and then subsequently developed by GHD. In the development of these options, it was identified that anchoring of the Dam may be needed to address potential stability concerns for the Dam which were related to the shear strength of the RCC. The review and development of the options is presented in a GHD report entitled 'Paradise Dam Facility Strategy & Options Analysis – Preliminary Business Case – Supporting Technical and Environmental Review' dated March 2018 [[IGE.033.0001]].

Preliminary design of improvement works options - late 2018 to present

19. In late 2018, GHD was engaged to undertake the preliminary design of two improvement works options (**preliminary design project**). These options had been selected by Sunwater out of the preliminary business case process. The first of these options included strengthening of the Dam with the full supply level retained at its

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current level. The second option included a permanent lowering of the full supply level by 10 m as well as other strengthening works. Both options include post-tensioned anchoring of the Dam, and an extended stilling basin for the primary spillway. I have been, and remain, GHD's design manager and project manager for the preliminary design project.

20. The remainder of this statement predominantly presents information related to tasks that were undertaken as part of the preliminary design project.

Review and update Sunwater's risk assessment for the Dam

21. As part of the preliminary design project, one of the tasks GHD was engaged to do was to review and update Sunwater's risk assessment for the existing Dam (**review risk assessment task**).

22. There is a draft report dated January 2019 setting out the basis, assumptions and finding of GHD's review and update of Sunwater's risk assessment for the existing Dam. It has not yet been finalised because further studies and investigations need to be completed before it can be formally updated.

Draft preliminary design report- July 2019

23. The other main deliverable issued by GHD to Sunwater as part of the preliminary design project has been the draft preliminary design report dated July 2019 [[IGE.036.0001]] for the two improvement works options nominated by Sunwater (**draft preliminary design report**).

24. The draft preliminary design report presents background information, design basis, key assumptions and the resulting preliminary design for the two improvement works options.

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25. Given the progress on the project since the draft preliminary design report was prepared and issued, there are sections which are now superseded. These include:

    - Section 5 – Geological and Geotechnical Model – Additional investigations were undertaken in 2019 and additional data review and interpretation has been undertaken. This task is ongoing, and it is expected that the draft report on the geological and geotechnical model will be issued in early April 2020.

    - Section 6 – Review of Existing Dam Stability – This assessment is superseded by the information presented in the Memoranda.

    - Section 7 – Update of Comprehensive Risk Assessment – As the design process has progressed and additional information has been obtained, the risk assessment has also been updated. The report on this task requires updating as discussed in paragraph 22.

    - Sections 8 to 10 – Scope Confirmation for Improvement Works Option, Preliminary Design – Option 2, Preliminary Design – Option 3 – Based on changes to Sections 5 to 7, there will also be changes required to these sections to reflect the most recent understanding.

26. The timeframe for finalisation of the preliminary design report is not clear. This is currently under discussion with Sunwater taking into consideration the tasks that still need to be completed. The report will most likely not be finalised until April 2020, pending confirmation of the scope.

The Memoranda

27. The Memoranda were prepared as part of the preliminary design project and issued to Sunwater. GHD does not usually present its findings by way of memoranda. Our usual

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practice is to issue formal reports at the conclusion of a piece of work. However, at the time of preparing the Memoranda, the relevant report was not due to be completed for approximately 5-6 months. Therefore, the Memoranda were issued to Sunwater as interim progress updates. Sunwater did not specifically request that the Memoranda be prepared.

28. The initial drafts of the Shear Strength Memorandum and Stability Memorandum were issued to Sunwater in November 2018 and December 2018, respectively. A final version of the Stability Memorandum was issued to Sunwater in December 2018. The current versions of the memoranda were issued to Sunwater in September 2019, and were subsequently made public by Sunwater.

29. As noted in the Shear Strength Memorandum, testing was underway on additional samples at the time of finalising the memorandum in September 2019. The results of those tests were not available for inclusion in the memorandum. The revision of the Shear Strength Memorandum to include those additional results is in progress at this time.

30. After the initial issue of the Memoranda in late 2018 and the update of the risk assessment as presented in the draft report dated January 2019 (refer paragraph 22), Sunwater made the decision to implement risk reduction measures as soon as possible. Sunwater engaged GHD to undertake the detailed design of this project which has been given the title of "Essential Works" by Sunwater.

Essential works project

31. The essential works project includes lowering the primary spillway crest of the Dam with the aim of reducing the dam safety risk.

32. The deliverables issued by GHD for the essential works project predominantly include tender drawings and technical specifications related to the tender process for the project. There is an updated design report for the essential works that is due for completion by GHD in March 2020. These documents are focused specifically on the design of the essential works rather than reviewing the condition of the existing structure.

Approach to the assessment of dam stability

33. This section presents the approach to the assessment presented in the Memoranda.

34. In assessing relevant failure modes and identifying appropriate strength parameters in relation to the dam stability, the key consideration is the assessment of the likelihood of unbonded lift joints being present within the Dam, and their extent. There is no argument that some of the lift joints are bonded. However, if unbonded lift joints are present, this dictates a certain range of shear strength values applicable to the lift joints. For conventional concrete, the ANCOLD *Guidelines on Design Criteria for Concrete Gravity Dams* (ANCOLD Guidelines) dated September 2013 **[[ACD.001.0001]]** provide guidance on the range of strengths expected for bonded intact concrete (peak shear strength), unbonded peak strength (sliding shear strength), and unbonded residual conditions (residual shear strength). More limited guidance is provided for the shear strength of RCC.

35. I have seen no evidence to indicate that the Burnett Dam Alliance (the Alliance) undertook testing to confirm the proposed RCC mix and construction approach would achieve the original design assumptions in relation to lift joint shear strength. I have also not seen evidence of testing undertaken during or post-construction to confirm the design shear strength assumptions were achieved. If the Alliance design strength is used in the stability analyses, the stability of the dam is considered acceptable when

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compared against the ANCOLD criteria for the "well-defined peak strength" condition. It is noted that the design and construction of the Dam predates the issue of the ANCOLD Guidelines listed above. The focus of this project has been on assessing the dam against the current criteria as presented in the ANCOLD Guidelines.

36. In preparing the Memoranda, GHD reviewed information relating to an inclined cored investigation hole which was drilled by the Alliance in January-February 2006. The hole was drilled from the crest of the secondary spillway in Monolith L with an inclination of 60° to the horizontal towards Monolith K in the Primary Spillway. It is understood that this investigation was supervised, and the logs prepared, by representatives of the Alliance. The information I reviewed included:

- A report entitled "Preliminary Comments on RCC Coring at Paradise Dam" **[[ALC.001.001.1683]]**

- Spreadsheet form core logs as follows:

[[ALC.002.001.0717]]	CORING LOGBOOK – 1st Run.xls
[[ALC.002.001.0718]]	CORING LOGBOOK – 2nd Run.xls
ALC.002.001.0719	CORING LOGBOOK – 3rd Run.xls
ALC.002.001.0720	CORING LOGBOOK – 4th Run.xls
ALC.002.001.0721	CORING LOGBOOK – 5th Run.xls
ALC.002.001.0722	CORING LOGBOOK – 6th Run.xls
ALC.002.001.0723	CORING LOGBOOK – 7th Run.xls
ALC.002.001.0724	CORING LOGBOOK – 8th Run.xls
ALC.002.001.0725	CORING LOGBOOK – 9th Run.xls
ALC.002.001.0726	CORING LOGBOOK – 10th Run.xls
ALC.002.001.0727	CORING LOGBOOK – 11th Run.xls
ALC.002.001.0728	CORING LOGBOOK – 12th Run.xls
ALC.002.001.0729	CORING LOGBOOK – 13th Run.xls
ALC.002.001.0730	CORING LOGBOOK – 14th Run.xls
ALC.002.001.0731	CORING LOGBOOK – 15th Run.xls

- Powerpoint compilations of photos of the core as follows:

ALC.001.001.1503	Eighth Run.ppt
ALC.001.001.1511	Eleventh Run.ppt
ALC.001.001.1540	Fifteenth Run.ppt
ALC.001.001.1580	Fifth Run.ppt
ALC.001.001.1589	First Run.ppt
ALC.001.001.1618	Fourteenth Run.ppt
ALC.001.001.1657	Fourth Run.ppt
ALC.001.001.1672	Ninth Run.ppt
ALC.001.001.1740	Second Run.ppt
ALC.001.001.1751	Seventh Run.ppt
ALC.001.001.1773	Sixth Run.ppt
ALC.001.001.1789	Tenth Run.ppt
ALC.001.001.1809	Third Run.ppt
ALC.001.001.1823	Thirteenth Run.ppt
ALC.001.001.1846	Twelfth Run.ppt

37. As summarised in the Shear Strength Memorandum, the information on the 2006 cored hole was one of the pieces of information which was used to inform the conclusions in relation to the extent of unbonded lift joints and ultimately the assessment of dam stability. This included all three pieces of information listed above in paragraph 36. In this process, greater weight was placed on the information included in the spreadsheet logs as discussed further below.

38. The report [[**ALC.001.001.1683**]] does not appear to specifically address the classification of the lift joints with respect to bond. The report presents results of laboratory testing on the retrieved cores, but this appears to have been limited to testing of compressive and tensile strength. It appears that no testing of the shear strength of lift joints was performed and therefore the testing presented is not relevant to the assessment of shear strength. The report also includes selected photos of pieces of core, some of which include intact lift joints. The basis of selecting the photos that

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were included in the report is not known. As noted above, it is acknowledged that some of the lift joints are bonded.

39. On page ALC.001.001.1732 of this report, there is a table that lists 64 cores and that 30 of these were bonded. When this table mentions "cores", it is unclear exactly what this refers to as the number of cores does not correspond to the number of lifts intersected. The status or condition of the remaining 34 cores is unclear. It is not specific whether the remainder had no bond or did not include a lift joint.

40. The information in report [[**ALC.001.001.1683**]] did not materially inform our assessment.

41. The spreadsheet core log, referenced in paragraph 36, specifically note the reason for breaks in the core with these classified as either "segregation", "sawed to fit in box", "no bond", or "drill machine". The last classification is understood to mean that the lift joint was originally bonded but was broken by the drilling process. It is noted that in some cases, the core was broken and more than one piece was retrieved for each lift. There is also a comments field that noted whether there was good bond on the lift joint as well as other comments on the condition of the core. It is considered that this classification of the lift joints is in accordance with the Dr Schrader's recommendations in [[ALC.002.001.0936]].

42. The Shear Strength Memorandum states that 108 lifts were intersected, however this was subsequently reviewed and it was identified from the logs that the core hole intersected only 98 lifts. In the Shear Strength Memorandum, it is stated that 78% had no bond and 22% had good bond or were broken by the drilling process, noting that this is based on intersection of 108 lifts. Based on reassessment for the 98 lifts, it was identified that 68% were logged as having no bond with 27% as either having good bond or broken by the drilling process. A further 5% were not classified or the

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classification was not clear on the logs. This difference, that is 78% unbonded as reported in the Shear Strength Memorandum compared with 68% as per the revised assessment, has no material effect on the assessment undertaken with respect to the presence of unbonded lift joints. It must be highlighted that our assessment is based solely on the classification of the lift joints as reported by the Alliance. This point will be corrected in the revision of the Shear Strength Memorandum that is in progress.

43. The core photographs of the 2006 hole included in the Powerpoint compilations, referenced in paragraph 36, were reviewed but greater reliance was placed on the classification of the lift joints as presented on the spreadsheet logs.

44. In 2014-2015, Sunwater undertook a programme of investigations which included RCC coring and laboratory testing to investigate the lift joint shear strength. These results were considered in the Shear Strength Memorandum (refer Section 3). At the time of doing this testing, Sunwater and the Technical Review Panel engaged by Sunwater at that time (2015 TRP), noting this is a different TRP to that currently engaged by Sunwater for the preliminary design project, made an assessment of the shear strength based on the interpreted condition of the dam and the laboratory test results.

45. The TRP report dated 15th December 2015 (which is included in Appendix K of the 2016 Sunwater CRA report [[**IGE.068.0001**]]), specifically Section 9.2.2 (e), notes concerns about the "lack of quality in the RCC layer boundaries in the secondary spillway". The report goes on to state "we will need to be very conscious about these weaknesses elsewhere". The conclusion in this section of the 2015 TRP Report is that the strength of the lift joints should be at least 45°. No mention is made of including a cohesion intercept to be applied in addition to the frictional strength.

46. The 2014-2015 direct shear testing was undertaken on two samples of 83 mm diameter core from vertical (or near vertical) boreholes DD600 (left abutment) and DD601

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(secondary spillway) and one sample of 142 mm diameter core from a horizontal hole in the toe of Monolith G in the primary spillway. The locations of DD600 and DD601 are shown on the plan [[**WJY.001.001.0001**]]. The specific location for the horizontal core in Monolith G is not known, except that the test certificate notes it is from Ch 340.

47. As discussed above, GHD was engaged by Sunwater in October 2018 to undertake the preliminary design of the dam improvement works. One of the tasks included in the scope of that project was to undertake an independent evaluation of the shear strength based on the available data and to re-evaluate the existing dam stability. In November 2018, GHD re-evaluated the 2014-15 shear strength laboratory test data and derived a shear strength of 37-38° with zero cohesion. This was based on statistical assessment of the data with the intent of meeting the requirements of the ANCOLD Guidelines in relation to 80% of the results exceeding the adopted design strength. We do not have details of the previous strength evaluation undertaken by Sunwater and the 2015 TRP so we were not able to comment on the differences in the assessed shear strength.

48. It is noted that the RCC aggregate had a grading with 100% passing 63 mm, at least 98% passing 51 mm and at least 92% passing 38 mm. It is acknowledged that the larger aggregate, if present across the failure plane in the direct shear test, can influence the results when undertaking tests on smaller diameter cores. With specific reference to the tests on the two 83 mm diameter cores from the 2014-2015 investigations, the direct shear tests were done on lift joint samples where there appears to be no bond and no coarse aggregate interlock across the lift joint. This is evident, specifically in the photos of the sample from DD600 from before and after the test which are included on the test certificates attached to the Shear Strength Memorandum. It is noted that the photos of the sample from DD601 indicate that there was breakdown in the material in the top half of the sample during the test but the lower half shows a relatively planar

surface. The lack of coarse aggregate interlock across the failure plane confirms the validity of these tests with respect to consideration of particle size.

49. Stability analyses were undertaken using this design shear strength from the November 2018 reassessment and the results of the stability analyses were initially reported to Sunwater in December 2018 (noting that this assessment was then updated as presented in the Dam Stability Memorandum dated September 2019). These analyses, as reported in December 2018, indicated that the stability of the dam was not acceptable when compared with the ANCOLD acceptance criteria.

50. Because of the results of these stability analyses, additional sampling and testing were recommended in the summary of the December 2018 stability assessment. The decision was made by Sunwater, after consultation with GHD, to obtain samples from 150 mm (nominal) diameter cores drilled horizontally along lift joints from the downstream toe of the secondary spillway and left abutment monoliths.

51. SMEC was engaged by Sunwater to undertake the site investigations and the laboratory testing was done by Trilab as a subconsultant to SMEC. Horizontal cores were taken at eight locations at the toe of Monoliths B, C, N, P, Q, R, S and U. The locations of these are shown on [[**WJY.001.001.0001**]] , noting that they are labelled as RCC-B, RCC-C etc. No samples were taken from the primary spillway monoliths. The reason for this is that the downstream face of the primary spillway has a reinforced concrete skin and it is not possible to specifically pin-point the location of the lift joints. On the secondary spillway and left abutment, the RCC lift joints are exposed and it is easier to locate the coring rig to sample along a lift joint.

52. I am aware that, in locations where unbonded lift joints were intercepted by the horizontal core holes, there was difficulty in retrieving a sample suitable for testing. I also understand there was difficulty in following the alignment of the lift joint due to

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both equipment setup and variability in the as-constructed lift joint level and alignment. I do not specifically recall how I was made aware of these points.

53. Given the concerns in relation to the likely presence of unbonded lift joints within the Dam, the intention was to undertake laboratory tests on unbonded lift joint samples obtained from these horizontal cores. As a result of the challenges noted above in relation to obtaining unbonded samples, the decision was made to test bonded samples and develop a testing method to yield a strength which would be representative of the interpreted condition of the lift joints.

54. The logs, photographs and other data obtained from the 2006 post-construction hole drilled by the Alliance, the 2014-15 Sunwater investigations, and the 2019 SMEC investigations indicate that of the order of 60-90% of lift joints are unbonded.

55. Based on limited review of the construction records, it is understood that cold lift joints were common and as such, the top surface of the lower lift would be a smooth rolled surface, with no mechanical interlock across the joint. For this lift joint condition, there is expected to be minimal difference between the peak unbonded strength and the residual strength. This was typically the observed behaviour in the 2015 testing.

56. The direct shear testing of the RCC samples was undertaken in accordance with ASTM D5607 [[**WLJ.001.001.0001**]], with the staging and sequencing of the multi-stage direct shear test developed to derive a residual strength on the lift joints.

57. It is noted that the results of GHD's assessment as presented in the Memoranda, the current TRP for the preliminary design, and the report of Tatro Hinds dated 25 November 2019 'Paradise Dam Shear Strength Evaluation Comments' (**Tatro Hinds Report) [TAT.001.0001]** all concluded or endorsed that there is likely to be unbonded lift joints within the dam. As noted above, comments from the 2015 TRP

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Report also highlight the lack of quality in the lift joints and the need to be conscious of this in the assessment.

58. With specific reference to the Tatro Hinds Report, it is noted that they make the following comment in the section entitled "Summary of Findings":

> "After review of the GHD reports and the supplied reference information we have concluded the observed and anticipated extent of unbonded lifts in the structure is sufficient to invoke the ANCOLD requirement to assume no bonded lift joints in the structure and limit stability analyses to only residual shear strength."

59. This statement supports the conclusion of the Shear Strength Memorandum that unbonded lift joints are widespread. On this basis, residual strength should be used together with the appropriate acceptance criteria from Table 6.1 in the ANCOLD Guidelines.

60. Guidance notes are included with Table 6.1 of the ANCOLD Guidelines (Guidance Notes). In Guidance Note No 2, it is stated that "for these lower FoS to apply the residual strength c' would normally be expected to be zero" and "it is common practice to assume c'=0 and $\Phi' = 45°$ for residual strength of concrete". Section 5.1 of the ANCOLD Guidelines states that the lower bound sliding friction strength (or peak unbonded strength) is 38° and that the residual strength could be 2-3° less than this, ie 35-36°. It is understood that this is based on data for conventional concrete. The residual shear strength derived from the testing to date is within the range of the published data presented in the ANCOLD Guidelines.

61. In my opinion, the focus should be on the insitu condition of the lift joints within the dam, that is whether they are bonded or unbonded, and as discussed above, the inferred condition of the lift joints then dictates an applicable range of shear strength values.

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62. **The Tatro Hinds report**

63. I was asked to comment, based on my experience, on the Tatro Hinds Report.

64. Key findings from the review presented in the Tatro Hinds Report are listed in the section entitled "Summary of Findings" and this includes the following:

- "After review of the GHD reports and the supplied reference information we have concluded the observed and anticipated extent of unbonded lifts in the structure is sufficient to invoke the ANCOLD requirement to assume no bonded lift joints in the structure and limit stability analyses to only residual shear strength."
- "The laboratory testing of residual shear strength provides reliable values of shear strength and the noted testing issues are not expected to have a significant impact on the test results determined to date."

65. These findings support the approach and findings presented in the Memoranda.

66. In the body of the Tatro Hinds Report, concerns are expressed predominantly in relation to the multi-stage testing of the lift joint samples and the number of samples tested.

67. In this regard, I note that the ASTM standard (D5607) referred to in paragraph 56 used for the RCC shear strength testing allows multi-stage testing.

68. In addition to this, it is noted that a publication from the Electric Power Research Institute entitled "*Uplift Pressures, Shear Strengths, and Tensile Strengths for Stability Analysis of Concrete Gravity Dams*" (**EPRI 1992**) [[**WYJ.001.004.0001**]], which is the source of the recommended shear strengths in the ANCOLD Guidelines, makes the following statement on direct shear testing of concrete samples (refer Section 12):

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"Direct shear tests are commonly performed as multi-stage tests in which the sample is tested at more than one normal stress. This procedure increases the amount of data available from one sample.

"Intact samples are broken to determine peak strength, then are tested at one or more normal stresses to determine residual strengths."

69. The approach adopted by GHD for the 2019 direct shear testing of the horizontal core samples is in line with this statement.

70. To the best of my recollection, Mr Tatro himself made comments to the effect that the United States Bureau of Reclamation do multi-stage testing on concrete during two meetings I attended. The first was a teleconference on 13 November 2019 and the second was the TRP workshop on 19 November 2019. At the first of these meetings, he also commented that the ASTM standard used for the testing allows multi-stage testing.

71. It is therefore not the case that this kind of multi-stage testing is not an accepted approach in the industry, it is understood to be Tatro Hinds' approach not to do multi-stage testing.

72. GHD was aware of the potential for the results of subsequent stages of testing to be affected by repetitive shearing. This is commonly acknowledged with respect to multi-stage testing and is noted in ASTM D5607, for example in Section 8.6.2.5. This standard also notes in Section 8.6.2.4 that "in order to reduce the potential for the effects of specimen degradation and wear, each consecutive stage should be performed with a higher normal load". Both these points were considered in developing the testing methodology.

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73. GHD, in developing the testing programme, was aiming to maximise the amount of information we could extract from a relatively limited amount of core that was suitable for testing.

74. Laboratory tests were performed across the relevant normal stress range. The multi-stage testing for this phase of the project involved testing the samples at 200 kPa, 500 kPa and 1000 kPa. The laboratory cut typically three separate samples from each section of the horizontal core which was selected by GHD and Sunwater. Each section of core was typically about half a metre to a metre long.

75. The first sample from the section of core was tested at 200 kPa to obtain a peak bonded strength. This was then tested at 200 kPa to obtain a peak unbonded strength (or sliding friction strength). The normal stress was then incrementally increased to 500 kPa and 1,000 kPa and tested to obtain the peak unbonded strength at the higher normal stresses. The load was then lowered and the sample tested at 200 kPa, 500 kPa and 1,000 kPa to obtain the residual strength. Finally, a check was made of the residual strength at 200 kPa by testing to a larger displacement.

76. The second sample was tested at 500 kPa to obtain a peak bonded strength and peak unbonded strength. This was then tested at 200 kPa and 1,000 kPa to obtain the peak unbonded strength. The residual test stages were conducted in the same order as the first sample. The check on the residual strength for the second sample was undertaken at 500 kPa.

77. The third sample was tested at 1,000 kPa to obtain a peak bonded strength and peak unbonded strength. This was then tested at 200 kPa and 500 kPa to obtain the peak unbonded strength. The residual test stages were conducted in the same order as the first and second samples. The check on the residual strength for the third sample was undertaken at 1,000 kPa.

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78. The ordering of the load increments for the peak bonded and peak unbonded acknowledged the potential for damage by repetitive testing. The sequencing of the testing for residual strength acknowledged the recommendation in the standard to incrementally increase the testing between stages. This multi-stage testing methodology is consistent with the approach outlined in EPRI (1992) and is in accordance with the ASTM standard used for the testing.

79. For the peak bonded and peak unbonded tests, we therefore have tests on samples that have not previously been sheared at each of the three normal stresses. GHD has compared the results of the testing from each separate stages of testing for the peak unbonded tests. The samples that were tested as the first stage (which are not the subject of criticism in the Tatro Hinds Report) provide similar indications of the shear strength as the samples that were tested in the second or third stages at the same normal stress. If anything, the second stage results yielded higher shear strengths than the first stage for the normal stresses considered.

80. With respect to the residual strength results used to derive the strength as summarised in the Shear Strength Memorandum, the results from only stages 1 and 2 were used as these results were representative of the expected normal stress on the failure plane in the stability analyses. Taking into account the Tatro Hinds comments on the effects of repetitive shearing, a residual shear strength of approximately 43° and no cohesion is obtained if only the first stage residual test results are used. If this strength is used in the stability analyses, the stability of the dam improves but does not meet the ANCOLD acceptance criteria.

81. In terms of the adequacy of the number of tests that were undertaken, more data is always beneficial, so I would support the idea that additional testing would be useful. With that said, there is a fair degree of consistency in the results of the testing we have

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obtained. In relation to this point, the comment made by the TRP in their third report dated 9 December 2019 [[**SUN.009.002.0001**]] (refer Section 2) is as follows:

> "More sampling is desirable but on the basis of test results to date it is difficult to see that strengths will increase such that conclusions on stability and risk reached by GHD will change markedly."

82. With respect to the comments in the Tatro Hinds Report in the 5th para of the section entitled "Shear Test Method" concerning the apparent degradation of the shear surface and the effect that might have on the shear strength, GHD reviewed our results against published shear strength data from EPRI (1992) as presented in the ANCOLD Guidelines. The shear strength that we have derived from the laboratory testing is consistent with the range of shear strengths listed in EPRI (1992) and the ANCOLD Guidelines for unbonded lift joints.

RCC construction data

83. In relation to RCC construction practices at the Dam, I have only reviewed limited information provided by Sunwater. Sunwater has undertaken the majority of the data mining in relation to the construction information and have shared some of that information with GHD. Some of the information shared was construction memoranda that Sunwater had selected from the archive. Those documents provide some limited understanding of the challenges faced during construction.

84. GHD received this information after issuing the Memoranda in September 2019.

Adequacy of primary spillway apron

85. The main purpose of the apron at the downstream toe of the primary spillway is to protect the downstream foundation during spill events, specifically the area where the hydraulic jump occurs and the energy dissipation is greatest.

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86. The presence of the apron has an influence on the stability of the dam, in that it affects the uplift distribution under the structure. The effect of the existing apron is accounted for in the stability analyses for the lift joints at the base of the structure, but the apron is not relied on to provide any structural restraint.

87. The key mode of failure considered in the stability analyses, as presented in the Stability Memorandum, is failure through the RCC on the lift joints. Neither the adequacy nor integrity of the apron is part of that assessment. The stability of the dam with respect to failures through the foundation was assessed by another consultant and this work is presented in Sunwater's 2016 Dam Safety Review. This included consideration of varying extents and depths of scour downstream of the Dam. GHD undertook only a high level review of these analyses, but ultimately we were reliant on the results of the stability analyses done by others for the existing Dam as an input to the risk assessment.

88. The risk assessment considered the potential for the apron to fail during a flood event, in which case the foundation downstream of the dam and under the downstream toe of the Dam would be exposed to the erosive power of the water. This could lead to undermining and failure of the Dam.

89. In terms of whether the apron is adequate, the performance of the apron in the 2013 flood indicates that it was not, given the extent of scour that occurred particularly on the left side of the primary spillway and also the damage that occurred to the apron during that event.

90. As part of the preliminary design project, GHD has been undertaking hydraulic modelling to inform our decisions on the required size of the stilling basin. The modelling is being done with computational fluid dynamics (CFD), which is a numerical modelling method. This modelling initially included validation against the

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results of physical hydraulic modelling that was undertaken by Sunwater in 2014 to 2015. The CFD modelling has informed the development of the arrangement which we have proposed in the preliminary design of the improvement options. This analysis is summarised in GHD's Draft Preliminary Design Report.

91. One of the options that is being developed is to retain the full supply level (**FSL**), that is, the crest of the primary spillway, at its current level. For the option where the FSL is retained at its current level, the length of the stilling basin proposed by GHD on the basis of the CFD hydraulic modelling is 60 metres.

92. In terms of construction material, the arrangement GHD is proposing for the new extended stilling basin is reinforced conventional concrete with extensive dowels to anchor the stilling basin slab to the foundation. My understanding is the original as-constructed apron, before the remedial works undertaken after the 2013 flood, was reinforced and anchored RCC.

93. For the option with FSL retained at its current level, the preliminary design also includes a large channel on the downstream side of the secondary spillway, which has been called a side channel, to collect the flow that overtops the right bank secondary spillway and convey that flow back to the main river channel. The existing apron downstream of the secondary spillway is about 10 m wide, with a low height end sill of 1 m height. The structure presented in the preliminary design is significantly larger. The intent of the proposed side-channel is to protect the foundation downstream of the secondary spillway from scour that could undermine and fail the secondary spillway monoliths.

Foundations

94. In terms of the global stability of the Dam, the foundation conditions are a factor. There is ongoing work in relation to this as part of the preliminary design project. The

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first part of that work was collation of historical information from previous phases of investigations. Further site investigations were undertaken in 2019 and laboratory testing has recently been completed. The review and interpretation of that information is ongoing. Following that, there may be a need to review the risk for the existing Dam if there are changes with the failure mechanisms or the material strengths.

95. GHD's geological and geotechnical assessment of the foundations is not likely to be completed until at least April 2020.

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***OATHS ACT* 1867 (DECLARATION)**

I, James Clayton Willey, do solemnly and sincerely declare that:

(1) This written statement by me dated 21 February 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 21 st day of February 2020.

Taken By D. M. Dest **.....**

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

	D. M. Dest
Witness	Taken by