Commission of Inquiry
PARADISE DAM

REPORT

APRIL 2020

Acknowledgement of Country

The Paradise Dam Commission of Inquiry acknowledges the traditional owners and custodians of the lands and waters across the State of Queensland, and pays its respects to Elders past, present and emerging.

Aboriginal peoples and Torres Strait Islanders should be aware that this publication may contain images or names of deceased persons.

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Paradise Dam Commission of Inquiry
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Commission of Inquiry
PARADISE DAM

30 April 2020

The Honourable Annastacia Palaszczuk MP
Premier and Minister for Trade
PO Box 15185
CITY EAST QLD 4002

The Honourable Dr Anthony Lynham MP
Minister for Natural Resources, Mines and Energy
PO Box 15216
CITY EAST QLD 4002

Dear Premier and Minister

In accordance with the *Commissions of Inquiry Order (No. 1) 2019*, we have made a full and careful inquiry into the root cause of structural and stability issues identified with the Paradise Dam near Bundaberg.

We are pleased to present the report of the Paradise Dam Commission of Inquiry.

Yours sincerely

John Byrne AO RFD
Chairperson and Commissioner
Paradise Dam Commission of Inquiry

John Carter AM FAA FTSE FRSN FIEAust FAIB
Commissioner
Paradise Dam Commission of Inquiry

Foreword

John Byrne AO RFD
Chairperson and Commissioner

John Carter AM
Commissioner

Humans have a long history of building dams and relying on them for their immediate water supply and future water security. The first recorded attempts to construct dams date back to the Egyptians and later, the Mesopotamians and the Romans. Closer to home, there is evidence of dams having been constructed in Australia well before the arrival of Europeans. Since those early attempts at dam building, the necessary technology and our collective understanding of the forces of nature that a dam must safely resist have advanced considerably. Society generally takes it for granted now that dam construction is a 'tried and true' activity. The community expects that any risks associated with dam construction are identifiable, quantifiable and manageable.

But occasionally things may not always go to plan when building a dam. Doubts may arise about the adequacy of the design of a dam or its construction. In recent times such doubts were raised with respect to Paradise Dam (**the Dam**). The Queensland Government initiated several investigations of matters relevant to the Dam's safety and the consequences should the Dam fail. One such investigation was the Paradise Dam Commission of Inquiry, the outcomes of which are described in this report.

The Terms of Reference of the Inquiry direct the Commission to find the 'root cause' of structural and stability 'issues' at the Dam. In investigating and reporting on those 'issues', a broad interpretation of root cause has been applied.

The search has not been confined to considering factors which, if removed, would certainly prevent recurrence of the problematic issue. Nor have the investigations been constrained by the approach that commonly would be taken in deciding legal causation, such as whether a factor – for example, a decision made in relation to design – contributed significantly to a stability issue. Instead, for the purpose of this Inquiry, a factor is perceived as a root cause if it significantly increased the risk of a relevant, adverse consequence.

The Dam is a major piece of public infrastructure that is important to the regional economy. It was built to supply water to the Bundaberg Irrigation Scheme. Growers in the Wide Bay – Burnett area supply fresh produce to the domestic and overseas markets, so the Dam is of considerable importance to the economy and to livelihoods.

In preparing this report, the Dam's significance to the region and to Queensland and its economy more generally have not been ignored. But as the Terms of Reference demand, we have focussed the Inquiry on the pursuit of what if anything went wrong with the design and construction of the Dam, and what important lessons could be learned that may be applied in the future to dam construction in Queensland and elsewhere.

Consideration of any remedial measures to ensure the Dam's safety, should they be required, has not been undertaken because such consideration is outside our Terms of Reference.

We would like to thank all the talented team members who assisted with the work of this Commission of Inquiry. Their skill and dedication to completing the task in a relatively short time frame while also being confronted by the COVID-19 pandemic are much appreciated.

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Executive Summary

Background

1. Paradise Dam (**the Dam**) was designed and built between 2003 and 2005. It is a gravity dam, made mainly from low cementitious, or 'lean mix', roller compacted concrete (**RCC**). The Dam is constructed of many RCC layers, also known as lifts, each about 300 mm thick. The Dam is the largest volume RCC dam in Australia.

2. The builders and designers were part of a group (**the Alliance**). Its original members were Burnett Water Pty Ltd (**Burnett Water**), Hydro Electric Corporation (trading as Hydro Tasmania), SMEC Australia Pty Ltd, Macmahon Contractors Pty Ltd and Walter Construction Group Limited (**Walter**). Walter's participation ended during construction.

3. Burnett Water was a 'special purpose vehicle' that was not intended to become the eventual owner of the Dam. SunWater Limited (**SunWater**), which had expertise in dam design, provided technical advice to Burnett Water about preliminary design. SunWater, which was not a member of the Alliance, eventually acquired the shares in Burnett Water and became the Dam's operator.

4. In late 2005, the Dam's lead designer certified the Dam as safe to fill to full supply level and that the works as constructed had been undertaken in a manner which met the design requirements.

5. The Dam was impounded in late 2005. The Dam first filled in March 2010. It experienced flooding in December 2010 and January 2011 (**the 2011 event**) and in January 2013 (**the 2013 event**). The floods resulted in damage to the primary spillway apron. The 2013 event also caused scouring of the riverbed immediately downstream.

6. The extent of the damage and scouring had not been anticipated. The 2011 and 2013 events were smaller than the maximum flows that the Dam had been designed to withstand.

7. As a consequence, investigations were commissioned.

Structural and stability issues

8. The investigations resulted in reports from technical review panels (**TRPs**) and the work of specialist consulting engineers, GHD Pty Ltd (**GHD**). Those studies revealed structural and stability issues with the Dam that the Commission has examined, principally:

    a. Sliding stability or shear strength. This concerns the risk of sliding along the interface between consecutive RCC layers, i.e. along lift joints.

    b. Scour protection immediately downstream of the primary spillway. This relates mainly to the sufficiency of the width of the apron and the capacity of the RCC from which it was constructed to resist the erosive force of water overtopping the main spillway.

Stability: shear strength

9. The principal building material for the Dam was RCC. The critical design parameters were values influencing the shear strength of the lift joints formed between consecutive layers of RCC (cohesion and friction coefficient or friction angle). The designers adopted values of the design shear strength parameters on the advice of the RCC advisor.

10. The friction angles were conservative according to industry guidelines. The cohesion values were not. As designed, the Dam's shear strength also relied upon some cohesion from 'bedding mix' inserted between lift joints.

11. Despite the relatively high cohesion values, the RCC was not tested in a laboratory or *in situ* during design or construction to determine if the design values could be achieved. Without confirmation testing, assessing whether the design parameters had been met relied on the construction quality assurance program.

12. In 2015, testing of core samples retrieved from the Dam called into question whether the design values had been attained. Despite further testing since 2015, doubts remain about whether the assumed design values of friction angle and cohesion have been achieved in the Dam as-constructed.

RCC construction practices

13. Without shear strength testing, the quality of construction became the main indicator whether the Dam had achieved the design parameters for shear strength. Construction quality was critical to bonding between the RCC layers and the resistance of lift joints to sliding.

14. Consecutive RCC layers had to be placed within relatively short times. Otherwise, a 'cold joint' resulted. That often happened. The Specification required that a cold joint be treated with bedding mix to give a better bond between successive layers. During construction those requirements were halved by the RCC advisor. The situation was complicated by the RCC advisor's claim that the Dam '*essentially achieves stability with current friction values alone*'. These things have contributed to the uncertainty about whether the design cohesion values have been attained.

15. At times, there were construction problems, including with segregation, density, compaction, cleaning and curing of RCC lifts: all had some potential to prejudice stability. Mostly, the problems were detected and remedied. Where they were not, the problems seem unlikely to have put the Dam in jeopardy but their cumulative effect is uncertain.

16. The Specification for the Dam's construction had adopted the RCC advisor's Lift Joint Quality Index (**LJQI**) as a construction quality control measure. The LJQI provided a useful check list for inspectors in detecting problems in construction.

17. However, the Alliance appears to have used the LJQI to check that the lifts achieved the assumed strength parameters, which placed more reliance on the LJQI than its author endorses and is problematic: it was not a suitable tool to estimate the shear strength of lift joints.

Is the Dam stable?

18. In circumstances that are no more severe than those it experienced in the 2011 and 2013 events, the Dam is stable. Uncertainty, however, attends the prospect of much larger floods. A stability assessment for more severe loads depends upon assumptions about which the experts disagree.

19. A factor of safety is an indicator of the capacity of a dam to withstand particular loads. GHD's computed factor of safety for sliding failure of the Dam was below 1 for 'Extreme' flood loads. This is less than the acceptance criteria in current Guidelines on Design Criteria for Concrete Gravity Dams published in 2013 by the Australian National Committee on Large Dams (**ANCOLD**). So, on GHD's conservative assumptions, the Dam is unsafe in relation to shear sliding in some remote eventualities. Other expert assessments, reflecting different inputs and assumptions, are not so pessimistic.

Root causes of uncertainty as to instability

20. The differences among the experts about stability account for the uncertainty on the topic. Other things contribute to the uncertainty.

21. It would have been preferable, and consistent with engineering good practice, if the Alliance had commissioned testing for shear strength of the lift joints. Its absence has contributed to the uncertainty about stability.

22. Moreover, a lean mix RCC is difficult to reliably sample and test. It is more difficult to work with and less forgiving than a higher cementitious mix. Those constructability issues were exacerbated because peak RCC placement occurred during the warmest and wettest months of the year. Using such a low cementitious RCC mix is another root cause of the uncertainty.

23. There was no proper peer review of the RCC aspects of the design (including the mix and its material properties). A better peer review process would likely have identified that the design values for cohesion were not conservative. Probably, it would have recommended a more conservative design or that confirmation testing be done. Absence of suitable peer review of RCC aspects is also a root cause of the uncertainty about the Dam's stability.

Possible causes of instability if it exists

24. Expert opinion is that more testing is needed to resolve the doubts about stability.

25. SunWater intends to conduct further testing. If that were to show that the design values were not achieved, there seem to be two possible explanations.

26. First, the RCC mix may intrinsically have been incapable of meeting the design values – particularly the less conservative cohesion assumptions.

27. Secondly, all construction quality problems may not have been remediated. While the quality assurance procedures were generally effective in identifying quality issues and in ensuring their remediation, the primary means of detecting lift joint problems, the LJQI, was applied in ways that masked deficiencies that it was supposed to detect. Accordingly, unremediated problems may have prejudiced the bonding at lift joints to such an extent that the Dam has not attained the design parameters for friction or cohesion.

Downstream protection

28. A root cause of the scour and erosion immediately downstream of the primary spillway apron was the apron's insufficient, 20 m width.

29. Advice from geotechnical engineers did not provide detail on the erodibility of the rock immediately downstream of the apron. And hydraulic studies gave the design team undue confidence that the erosive forces of the water would be largely contained within a 20 m apron.

30. It cannot be known with certainty what the result of comprehensive peer review would have been. But this accepted means to discover and correct problems was absent for the apron. Its absence created a significant risk of inadequate design of the apron. In this sense, the lack of full peer review of hydraulic structures was a root cause of the apron's inadequate width (and, therefore, also the scouring).

Governance and third party review

Special purpose vehicle

31. A 'special purpose vehicle' is not inherently deficient. Burnett Water had no history of designing, building and operating dams. Its capacity to contribute to the project as a 'true client', with resources and expertise of its own, was limited. Consideration needs to be given to such limitations when deciding how involved a special purpose vehicle should be in the design and construction of large infrastructure projects.

The 'Alliance' model

32. There are no inherent deficiencies with the alliance model that was used. Earlier identification of the ultimate owner, SunWater, may have allowed it greater participation in design and construction. In this instance, SunWater had knowledge and experience that could have been provided from a 'true client's' perspective. Because SunWater was not identified as the owner until after the Alliance was formed and construction had commenced, it could not sensibly have joined the Alliance.

Compliance with conditions of the development permit

33. Conditions were imposed upon Burnett Water in its development permit to construct the Dam. A couple were not met: preparation of a summary 'construction report' of the as-constructed documentation; and the terms of the certifications by the lead designer. The deficiencies are immaterial.

34. Dam safety conditions required Burnett Water to summarise the as-constructed documentation and incorporate it into the design report or produce it as a separate report. No such summary was prepared. Its absence, however, did not impair the Commission's work. All the construction documentation the Commission needed was provided. Nor has its absence prejudiced the safety of the Dam. All the construction documentation from which a summary is usually prepared was provided to SunWater and is held for future reference as required.

35. The conditions required '*certification by a registered professional engineer … that the works have been constructed in compliance with all appropriate engineering standards including signed statements from the dam designer that principal components of construction have been inspected and approved'.* Neither of the two certifications by the Principal Dam Designer was in the same terms as those required. The difference in the language, however, did not materially contribute to the uncertainty about the Dam's stability.

Independent expert panel

36. An independent expert technical panel to provide advice during the design and construction of dams has advantages. It can advise and expose aspects of design and construction to the scrutiny of independent experts who are not burdened with the time and cost pressures of designers and builders.

Peer review

37. Engineering good practice would support peer review of the principal components and critical design parameters of a large dam.

Recommendations

38. The Terms of Reference anticipate recommendations arising out of the evidence, considerations and findings of the Inquiry to ensure that future Queensland dams are designed, constructed and commissioned to acceptable standards. The standards are those in Queensland legislation and regulations, ANCOLD guidelines and engineering good practice.

39. The recommendations are set out below, along with a brief explanation of how they arise out of the Commission's work.

Independent verification of critical design parameters

40. **Recommendation 1**: The materials used to construct a dam and the dam as-built should be subjected to inspection and physical testing to confirm the values adopted for critical design parameters. It is preferable that those responsible for the dam's design and construction organise and oversee such testing.

41. Despite testing since 2015, doubts remain whether the assumed friction angles and cohesion values have been attained. Such uncertainty is undesirable. It is preferable that those who design and build a dam commission independent confirmation testing.

Independent technical review and peer review

42. **Recommendation 2**: The Commission encourages consideration by the Regulator of mandating the independent technical review of referable dam projects.

43. **Recommendation 3**: The panel or body established to conduct the independent technical review should have the authority to co-opt others with appropriate expertise to conduct peer review of matters beyond the collective expertise of the panel members or where obtaining additional views is considered advisable.

44. **Recommendation 4**: Matters for review should include but may not be limited to regulatory, safety and operational requirements, the principal components of the dam and its critical design parameters.

45. **Recommendation 5**: The Regulator should consider how best to ensure the independence of the persons chosen to conduct peer reviews and whether guidelines to assist and direct those in peer reviewing dam projects would be useful.

46. There was no independent technical review panel when the Dam was designed and built. Such panels have worked well for other dams, including Wyaralong Dam, and are commonly used now. They are sources of advice and scrutiny. The checks and balance they bring will be especially valuable where novel technologies or methods are employed.

47. Peer review of principal elements of a dam is engineering good practice and a further way of subjecting elements of design to the scrutiny of other experts for the detection and correction of problems and oversights. Future dam projects would benefit from proper peer review to ensure they are designed, constructed and commissioned to acceptable standards.

Downstream protection

48. **Recommendation 6**: The designer of a dam should give proper consideration to the erosive force of water and the capacity of the riverbed to withstand such force. This may include testing and simulation using computational and hydraulic modelling, as well as geotechnical investigations (and the interaction between those disciplines).

49. The designer of a dam needs to understand the erosive force of the water and the capacity of the riverbed to withstand it. This involves managing the interaction between geotechnical engineers and hydraulic engineers.

Monitoring compliance

50. **Recommendation 7**: The Regulator should consider suitable means of routinely monitoring compliance with conditions of development permits and other approvals relating to the construction of dams, including by audits and checks during construction.

51. It has not been the practice to conduct on site audits during a dam's construction to monitor compliance with conditions imposed by development permits, including a condition requiring the dam to be built in accordance with the Dam Safety Management Guidelines 2002, and other approvals. Those guidelines provide for good practice with respect to design, construction and management.

52. The Regulator could appoint persons to investigate compliance with development conditions and dam safety conditions. Increased monitoring during design and construction could conduce to the better governance of dam projects. Audits and checks are among the ways in which compliance can be monitored by the Regulator.

Model for dam delivery

53. **Recommendation 8**: To the extent practicable, the entity that is ultimately to own or operate the dam after its commissioning should have an opportunity to influence its design and construction; and if there is an alliance, preferably as part of that structure.

54. A 'true client' – the end owner or operator – is well placed to advise and test functional requirements and therefore to assist and guide design and construction. Its early involvement is desirable.

Recommendations

Recommendation 1: The materials used to construct a dam and the dam as-built should be subjected to inspection and physical testing to confirm the values adopted for critical design parameters. It is preferable that those responsible for the dam's design and construction organise and oversee such testing.

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Chapter 1 – Introduction

Background to the Inquiry

1.1 Paradise Dam (**the Dam**) is located on the Burnett River about 20 km North West of Biggenden and 80 km South West of Bundaberg.[1] Its main purpose is to supply the Bundaberg Irrigation Scheme.

1.2 The Dam first filled in March 2010.[2] It experienced flooding in December 2010 and January 2011 (**the 2011 event**) and in January 2013 (**the 2013 event**). The floods resulted in damage to the primary spillway apron. The 2013 event also caused scouring of the riverbed immediately downstream.

1.3 Technical and engineering studies were commissioned. They identified structural and stability issues with the Dam.

1.4 On 29 November 2019, the Honourable the Minister for Natural Resources, Mines and Energy announced the Government's intention to establish this Commission.

The Commission

1.5 The Paradise Dam Commission of Inquiry was established by Commissions of Inquiry Order (No. 1) 2019 made under the *Commissions of Inquiry Act* 1950. That Order, **Appendix 1**, was notified in the Government Gazette on 6 December 2019. It stated the Terms of Reference:

> *3. UNDER the provisions of the Commissions of Inquiry Act 1950 the Governor in Council hereby appoints the Honourable John Harris Byrne AO RFC as Chairperson and Commissioner, and Emeritus Professor John Phillip Carter AM FAA FTSE FRSN FIEAust FAIB as Commissioner, from 6 December 2019, to make full and careful inquiry in an independent manner with respect to the following matters:*
>
> *a. the root cause of structural and stability issues identified in engineering and technical studies conducted on the Paradise Dam between 30 January 2013 and 30 November 2019;*
>
> *b. where the root cause is attributable, or attributable in part, to the design, construction and/or commissioning stages of the Paradise Dam, the facts and circumstances that contributed to the structural and stability issues having regard to:*
>
> *i. the governance arrangements in place including expert third party review and response to any issues raised;*
>
> *ii. the scope and effectiveness of processes and systems to ensure quality in design, construction and/or commissioning, adopted*

1 Exhibit 2, **ALL.155.008.0001**, .0006.
2 **DNR.002.7814**, .0003.

by individuals, entities and government bodies involved in the design, construction or commissioning of Paradise Dam; and individuals, entities and government bodies involved in giving the necessary approvals required for the Dam;

iii. the reporting arrangements and obligations in place during design, construction and commissioning;

iv. remedial measures taken during design, construction and commissioning;

v. any other matter relevant to the Inquiry.

4. THE Commissioners may make any recommendations arising out of the evidence, considerations or findings of the inquiry in relation to the matters set out in paragraphs 3a) and b) above that the Commissioners consider appropriate to ensure future Queensland dam projects are designed, constructed and commissioned to acceptable standards, as defined in Queensland Government legislation and regulation, Australian National Committee on Large Dams guidelines and engineering good practice.

1.6 By 30 April 2020, the Commission was to transmit its report to the Honourable the Premier and Minister for Trade and the Minister for Natural Resources, Mines and Energy.

1.7 Steps taken in establishing the Commission and its later operations are outlined in **Appendix 2**.

1.8 Counsel Assisting were chosen. Staff with policy, technical, media, research and administrative expertise were appointed. The Commission's staff members are identified in **Appendix 3**.

1.9 **Appendix 4** lists the parties given leave to appear and their legal representatives.

Scope

1.10 The Terms of Reference call for inquiry into root causes[3] of those structural and stability issues identified in engineering and technical studies conducted on the Dam between 30 January 2013 and 30 November 2019.

1.11 At the first public hearing, the Chairperson and Senior Counsel Assisting made opening remarks: see **Appendix 5**. Senior Counsel Assisting nominated the studies that appeared to be those referred to in the Order and listed the structural and stability issues identified in them.

[3] As the Foreword explains, a factor is perceived as a root cause if it significantly increased the risk of a relevant, adverse consequence.

1.12 Senior Counsel Assisting subsequently proposed 'Key Issues' to be pursued:

1. Structural and stability issues (Terms of Reference paragraph 3(a))

1.1 Paradise Dam's (the Dam's) sliding stability. In the GHD Paradise Dam Stability Assessment dated 25 November 2019, the sliding stability is assessed as not meeting the Australian Committee on Large Dams (ANCOLD) factors of safety for different flood scenarios.

1.2 The adequacy of downstream protection immediately below the Dam, principally;

a. the adequacy of the primary spillway apron's dimensions;

b. the capacity of the materials from which the primary spillway apron was constructed (and the way in which it was constructed) to resist the erosive force of water.

1.3 The strength of the Dam's foundations. The Technical Review Panel (TRP) Report Number 1 dated October 2013 concerning Phases 2 and 3 of Remedial Works states that exploratory boreholes taken in 2019 are said to show areas of open contact between the roller-compacted concrete and bedrock at the foundation of some parts of the Dam. Section 2.4 of the second report of the TRP dated 23 September 2019 provides further information in this regard.

2. The 'engineering and technical studies' material to the issues stated above are, for the purposes of paragraph 3(a) of the Terms of Reference

2.1 Technical Review Panel Reports as follows:

a. No 1 dated October 2013

b. No 2 dated January 2014

c. No 3 dated November 2014

d. No 4 dated 15 December 2015

2.2 Technical Review Panel Reports as follows:

a. No 1 dated 29 May 2019

b. No 2 dated 23 September 2019

c. Although outside the date range provided in the Terms of Reference, Technical Review Panel Report No 3 dated 9 December 2019 is relevant as it deals with many of the matters dealt with in TRP Report No 2.

2.3 Report of TatroHinds 'Shear Strength Evaluation Comments' dated 25 November 2019.

2.4 Memoranda from GHD (Mr James Willey) dated 5 September 2019 and 25 November 2019.

2.5 Draft Inspection Report of the Dam Safety Regulator in April 2013.

2.6 SunWater, Dam Safety Review, Revised Report, provided in 2016.

3. Key issues

3.1 In terms of sliding stability:

a. adequacy of the bond between the roller-compacted concrete (RCC) lifts;

b. whether the consequences of using the particular 'lean' RCC mix adopted for the Dam limited the practicability of verifying shear strength parameters by in situ testing and, for that reason, necessitated greater reliance on quality management systems that recorded whether and to what extent specified construction methodologies and practices were adhered to than for mixes with higher cementitious content;

c. whether the specified construction methodologies and practices were:

i. appropriate according to accepted practices and guidelines at the time the dam was constructed;

ii. sufficient to verify that the Dam achieved the design parameters for shear strength;

iii. adhered to, including, in particular, with respect to the laying of RCC and the treatment of lift joints (including 'cold joints') with bedding mix;

d. the adequacy of testing and checking (and the standards against which such testing was undertaken) reliably to verify that the lift joints were of a quality likely to result in a dam about which there could be reasonable satisfaction of stability and structural integrity;

e. adequacy of remediation of non-conformances and quality shortfalls identified during the Dam's construction;

f. whether the Dam, as designed, 'essentially achieves stability with current friction values alone' (see, for example, SUN.010.002.0047) and the reliance placed upon this statement

in making decisions about the design and construction of the Dam;

g. what standards are properly to be applied in assessing the Dam's sliding stability.

3.2 In terms of the downstream protection:

a. the adequacy of the dimensions, structure and quality of construction of the apron downstream of the primary spillway;

b. whether the primary spillway apron was constructed of sufficiently strong material to withstand the erosive forces of water and abrasion;

c. the design process and the accuracy and adequacy of the hydraulic modelling, including as to the energy dissipation effects that tailwater would offer, and whether the complexity of anticipated flood flows had been properly accounted for in the apron's design;

d. the appropriateness and sufficiency of geological investigations prior to and during construction of the Dam and the availability of them to the Dam's designers;

e. the effect of damage from flooding in 2010/11 and how it may have influenced (if it did influence):

i. the damage sustained immediately downstream of the Dam in 2013;

ii. the hydraulic jump.

3.3 In terms of the Dam's foundations, the adequacy of contact between the Dam wall and the rock beneath it.

3.4 In terms of governance and reporting arrangements:

a. whether the use of a special purpose vehicle (Burnett Water Pty Ltd) was attended with weaknesses in terms of separating the design and build from the ultimate owner and operator of the Dam (SunWater);

b. whether an alliance arrangement was the appropriate delivery model for the design, construction and commissioning of the Dam in the sense of having contributed to the structural and stability issues identified in paragraph 1 above;

c. whether a 'declaration' made by some or all members of the Alliance that they would use, or seek to use, less conventional

concrete was desirable and the consequences for the project of such a stance;

d. whether the use of an independent review panel during the design and construction of the Dam would likely have improved governance and provided a wider lens across design and construction activities, including the avoidance of excessive reliance upon one, or a small number, of advisors;

e. whether the Dam Safety Regulator adequately discharged his statutory functions, including in properly conditioning the development permit for the Dam and ensuring those conditions were met;

f. whether the conditions of the development permit for the Dam were met;

g. the adequacy of peer review of the Dam's design, and of changes and adjustments to that design; and

h. the circumstances which contribute to a situation where there exists uncertainty among technical experts and engineers as to the Dam's structural integrity and stability and how this might be avoided in the future.

Processes

1.13 To arrive at its findings and recommendations, the Commission undertook a variety of investigations.

1.14 **Appendix 6** names those persons with pertinent knowledge and information who were interviewed. Many of them later testified at public hearings. The witnesses were ordinarily resident in, or gave their evidence from, Canada, Colombia, New Zealand, Peru, the United States of America and four Australian States.

1.15 **Appendix 6** also mentions the witness statements, submissions from the public, and submissions made by the parties in response to a discussion paper and to notices of potential adverse findings.

1.16 The Commission collected and analysed more than 37,700 documents and photographs from Government departments, statutory authorities, public and private organisations, and individuals. The literature was researched, including as to standards, guidelines and engineering good practice bearing upon design and construction of dams.

1.17 A website allowed the Commission to share and gather information, including Practice Guidelines: **Appendix 7**.

1.18 An electronic document management system received, stored and helped in analysing the large volume of material.

1.19 The Commission conducted public hearings in Bundaberg and Brisbane. **Appendix 2** mentions the dates and venues as well as when witnesses testified.

1.20 Some experts gave their evidence together. Hearing them concurrently had several advantages: principally, that the experts could identify and discuss points of agreement and disagreement about the Dam's stability. A Protocol and Agenda guided the sessions: **Appendix 8.**

1.21 Exhibits are described in **Appendix 9**.

1.22 A glossary of terms and abbreviations in this Report appears at the end.

Location, catchment, history

1.23 The Dam lies 131.4 km upstream from the mouth of the Burnett River,[4] located within the Bundaberg and North Burnett Regional Council local government areas.[5]

1.24 The Dam has a total length of 920 m.[6] The primary spillway has an ogee crest shape and is 315 m long.[7] The secondary spillway, which is located on the right abutment, is a 485 m long trapezoidal crest section.[8] The left abutment is much shorter (120 m).[9]

1.25 The surface area of the reservoir is about 3,000 ha.[10] It creates a 45 km long, narrow reservoir with a storage volume of 300,000 ML at full supply level (**FSL**).[11] At FSL, the water level reaches an elevation of 67.6 m Australian Height Datum (**AHD**).[12]

1.26 As the Dam was built for water supply and irrigation rather than for flood mitigation, it has no flood gates.[13]

1.27 The Dam's catchment area is 30,490 km^2 – one of the largest in Queensland.[14] Annual rainfall over the catchment area varies between 650 mm in the north-east to 1,000 mm in the upper reaches of the catchment.[15] The catchment varies in elevation between 30 m and 1,219 m above sea level.[16]

1.28 Large flood flows occur at the Dam because its reservoir is small relative to its large catchment. The Dam's Probable Maximum Precipitation Design Flood (**PMPDF**) is considered to be in the top ten highest spillway discharges in the world.[17]

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4 Exhibit 2, **ALL.155.008.0001**, .0013.
5 **IGE.036.0001**, .0015.
6 **IGE.036.0001**, .0015.
7 **IGE.036.0001**, .0016.
8 **IGE.036.0001**, .0015-.0016.
9 **IGE.036.0001**, .0016.
10 **IGE.036.0001**, .0015.
11 **DNR.002.5621**, .5636.
12 **DNR.002.5621**, .5659.
13 Exhibit 26, **IGE.084.0001**, .0008.
14 **DNR.002.5621**, .5636.
15 **DNR.005.7379**, .7384.
16 **DNR.015.0590**, .0599.
17 Exhibit 237, **SWA.512.001.0578**, .0585.

1.29 The Dam was once known as the Burnett River Dam. A competition resulted in support for a name derived from the area's fleeting history as a gold mining town, remnants of which were submerged under the waters of the reservoir. **Appendix 10** tells of Paradise in the 19th Century.

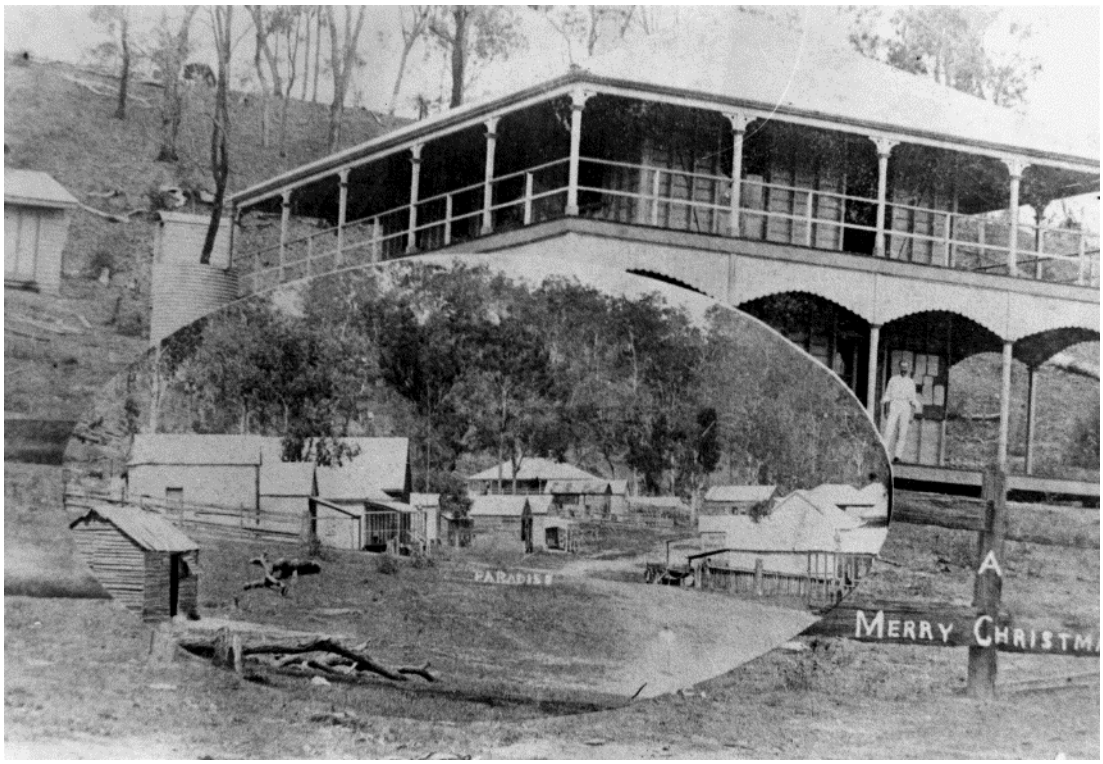

Figure 1.1 – Paradise, 1891. Image courtesy of John Oxley Library, State Library of Queensland Neg: 67269

Arrangement of this report

1.30 This report is organised as follows:

a. **Chapter 2** – Damage to the Dam: structural and stability issues

b. **Chapter 3** – Stability of the Dam: general principles and approach

c. **Chapter 4** – RCC aspects of the Dam

d. **Chapter 5** – Sliding stability of the Dam

e. **Chapter 6** – Downstream protection

f. **Chapter 7** – Governance.

Chapter 2 – Damage to the Dam: structural and stability issues

Introduction

2.1 Paradise Dam (**the Dam**) experienced flooding in December 2010 and January 2011 (**the 2011 event**) and spilled for a significant time. A flood of greater magnitude occurred in 2013 (**the 2013 event**). Both events damaged the Dam.

The 2011 event

2.2 In December 2010, heavy rain fell in the Dam's upper catchment causing extensive flooding in the Burnett River. That flooding generated, for the first time, spillway flows at the Dam.[1] The peak headwater level recorded on 29 December was EL 73.56 m (5.96 m above the spillway crest), with a peak discharge rate estimated at over 8,770 m^3/s.[2] That was followed by a smaller peak estimated to be at EL 71.28 m (3.68 m above the spillway crest) on 12 January 2011 when the discharge rate peaked at approximately 4,063 m^3/s.[3]

*Figure 2.1 – The primary spillway in flood during the 2011 event viewed from the right abutment looking across the river. (Exhibit 230, **DNR.006.3156**, .3212)*

2.3 The primary spillway continued to spill until September 2012, except for three days in November 2011 and two days in January 2012.[4] The prolonged flow precluded detailed inspection of the apron.

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1 **DNR.001.0036**, .0072.

2 Exhibit 230, **DNR.006.3156**, .3158. A nominated Annual Exceedance Probability of 1:25: Exhibit 230, **DNR.006.3156**, .3168.

3 Exhibit 230, **DNR.006.3156**, .3158. A nominated Annual Exceedance Probability of 1:8: Exhibit 230, **DNR.006.3156**, .3168.

4 **DNR.001.0036**, .0072.

Condition after 2011 event

2.4 In March 2012, some obviously damaged parts of the Dam were inspected, including the left bank spillway training wall,[5] the adjacent abutment and the right bank training wall adjacent to the 'central island'.[6]

*Figure 2.2 – Downstream of the primary spillway following the 2011 event. The outlet works are on the right and the 'central island' is in the centre of the image. (Exhibit 230, **DNR.006.3156**, .3219)*

2.5 Those areas were said to be of *'elevated'* concern with respect to dam stability or operation.[7] The nature of the damage also raised concerns about the stability of the left and right training walls.[8]

*Figure 2.3 – Erosion on the left bank following the 2011 event. (Exhibit 230, **DNR.006.3156**, .3192)*

5 A 4 m high wall keyed against the RCC apron slab: Exhibit 230, **DNR.006.3156**, .3169.
6 Exhibit 230, **DNR.006.3156**, .3158.
7 Exhibit 230, **DNR.006.3156**, .3169.
8 Exhibit 230, **DNR.006.3156**, .3158.

2.6 Areas of the banks were eroded.

2.7 Shotcrete below the left abutment had been scoured back past the downstream end of the training wall, with rock lodged between the wall and the shotcrete face.[9]

2.8 In September 2012, SunWater inspected the Dam. At that time, the outlet channel and lower portion of the primary spillway apron and end sill were underwater. After dewatering, in November 2012,[10] more extensive inspections were carried out.[11]

2.9 SunWater noted damage to the instrumentation, outlet and inlet buildings and the mini-hydro power plant, with erosion in the downstream river channel.[12] A protective layer of shotcrete, which had been used to treat the basalt pimple at the toe of the Dam, was displaced by flood flows.[13]

2.10 The fishway – a structure designed to enable the passage of fish past the dam in both the upstream and downstream directions[14] – was also affected. Large rocks and gravel were deposited in the fishway channels.[15]

2.11 SunWater's report detailed the damage to the apron and end sill as well as to the downstream face of the stepped primary spillway.

*Figure 2.4 – Damage to the basalt 'pimple' above the fishway channel (to the right of the image). Material was deposited in the fishway as a result of the flood. (Exhibit 230, **DNR.006.3156**, .3218)*

9 Exhibit 230, **DNR.006.3156**, .3169.
10 Exhibit 230, **DNR.006.3156**, .3158.
11 Exhibit 230, **DNR.006.3156**, .3159. The inspection covered the right bank outlet channel, the basalt pimple, the right bank downstream of the primary spillway, the downstream face of the primary spillway, the primary spillway apron and end sill, and the left bank training wall.
12 Exhibit 237, **SWA.512.001.0578**, .0585.
13 Exhibit 237, **SWA.512.001.0578**, .0585.
14 Exhibit 24, **GHD.002.0001**, .0348.
15 Exhibit 230, **DNR.006.3156**, .3172.

Downstream face of primary spillway

2.12 Most of the concrete on the downstream stepped face remained in reasonable condition. However, in places, concrete had been removed from the steps, possibly by log impact. There was some 'pitting' of the vertical face just below the horizontal step which may have been caused by cavitation.[16]

*Figure 2.5 – Primary spillway steps showing some damage after the 2011 Event. (Exhibit 230, **DNR.006.3156**, .3221)*

2.13 Many of the spillway contraction joints below the walkway for accessing the horizontal spillway drains were leaking. Some foundation drainage holes in the face of the lowest step were flowing. The leaking joints and flowing drains suggested that water was '*seeping through the dam foundation which in some areas is above the spillway apron level*'.[17] SunWater reported that the '*stability of the structure depends on the uplift pressure relief provided by the drains*'.[18]

Primary spillway apron and end sill

2.14 The 20 m wide apron was constructed at a lower level on the right side and then rose, by way of a sloping section, to a higher level on the left.[19]

2.15 The higher and lower level sections of the apron were in '*reasonable condition*'.[20]

2.16 Apart from a section about 1 m wide upstream of the sill, the sloping section of the apron also remained in reasonable condition. There was exposed reinforcement on

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16 Exhibit 230, **DNR.006.3156**, .3173.
17 Exhibit 230, **DNR.006.3156**, .3174.
18 Exhibit 230, **DNR.006.3156**, .3174.
19 Exhibit 230, **DNR.006.3156**, .3175. The slope is described in greater detail in Chapter 6. This resulted in an 'asymmetry' in the apron.
20 Exhibit 230, **DNR.006.3156**, .3175.

the apron itself. In one location a hole had been scoured to a depth of about 300 mm through the apron.[21]

Figure 2.6 – Part of the apron and sill following the 2011 event.
*Abrasion at the top of the end sill is visible. (Exhibit 230, **DNR.006.3156**, .3229)*

Figure 2.7 – The primary spillway apron end sill after the 2011 event. Abrasion of the end sill was most likely caused by 'rock drawn into the basin and then tumbled around in the roller flow'.
*(Exhibit 230, **DNR.006.3156**, .3178 and .3231)*

2.17 The concrete surfaces of all sections of the end sill, however, showed '*varying degrees of abrasion*'.[22] Reinforcing bars within the end sill were exposed in certain places. On the sloping section, abrasion occurred over '*most of the top of the sill*'.[23]

21 Exhibit 230, **DNR.006.3156**, .3175.
22 Exhibit 230, **DNR.006.3156**, .3176.
23 Exhibit 230, **DNR.006.3156**, .3176.

There was no evidence of undermining of the rock foundation on the downstream side of the sill.

2.18 Some concrete testing was carried out following the flood event.[24]

2.19 Rectification was proposed[25] but the continuing spillway flow precluded carrying out the works.[26] Construction had not progressed before another, larger flood supervened.

The 2013 event

2.20 On 26 January 2013, ex-Tropical Cyclone Oswald crossed the Burnett River catchment. Rainfall depths of up to 930 mm were recorded in the vicinity of the catchment.[27] The Dam soon experienced a major flood event. A smaller flood followed in March.[28]

2.21 The flood peaked at the Dam on 28 January. The primary spillway crest was overtopped by 8.65 m. The peak outflow of approximately 17,000 m^3/s represented an estimated 1 in 170 Annual Exceedance Probability (**AEP**) event.[29] The Dam's emergency action plan was activated for the first time.[30]

2.22 At the peak of the flow, the secondary spillway, designed to be overtopped only in an extreme flood event, was not engaged.

*Figure 2.8 – The Dam's primary spillway overflowing near the peak of the 2013 event. (Exhibit 237, **SWA.512.001.0578**, .0710)*

2.23 A second flood peak of 3.86 m above the crest occurred on 3 March 2013.

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24 Exhibit 230, **DNR.006.3156**, .3177.
25 Exhibit 230, **DNR.006.3156**, .3159 and .3178-80.
26 **DNR.020.012.3848**, .3923.
27 **DNR.001.0036**, .0075.
28 Together, these are referred to as the 2013 event.
29 **DNR.001.0036**, .0072. cf: Exhibit 27, **IGE.076.0001**, .0013 [61].
30 **DNR.001.0036**, .0075.

Flood Rank	Date	Peak Height	
		EL (m, AHD)	Above Crest (m)
1	Jan-13	76.25	8.65
2	Dec-10	73.56	5.96
3	Jan-11	71.83	4.23
4	Mar 13	71.457	3.857
5	Dec-10	70.887	3.287

Flood Rank	Date	Peak Height	
		EL (m, AHD)	Above Crest (m)
6	Mar-10	69.427	1.827
7	Sep-10	68.208	0.608
8	Feb-11	67.986	0.386
9	Feb-11	67.903	0.303
10	Dec-10	67.659	0.059

*Figure 2.9 – Flood peaks ranked in the period from the Dam's construction to March 2013. (**DNR.001.0036**, .0072-.0073)*

2.24 The flooding was well within the limits that the Dam had been designed to withstand.

2.25 A preliminary damage inspection was carried out in February 2013 while the Dam was still spilling.[31] A dam failure risk assessment was carried out in March.[32]

Damage to the Dam and downstream areas

2.26 The 2013 event damaged the Dam.

2.27 Around half of the top layer of the Roller Compacted Concrete (**RCC**) in the apron slab was damaged. Some of the lower layer also scoured away.[33]

2.28 The apron's sill had been almost completely removed. Large sections were washed downstream.[34]

2.29 There was also severe scouring of the riverbed immediately downstream of the primary spillway apron. The scour included a 13 m deep hole on the left side and a 3.5 m deep and 25 m long scour on the right side.[35] The scour developed along a fault zone in the rock.[36]

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31 **DNR.001.0036**, .0076-.0077.
32 **DNR.001.0036**, .0077.
33 **DNR.001.0036**, .0080.
34 Exhibit 237, **SWA.512.001.0578**, .0585-.0586. See also Figure 2.12.
35 **DNR.020.012.5265**, .5272.
36 **DNR.001.0036**, .0081. The fault zones are described in more detail in Chapter 6.

*Figure 2.10 – Scour on the left side, immediately downstream of the primary spillway apron. The hole was approximately 13 m deep. (**DNR.001.0036**, .0095)*

*Figure 2.11 – Scour on the right side, immediately downstream of the primary spillway apron. (**DNR.001.0036**, .0093)*

*Figure 2.12 – Image looking over the primary spillway apron from the right abutment. The approximate locations of the scour are identified. (Exhibit 7, **IGE.017.0001**, .0058)*

Works after the 2013 event

2.30 SunWater assessed the Dam's stability. Rectification of the damage ensued, in phases.

2.31 Phase 1 comprised emergency repairs, concentrating on the apron and scour holes.[37] Phase 2 involved interim repairs to reinforce the primary spillway apron. The end sill was reinstated and a heavily reinforced and anchored 'stepped scour protection wall' was placed on the upstream face of the scours in the riverbed.[38] Extensive repairs of the apron floor at each end of the primary spillway were carried out.[39] That phase was completed towards the end of 2013.[40]

37 **DNR.001.5574**, .5674. That phase was completed in 2013.
38 **DNR.001.5574**, .5674.
39 Exhibit 7, **IGE.017.0001**, .0021.
40 Exhibit 234, **IGE.033.0001**, .0005; cf. **DNR.001.0152**, .0156.

*Figure 2.13 – Phase 2 remediation works underway on the primary spillway apron following the 2013 event (Exhibit 7, **IGE.017.0001**, .0010)*

*Figure 2.14 – Localised erosion backfilled with concrete in the area immediately downstream of the apron. This image was taken in October 2013. (Exhibit 237, **SWA.512.001.0578**, .0609)*

2.32 The next phase involved a dam safety review and comprehensive risk assessment which were completed in 2016.[41] Substantial improvements have since been made, including strengthening the primary spillway at monoliths D and K with reinforced concrete buttresses. The apron has been further reinforced.[42]

41 Exhibit 234, **IGE.033.0001**, .0005.

42 **DNR.001.5574**, .5674. The remedial work to the apron is discussed in further detail in Chapter 6.

Structural and stability issues

2.33 Technical and engineering studies had informed SunWater's consideration of the Dam's structural integrity and necessary remediation. Those studies came from more than one source, including a Technical Review Panel (**TRP**) appointed in 2013, GHD Pty Ltd (**GHD**), engaged in 2017,[43] and a differently constituted TRP appointed in 2019.

2.34 Structural and stability issues were investigated in studies conducted between the dates mentioned in the Terms of Reference: 30 January 2013 to 30 November 2019. The Commission's work focuses on three such issues. All were identified in the Key Issues document:

a. Sliding stability, with reference mainly to GHD's Stability Assessment dated 25 November 2019, which concluded that the Dam did not meet Australian National Committee on Large Dams (**ANCOLD**) factors of safety for different flood scenarios.

b. Adequacy of protection immediately downstream of the Dam, principally:

   i. adequacy of the primary spillway apron's dimensions

   ii. capacity of the materials from which the primary spillway apron was constructed (and the way in which it was constructed) to resist the erosive force of water.

c. Strength of the foundations: TRP report number 1 dated October 2013[44] concerned Phases 2 and 3 of Remedial Works. It considered failure mechanisms including foundation shear strength parameters.[45] Later, exploratory boreholes showed areas of open contact between the RCC and bedrock at the foundation of parts of the Dam.[46]

2.35 The engineering and technical studies referred to in the Terms of Reference are explained in Chapter 1.

2.36 Another structural and stability issue, also identified in the Key Issues, was:

> *… the circumstances which contribute to a situation where there exists uncertainty among technical experts and engineers as to the Dam's structural integrity and stability …*

2.37 The discussion that follows gives an overview of the studies and how they came to be produced. The analyses they contain is discussed later when examining the structural and stability issues in detail.

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43 GHD had been engaged earlier, but late 2017 is the time from which Mr Willey said the work as to the stability of the structure commenced: **TRA.500.003.0001**, .0013 ln 30-37.

44 Exhibit 7, **IGE.017.0001**.

45 E.g. Exhibit 7, **IGE.017.0001**, .0025-.0026.

46 Section 2.4 of the second report of that TRP dated 23 September 2019 provides further explanation of this condition: Exhibit 12, **IGE.051.0001**, .0008.

Foundations

2.38 The strength of the Dam's foundations was raised in the opening remarks of Counsel Assisting[47] and as one of the Key Issues. However, no party pursued the matter, in evidence or submissions.

2.39 Sub-issues were raised about the foundations in reports of the TRPs:

a. The first TRP's Report No 1 in October 2013 recognised the potential for foundation failure through continuous and low angle defects in the Goodnight Beds and the basalt, and what had been identified as the Paradise and Apron Faults (both are discussed in Chapter 6).[48]

b. The second TRP Report No 2 dated 23 September 2019 mentioned that of the 12 exploratory boreholes in 2019, seven showed 'open contact' on various degrees of quality of rock ranging from good to poor and 'slightly weathered' to 'moderately weathered'.[49]

2.40 'Open contact' occurs where contact between the rock and concrete of the dam is not '*intimate or tight*'. [50] Some samples had '*tight contact*' but not '*good bonding*'.[51]

2.41 John Young, a geotechnical engineer who was a member of the 2019 TRP,[52] stated that such cases of open contact as were identified:[53]

> *… [are] not necessarily, ultimately, a problem, it just means that the strength will be a bit lower. It is not known how many occasions of open contact there are. This is something to be resolved in the ongoing investigations*.

2.42 More than one expert said that the foundation was not as significant a stability issue as others. James Willey, of GHD, an engineer with experience in dam design and safety,[54] considers the RCC to be one of the more critical potential failure modes.[55] Mr Young regards the state of the RCC lift joints as *'probably the more serious issue*'.[56] Peter Foster, a dam design and safety engineer,[57] thinks that weak RCC lift joints in the RCC dominate possible failure mechanisms, except for erosion of the weaker materials on the right abutment if the secondary spillway were to operate.[58]

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47 **TRA.500.001.0001**, .0025 ln 11-18.
48 Exhibit 7, **IGE.017.0001**, .0014-.0015, s 4.1.2.
49 Exhibit 12, **IGE.051.0001**, .0008, s 2.4.
50 Exhibit 76, **YOJ.001.001.0001**, .0007 [18.b.iii].
51 Exhibit 76, **YOJ.001.001.0001**, .0005 [19].
52 Exhibit 76, **YOJ.001.001.0001**, .0002 [6].
53 Exhibit 76, **YOJ.001.001.0001**, .0007 [20].
54 Exhibit 56, **WYJ.001.003.0001**, .0001 [1], .0002 [4].
55 **TRA.500.003.0001**, .0018 ln 25-30.
56 Exhibit 76, **YOJ.001.001.0001**, .0007 [22].
57 Exhibit 67, **PTF.001.001.0001**, .0002 [2].
58 Exhibit 67, **PTF.001.001.0001**, .0023 [62].

2.43 GHD, too, says that there are more likely failure mechanisms than through the foundations.[59] SunWater provided its latest assessment of combined risk as received from GHD. It suggests that the risk of sliding or overturning of monoliths through RCC lift layers contributes to more than half the risk, at 53.1%, followed by the combined risk of undermining the spillway monoliths from overflow scour, with loss of the apron or scouring at the toe to be the next likely.[60] The tables below show possible failure modes and contribution of risks:

Failure Mode ID	Failure Mode Description
PS1	Undermining of the Primary Spillway monoliths due to overflow scour
PS2	Undermining of the Primary Spillway monoliths due to scour at toe (Monoliths D & K)
PS3	Overturning/sliding of Primary Spillway monoliths through the RCC due to water loads
PS4	Failure of Primary Spillway monoliths during seismic event
SS1	Undermining of the Secondary Spillway monoliths due to loss of apron
SS2	Overturning/sliding of Secondary Spillway RCC Gravity Monoliths through the foundation due to flood loads
SS3	Piping through the foundation
SS4	Failure of Secondary Spillway monoliths during seismic event
SS5	Overturning/sliding of Secondary Spillway monoliths through the RCC due to water loads
SS6	Undermining and sliding of Monolith L due to erosion of shotcrete
LA1	Undermining of the Left Abutment monoliths due to scour at toe and natural abutment
LA3	Failure of Left Abutment monoliths through RCC due to increased water load

*Figure 2.15 – Summary of Failure Modes. (**SUN.026.0001**, .0020)*

59 Exhibit 301, **GHD.021.0001**, .0003.
60 **SUN.026.0001**, .0021 [78].

Combined Failure Mode Description	Failure Mode ID	Individual Risk Contribution %	Combined Risk Contribution %
Sliding/overturning of monoliths through RCC	PS3	48.5%	53.1%
	SS5	4.5%	
	LA3	0.1%	
Undermining of Spillway monoliths due to overflow scour, loss of apron, or scour at toe	PS1	35.3%	41.9%
	PS2	4.0%	
	SS1	2.6%	
Sliding/overturning of Secondary Spillway monoliths through the foundation	SS2	4.9%	4.9%
Piping through the foundation	SS3	0.1%	0.1%
All other risks (not listed above)	(various)	-	<0.1%

*Figure 2.16 – Failure Mode Risk Contribution. (**SUN.026.0001**, .0021)*

2.44 Hydro Tasmania submitted that no evidence proves that the contact between the dam wall and the foundation is inadequate.[61]

2.45 Mr Young said that the extent of open contact under the dam is unknown, leaving this as a live issue.[62] Mr Foster thinks that whether the foundations provide a failure mechanism is unknown. He did not have the numbers required for a reliable stability analysis.[63] No expert regarded the state of the foundations as a likely contributor to the risk of failure.

2.46 There is insufficient evidence to establish that the foundations do constitute a structural or stability problem. Accordingly, this issue is not considered further.

Downstream protection

2.47 The engineering and technical studies expressed opinions about damage to the primary spillway apron and erosion and scour of the rock immediately downstream.

2.48 The first TRP:

a. criticised the engineering geological model and the extent of the foundation mapping during design of the Dam[64]

b. identified an inadequacy in the apron's width to dissipate the energy of the water[65]

61 **HYT.008.0001**, .0161 [592].
62 Exhibit 76, **YOJ.001.001.0001**, .0008 [25].
63 Exhibit 67, **PTF.001.001.0001**, .0022 [59].
64 Exhibit 7, **IGE.017.0001**, .0014-.0016.

c. expressed a concern that the apron was constructed of RCC rather than heavily reinforced Conventional Concrete (**CVC**).[66]

2.49 The criticism of the foundation mapping was not repeated by Mr Young, who was a member of the second TRP.[67]

2.50 If severe enough, scouring could undercut the Dam, causing it to slide or overturn. Mr Young considered the extent of the scouring in 2013. He believes that, had it:[68]

> *… persisted for a significantly longer period, the left bank scour hole that occurred could well have worked its way back to the dam and caused some movement or sliding of it. … It is very worrying to see a 6 metre hole so close to a dam.*

2.51 Downstream protection issues are considered in Chapter 6.

Shear strength

2.52 Consideration of the shear strength of the RCC lift joints constituted much of the Commission's work. Experts have different views on the topic. The variety of opinions is an important reason for uncertainty about the Dam's stability.

2.53 In 2018, SunWater engaged GHD to undertake the preliminary design of improvement works options. GHD produced the two memoranda relating to shear strength (September 2019) (**the Shear Strength Memorandum**) and stability (November 2019) (**the Stability Memorandum**). As GHD's work would take months to complete, it provided its memoranda to keep SunWater apprised of its work and the tentative conclusions being reached.[69] A later memorandum dated 28 February 2020 updated the earlier Shear Strength Memorandum using test results from samples obtained during investigations in 2019 (**the 2020 Memorandum**).[70]

2.54 GHD considered, based on assumptions it adopted, that acceptable factors of safety were not met for certain load cases. For example, for a flood event equivalent to that which occurred in 2013, the factor of safety for sliding was approximately 1.11 compared with a required minimum factor of safety of 1.3 under the ANCOLD *Guidelines on Design Criteria for Concrete Gravity Dams*, September 2013 (**2013 ANCOLD Guidelines**).[71] For larger flood events, the factor of safety was less than

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65 Exhibit 7, **IGE.017.0001**, .0048; Exhibit 10, **IGE.020.0001**, .0021. A more recent assessment by GHD suggested that an apron of 20 m width would not contain the hydraulic jump and that the slab and anchor system was not adequate to resist uplift forces from the events considered. Exhibit 227, **MAL.002.0001**, .0005-.0006 [15]-[18]; Exhibit 228, **GHD.041.0001**, .0059.

66 Exhibit 7, **IGE.017.0001**, .0023.

67 Exhibit 76, **YOJ.001.001.0001**, .0010.

68 Exhibit 76, **YOJ.001.001.0001,** .0009-.0010 [29].

69 Exhibit 56, **WYJ.001.003.0001**, .0008 [27].

70 Exhibit 61, **SUN.009.004.0037**.

71 **TRA.500.003.0001**, .0065 ln 28-37. The 2013 ANCOLD Guidelines are Exhibit 35, **ACD.001.0001.**

1,[72] meaning that the Dam was considered unstable in that event. This assessment rated the Dam's stability as significantly less than had been assumed in its design.

2.55 GHD's assessment had relied upon shear strength testing – a means by which lift joint shear strength can be determined. There is disagreement about whether that testing yielded reliable indications of the state of the lifts. Some experts consider that multiple stage testing had degraded the samples, causing them to indicate a lower residual friction angle than the lift joints would have *in situ*.

2.56 The report of Stephen Tatro and Jim Hinds, dated 25 November 2019, *Paradise Dam - Shear Strength Evaluation Comments* concluded that:[73]

> *… 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 strength provides reliable values of shear strength .... However, additional testing as recommended may narrow the range of variability [in the residual friction angle] and improve assurance in the test results.*

2.57 The authors were, however, critical of the multiple stage testing done on the same sample, regarding that as '*very problematic*'. The significance of this reservation is that, if the strength of the lift joints is frictional only, and characterised by a friction angle of 39.3°, then the stability analyses conducted by GHD (and endorsed by the second TRP) indicate '*[t]he stability of the primary spillway monoliths is considered to have been marginal at the peak of the 2013 flood event*'.[74]

72 Exhibit 16, **GHD.005.0001**, .0015.
73 Exhibit 14, **IGE.028.0001**, .0002.
74 Exhibit 16, **GHD.005.0001**, .0019.

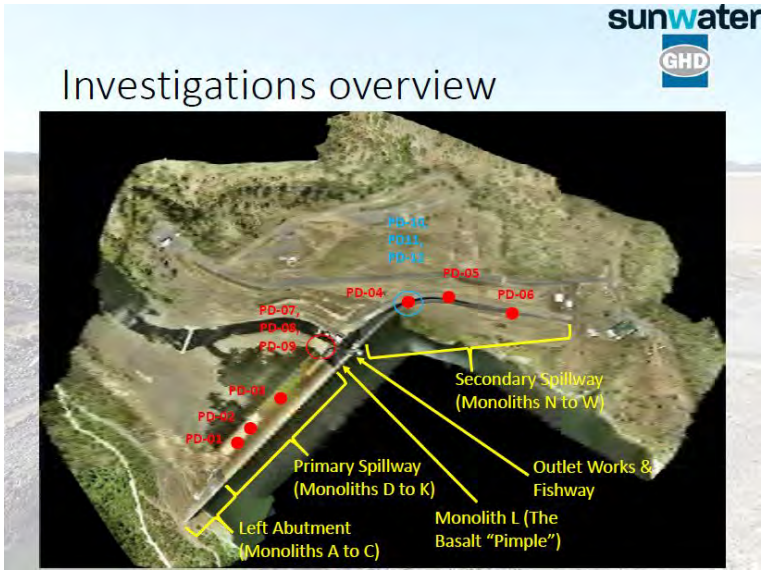

*Figure 2.17 – An image from GHD's 27-28 August 2019 TRP presentation showing the location of the various monoliths. (**ALC.002.001.1085**, .1089)*

2.58 GHD considered possible failure mechanisms. It reported:[75]

> *Overturning/sliding of the primary spillway monoliths through the RCC is the highest contributor to the risk as a result of the revised RCC lift joint shear strength adopted for this phase of the project. Undermining of the Primary Spillway Monoliths D to K due to overflow scour is the next highest contributor. These two failure modes account for over 85% of the total risk.*

75 **IGE.036.0001**, .0059. The sliding stability of the Dam is considered in the following 3 Chapters.

Chapter 3 – Stability of the Dam: general principles and approach

Introduction

3.1 This Chapter considers the factors that inform an assessment of the stability of gravity dams. These include the potential failure modes that need to be considered by designers of gravity dams, the methods commonly adopted to analyse whether failure is likely in any of those modes (i.e. an analysis of the stability of a dam), and the criteria that inform decisions whether adequate stability has been achieved. Particular treatment is given to sliding stability because of its prominence as an issue in the engineering and technical studies identified in the preceding Chapter.

Gravity dams

3.2 A dam is a barrier constructed in a river valley to block or control the flow of the river or stream and to impound water on its upstream side. According to the US Bureau of Reclamation (**USBR**),[1] a gravity dam is '*a solid … structure so designed and shaped that its weight is sufficient to ensure stability against the effects of all imposed forces*'. Gravity dams usually consist of parts or sections, some of which are designed for overflow (known as spillways) and others which are not.

3.3 Paradise Dam (**the Dam**) is a mass concrete gravity dam. Such dams can be built in a number of ways. Some older gravity dams are constructed from masonry. Modern gravity dams are often built from solid Conventional Concrete (**CVC**). In more recent times, Roller Compacted Concrete (**RCC**) has been adopted as an alternative construction material.

3.4 Concrete is the world's most widely-used building material. It consists of aggregates, cement and water that harden over time, and it is known as a composite material. Portland cement is the type of cement most commonly used in concrete. It is a combination of common natural materials such as limestone and clay manufactured with a chemical combination of calcium, silicon, aluminium, iron and other components. When heated at high temperatures, these components form a rock-like substance that is subsequently ground into a fine powder known as cement.[2]

3.5 Concrete is favoured as a construction material for its versatility, as it has the ability to resist the erosive, corrosive and abrasive forces of wind and water, and to withstand high temperatures. These qualities make it suitable for use in building large infrastructure, such as dams.

3.6 Dams are often massive structures, requiring large volumes of concrete, and involving substantial cost. Developments have occurred to speed up and make less

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1 US Bureau of Reclamation, *Design of Gravity Dams - Design Manual for Concrete Gravity Dams*, (1976), accessed on 19 April 2020, <https://www.usbr.gov/tsc/techreferences/mands/mands-pdfs/GravityDams.pdf>.

2 Portland Cement Association, *How Cement is Made*, accessed 17 April 2020, <https://www.cement.org/cement-concrete-applications/how-cement-is-made>.

costly the construction of concrete gravity dams. One such development has been the use of RCC, which is a composite construction material characterised by a 'no-slump' consistency in its unhardened state.[3] Freshly mixed concrete will have a no-slump consistency when it has a measured slump of less than 6 mm.[4] RCC dams also have the advantage of usually requiring less cement than CVC dams. The advantages of RCC and its technical specifications and methods of placement are discussed in more detail in Chapter 4.

3.7 The design of gravity dams, whether constructed of CVC or RCC, usually follows a similar process. There are differences, however, in construction methods, concrete mix design and the nature of appurtenant structures.

3.8 In addition to their classification based on the type of construction material, gravity dams may also be classified according to their shape in plan. They may be either straight or curved, depending upon the alignment of their axes. The principal difference between these two dam types is the method of analysis adopted when assessing stability.

3.9 For the purpose of stability analysis, the Dam is a straight gravity dam even though the secondary spillway section on the right abutment contains a curved section. The main and tertiary spillways are straight. The secondary spillway consists of two straight sections joined by a relatively short curved section. Only the case of straight gravity dams is considered here, given the Dam's type.

3.10 Straight gravity dams often consist of separate monoliths constructed across the river valley and separated by vertical joints. These monoliths may be constructed with shear keys between adjacent monoliths. These keys allow the transfer of horizontal loads between adjoining monoliths. The Dam was constructed without shear keys between its monoliths.

Structural design of gravity dams

3.11 This section describes the major design requirements of straight gravity dams, the forces such dams must resist, as well as the available engineering design checks and methods to ensure that adequate resistance against these forces can be mobilised.

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3 Concrete 'slump' is measured in a test designed to assess the consistency (i.e. the workability) of fresh concrete before it sets. The test is carried out using a metal mould in the shape of a truncated cone (a conical frustum) known as a slump cone that is open at both ends and has attached handles. This inverted cone is filled with fresh concrete in stages until the fresh concrete is flush with the top of the mould. The mould is then carefully lifted vertically upwards, so as not to disturb the cone of concrete but allowing it to slump (or subside) under the action of gravity. The 'slump' of the concrete is determined by measuring the distance from the top of the slumped concrete to the level of the top of the slump cone.

4 American Concrete Institute, Cement and Concrete Terminology: ACI 116R-00, (2000) ACI Committee 116, accessed on 15 April 2020 <http://dl.mycivil.ir/dozanani/ACI/ACI%20116R-00%20Cement%20and%20Concrete%20Terminology_MyCivil.ir.pdf>, p 116R-17.

3.12 The focus for present purposes is on fundamental principles. The detailed engineering calculations normally carried out by dam designers to be satisfied that a dam will remain stable under normal working conditions, as well as the most adverse loading conditions that a dam may experience are dealt with later.

Design considerations

3.13 The design of a gravity dam, and analysis undertaken for that purpose, generally include an assessment of:

a. the adequacy of the foundation on which the dam will sit

b. the structural competency of the CVC or RCC to be used in the construction of the dam

c. the possibility of leakage of water through, under and around the dam

d. the likely behaviour of the dam when its spillway is overtopped during flood flows, during earthquakes and under normal operating conditions.

3.14 In terms of a dam's structural stability, it is usual, and engineering good practice, for a designer to be satisfied of the following more detailed matters:

a. Rock formations at the dam site should, or will after treatment, be capable of carrying the loads transmitted by the dam.

b. The dam should be thoroughly bonded to the foundation rock in its contact with the river bed and abutments.

c. The CVC or RCC in the dam should be, in general, relatively homogeneous, uniformly elastic in all directions, and strong enough to carry the applied loads with stresses remaining below acceptable limits (known as the 'elastic limits'). Possible exceptions may be dams that are designed and built with different RCC mixes in different zones of the dam.

d. Contraction joints that are keyed and grouted may be considered as helping to create a single monolithic structure, and loads may be transferred horizontally to adjacent blocks by both bending and shear. If the joints are keyed but not grouted, loads may be transferred horizontally to adjacent blocks by shear across the keys. Where vertical joints between individual monoliths are neither keyed nor grouted (as is the case for the Dam), the entire load on each separate monolith of the dam will be transferred vertically to the foundation. If joints are grouted, that should occur before the reservoir loads are applied so that the structure acts monolithically upon first filling.

e. The seepage of water leaking through, underneath and around the dam should be limited and the water pressures in the foundation beneath the dam should normally be controlled by a combination of measures. These may include grouting the foundation rocks to fill voids, fissures and cracks and the provision

of drainage systems, possibly including a gallery inside the dam, and/or a membrane or impermeable lining on the upstream face of the dam.

3.15 The Dam was constructed using an impermeable ('Carpi') membrane attached to its vertical upstream face. The Dam does not contain an internal gallery. Work in grouting of the foundation and in providing drainage measures was directed to control water leakage and to limit uplift pressures underneath the Dam.

3.16 The Dam was constructed as a series of separate blocks or monoliths across the valley without effective shear keys at the interface between adjacent monoliths. One of the major reasons for adopting a series of separate monoliths is to exercise some control over the location of shrinkage cracking which is almost inevitable in CVC and RCC structures. Shrinkage generally occurs as the cement in the CVC or RCC hydrates, and so building the dam as a series of separate monoliths allows for contraction of the concrete material to be accommodated *at the ends* of each monolith, rather than by cracking of the material *within* the monolith.

3.17 Given the absence of shear keys between monoliths at the Dam, for design purposes it is reasonable to assume that the entire load on each monolith would have to be transferred vertically to the foundation. It is instructive then to consider these loads and how they are resisted.

Loads on a gravity dam

3.18 During the design of a gravity dam, it is engineering good practice to identify the loads that are likely to act on the dam wall and its monoliths under a variety of conditions. These conditions include partial and full impoundment, as well as those experienced during different types of flood flows over the spillway.

3.19 Figure 3.1 below shows a schematic representation of a vertical cross-section through a typical dam monolith on a plane perpendicular to the axis of the dam. In this case, the upstream face of the dam is vertical, as with the Dam. Also shown in the figure are the static forces assumed to act on the monolith. Not shown in the figure are any internal water pressures within the monolith (e.g. acting on lift joints), nor the seismic inertial forces that would be exerted dynamically on a dam for the brief duration of an earthquake, such as would result from the ground shaking. The latter may have both horizontal and vertical force components, some of which may tend to destabilise the dam. Earthquakes are normally considered to be extreme loading events when analysing dam stability.

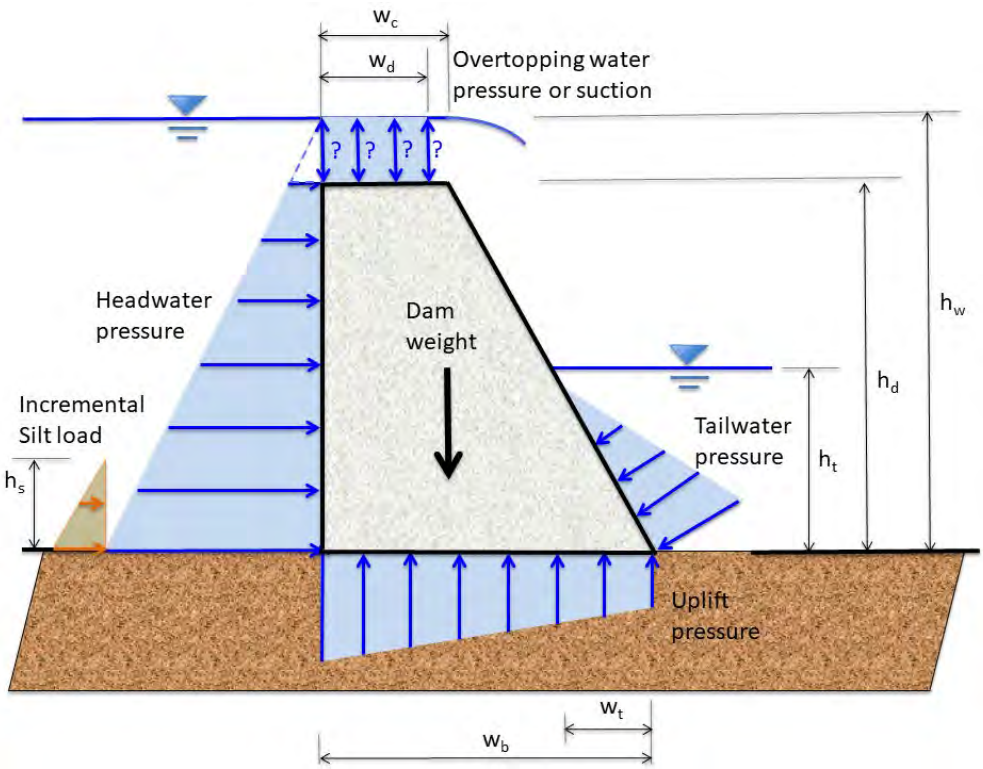

*Figure 3.1. Schematic cross-section through a dam spillway monolith showing the forces assumed to act on the dam. (Exhibit 62. **PDI.067.0001**)*

3.20 Figure 3.1 depicts the general case of a dam being overtopped during a flood flow event. A number of forces and pressure distributions acting on the dam monolith are shown schematically, including:

a. A self-weight force that acts on the dam due to its own mass in the presence of the earth's gravitational field. This force is assumed to act vertically downwards, being transmitted to the dam foundation. It is usually a major source of the stability of the dam, as explained in more detail below.

b. The horizontal water force in the headwater, tending to destabilise the dam by its exertion in a downstream direction. It is assumed to be the result of the hydrostatic[5] water pressures acting on the vertical upstream face of the dam. This water pressure is assumed to vary linearly with depth below the headwater free surface.

c. A smaller horizontal destabilising force that may also occur near the base of the dam due to the accumulation of silt at the bottom of the impoundment. It provides a horizontal pressure distribution, the magnitude of which is also assumed to increase linearly with depth below the top of the sediment.

d. The tailwater that exerts normal pressures on the downstream face of the dam. If the downstream face is inclined to the vertical, then this normal pressure distribution will have both horizontal and vertical force components that act on

5 Fluid pressure acts with the same magnitude in all directions. A hydrostatic pressure is a fluid pressure the magnitude of which varies linearly with depth below a free or phreatic surface.

the downstream face. It is usual for design purposes to assume that the tailwater force is also hydrostatic. This may not correspond to the full depth of the tailwater, because of turbulence effects in the tailwater region. The horizontal and vertical components of the tailwater force generally display the following actions:

i. The horizontal force component generated by the tailwater tends to push the dam upstream, i.e. it acts in the opposite direction to the headwater force, tending to stabilise the dam, helping to resist those forces that act on the dam in the downstream direction.

ii. The downward force component generated by the water pressures acting on the downstream face of the dam complements the self-weight force of the dam and is also transmitted downwards to the dam's foundation.

e. The water pressures or suctions to which the crest of the dam spillway may be subjected during flooding events that cause the dam to be overtopped. Generally, during lower flood flows that overtop the crest, the pressures of the water may result overall in a downward component of force on the crest of the dam. However, at higher flow rates, studies have indicated that the water may exert suction on the crest of the dam, resulting overall in an upward component of force applied to the dam crest. Suctions not only contribute to an upward component of force counteracting the weight of the dam; they also present the risk of local damage to the concrete crest of the dam due to cavitation.

f. Water under pressure in the voids, fissures, cracks and joints in the rock beneath the dam that will result in an uplift component of vertical force acting on the base of the dam. In addition, if water under pressure is able to enter the dam, for instance via cracks, porous concrete or horizontal lift joints in or through the dam structure, then it may also exert an uplifting force within the dam itself and in particular on the section of the dam above the horizontal crack or joint in question, tending to destabilise that section of the dam monolith. Those uplift forces would tend to oppose the weight of the section of a dam above that lift joint. The magnitude of water pressures within a dam will depend on the effectiveness of the water sealing and drainage measures employed within the dam.

3.21 The horizontal headwater force plus the silt loading is usually much greater than the horizontal tailwater force. It is the resultant of these three forces that provides the overall destabilising action, tending to push the dam downstream and also tending to overturn it, pivoting about the toe region of the dam.

Loading cases

3.22 It is usual when analysing the stability of a dam to consider the combination of loads that may act upon it under several different sets of conditions. This provides a number of distinct 'cases' that dam designers should consider. In 2013, the Australian National Committee on Large Dams (**ANCOLD**) published *Guidelines on Design Criteria for Concrete Gravity Dams* (**2013 ANCOLD Guidelines**) which state

that '*[t]he dam and the foundation should be assessed for Usual, Unusual and Extreme load cases and combinations*'.[6] The '*Usual*' load condition is defined as loading that is likely to occur a number of times during the design life of the dam. This would include, for example, the loads corresponding to a full reservoir.

3.23 As the name suggests, '*Unusual*' loads are not likely to occur regularly during the life of the dam. If they do occur, the dam can be expected to continue to behave in a satisfactory manner, perhaps with minor structural damage, provided the design and construction have been adequate. Unusual loading cases include infrequent and very infrequent flood conditions.

3.24 *'Extreme'* loads result from circumstances near the limit of possibility and are only considered because of the critical and hazardous nature of a dam. They include earthquake loading and loadings corresponding to the probable maximum flood (**PMF**) level.

3.25 The purpose of examining Unusual and Extreme loading cases is to ensure the dam will have a low likelihood of failure (failure here means uncontrolled release of the reservoir), even though damage may occur to the dam itself or its appurtenant works.

Acceptance criteria

3.26 When calculating the stability of the dam, it is usual for 'acceptance criteria' to be defined and applied to each category of loading. This is discussed in more detail below. In general, bigger safety margins are required for Usual load combinations, whereas smaller margins may be considered acceptable for Unusual and Extreme load combinations. Guidelines for dam design, including the 2013 ANCOLD Guidelines, suggest the adoption of different safety margins for events that are more likely and may be sustained for long periods. This is distinguished from those that are infrequent or rare and which may subsist for shorter periods.

Potential failure modes

3.27 The potential failure modes usually considered in the design of a CVC or RCC gravity dam include:

a. Sliding of the dam downstream at the interface between the dam and its foundation material. This interface is often assumed to be horizontal or slightly inclined to the horizontal, i.e. sub-horizontal.

b. Sliding of the dam downstream along a horizontal plane through the dam wall. This case is particularly relevant to RCC dams in which the lift joints, being the interfaces between successively placed horizontal layers of RCC, are frequent and represent potential planes of shearing and tensile weakness.

c. Sliding of the dam downstream along planes of weakness through the foundation rock mass.

6 Exhibit 35, **ACD.001.0001**, .0022.

d. Overturning of the dam, usually about its downstream toe, under the action of the destabilising forces. The likelihood of overturning could increase significantly if the toe of the dam were to be undermined by flood flows passing over the spillway causing erosion of the foundation rocks.

3.28 Each of these potential failure modes is usually checked in detail during the design phase of a gravity dam. Such checking normally requires engineering stress and stability analyses and calculations. In each case, a number of important assumptions need to be made about what combination of loads is most likely to act upon the dam and under which circumstances. These assumptions are considered in further detail below. They can have an important bearing on any assessment of the dam's structural stability.

3.29 For the first to third failure modes listed above in sub-paragraphs 3.27(a) to (c), it is usual to treat the dam itself as a rigid body and then to resolve all forces acting on the dam. The destabilising shear force acting on the sliding plane is then compared to the shear strength that can be mobilised along that plane.

3.30 For the analysis of overturning (potential failure mode 3.27(d) above), all forces acting on the dam and their lines of action are considered. If the resultant of all these forces is calculated to act through the middle one-third of the base of the dam, then it is usually considered stable with respect to rotation (or overturning) about the toe of the dam. Various possibilities may be considered when carrying out this analysis, including the dam under its normal operating conditions, but also allowing the possibility of a crack forming at the base of the dam under one or more of its monoliths, and that crack being subjected to significant water pressure.

3.31 The assessment of structural stability also requires an appropriate method for predicting the stresses likely to be experienced within the dam and also at its base, especially if the effects of cracking are to be considered. One of the major concerns is whether tensile stresses are likely to occur, particularly parallel to the upstream face of the dam or at the interface between the dam and its rock foundation. If such tensile stresses exceed the tensile strength of the dam material or any joints or interfaces it may contain, then cracking may occur, typically forming horizontal cracks. If cracking does occur, water under pressure may enter the crack, destabilising the dam and increasing the likelihood of either a sliding or overturning failure.

3.32 Overturning and sliding failure become more likely if the rock mass at the toe of the dam is eroded by flood flows over the dam. There is an obvious requirement to protect the area immediately downstream of the toe of the dam from the erosive forces of water flowing over the spillway. Dam designers therefore consider the inclusion of an apron or other structure that can resist the water forces and dissipate the energy of the water flowing over the dam and down the spillway.

Methods of stability analysis

3.33 Traditionally, the Gravity Method of Stress and Stability Analysis (**Gravity Method**) is used for preliminary studies of gravity dams, depending upon the phase of design and the information required. The Gravity Method is also used for final designs of straight gravity dams in which the transverse contraction joints between adjacent monoliths are neither keyed nor grouted. In recent times, more sophisticated numerical modelling of gravity dams, using the Finite Element Method, is often conducted to analyse both the overall stability of a dam and the stresses within it.

3.34 The Gravity Method also provides an approximate means for determination of stresses in the cross section of a gravity dam. It is applicable to the general case of a gravity dam with a vertical upstream face and with a constant downstream slope and to situations where there is a variable slope on either or both faces. Details of the stress calculations can be found in various authoritative reference sources, for example, in USBR publications.[7]

Shear sliding

3.35 In this section, the potential for shear sliding failure of a dam is considered. It is often the most critical mode of failure for RCC dams because of the number of lifts and, therefore, the greater number of possible failure planes than a CVC dam would typically have. This is, therefore, the failure mode that often governs the design for stability of an RCC gravity dam.

3.36 In conducting the analysis of potential sliding on a horizontal plane, either through the dam, at its base, or through the supporting foundation material, it is usual to consider the dam as a rigid body. The overall shear force acting on the sliding plane is then calculated and compared to the estimated shear strength of that shearing plane. It is standard practice to consider a unit thickness of the dam in the longitudinal direction, i.e. in the direction perpendicular to the plane of the analysis, as shown in Figure 3.1 above.

3.37 The total shear force acting on the horizontal shear interface is the resultant of all horizontal forces acting on a dam monolith above the level of the shearing plane. For a potential shearing plane located at or near the base of a dam, the horizontal water force in the headwater (tending to push the dam downstream) is assumed to be the resultant of the hydrostatic water pressures acting on the vertical upstream face of a dam. A smaller horizontal destabilising force may also occur due to the accumulation of silt at the bottom of the impoundment. The latter provides a horizontal pressure distribution, the magnitude of which increases linearly with depth below the top of the sediment.

3.38 A horizontal force also acts in the tailwater, but it tends to push the dam upstream, i.e. it acts in the opposite direction to the headwater force. The tailwater force is also normally assumed to be hydrostatic but, as previously explained, perhaps not

[7] See, for example, note 1 above and Exhibit 229, **BOR.001.0001**, also available online at: US Bureau of Reclamation, *Design of Small Dams* (3rd edition) accessed 19 April 2020 <https://www.usbr.gov/tsc/techreferences/mands/mands-pdfs/SmallDams.pdf>.

corresponding to the full depth of the tailwater, because of turbulence in the tailwater region.

3.39 As is referred to above, the horizontal headwater force and the horizontal silt loading are usually much greater than the horizontal tailwater force. It is the resultant of these three forces that provides the destabilising action, tending to push the dam downstream by sliding on the shearing plane.

3.40 It is usually assumed that the shear strength of the shearing interface or potential shearing plane through or under a dam is given by a strength criterion known as the Mohr-Coulomb criterion. This states that the maximum shearing resistance may have two components: (1) a cohesive strength component that is independent of any normal force acting across the plane of shearing; and (2) a frictional component, the magnitude of which is directly proportional to the normal force acting on the shearing plane.

3.41 In other words, this strength envelope assumes a straight line (or linear) relationship between the shear strength of the sliding plane and the normal force acting across it. The larger the normal force the larger the shearing resistance. In the context of a gravity dam, the larger the mass of the dam, generally the greater is the shearing resistance that can be mobilised. In reality, the relationship between the shear strength and the normal force on the shearing plane may actually be curved rather than straight, but a straight line is often assumed for mathematical convenience.

3.42 For the friction component of shear strength, the constant of proportionality is the friction coefficient that is conveniently expressed as the tangent of the so-called friction angle, ϕ, i.e. $\tan\phi$. The cohesive component of the shearing resistance is defined by the cohesion parameter, c, expressed in units of stress. When the cohesion c is multiplied by the area of the shearing plane the cohesive component of the overall shear strength of that plane is calculated. That shear strength is expressed in units of force (or, strictly speaking, force per unit length when a vertical slice of a monolith of unit thickness is considered).

3.43 In summary, the maximum shearing resistance that can be developed along a sliding plane therefore has two parts: one due to cohesion and the other due to friction. Friction will always be present, but the presence of cohesion depends on the particular circumstances of the sliding interface. The concepts of cohesion and friction are considered in more detail below.

Shear strength concepts – cohesion and friction

3.44 One part of shearing resistance arises from any cohesive strength that may be mobilised along the shearing plane, independent of the normal force acting on that plane. In practice, such cohesion (or, more strictly speaking, the cohesion intercept of the Mohr-Coulomb shear strength envelope) often results from the bonding of the particles either side of the potential shearing surface. It may also result from the strength of the larger aggregate particles, if some of those particles (e.g. those intersecting the potential shearing plane) split under the imposed shear force to allow the formation of a distinct shearing plane or thin shear zone through the dam. Sometimes, for the purpose of analysis, the sliding interface may be assigned an

'apparent' cohesion, which can correspond to true physical phenomena other than particle bonding. It may also be an artefact of assuming a linear shear strength envelope when that envelope is actually curved. An example of a physical phenomenon resulting in apparent cohesion is capillary action that may arise due to the presence of a mixture of water and air in any voids in the CVC or RCC or along lift joints. Surface tension in the water in the voids may be a source of apparent cohesion.

3.45 The second part of the shearing resistance arises because of friction generated as a dam tends to slide along the shearing plane. The magnitude of the frictional component of shear strength is highly dependent upon the angle of friction of the interface (or the friction coefficient). It also depends upon the effective normal force assumed to act across the potential sliding plane.

3.46 The value of the effective friction angle (or mobilised friction angle) and therefore the effective friction coefficient may vary during shearing and will depend on a number of factors. For example, for a sliding interface without cohesion, the effective friction coefficient may be thought of as the ratio of the shear to normal force acting on the potential sliding plane as the shearing displacement increases. This ratio may rise to a maximum or peak value as shearing progresses and in some cases may reduce after that peak with further shear displacement. For concrete and concrete-rock interfaces, the factors that influence the effective friction angle and friction coefficient include (predominantly) the mineralogy of the rock and the larger aggregate in the concrete. It may also depend upon the frictional properties of the cement paste and whether any dilation (i.e. opening) of the shearing interface is required to permit shear sliding to take place. Displacement of relatively strong aggregate particles in close proximity to the shearing plane, corresponding to either dilatational displacement or rolling or both, may be necessary for the shear sliding to occur. Initially this may contribute to a small increase in the value of the effective friction angle or friction coefficient. However, even for a dilatant interface, with sufficient shear displacement the effective friction angle ultimately approaches a limiting value. This value may be less than the value mobilised at smaller shear displacements. The friction angle at larger shear displacements is generally known as the 'residual friction angle'.

3.47 For a gravity dam, the effective normal force acting across a potential shearing plane has several components, including:

a. the downward acting weight of the section of a dam monolith above the shearing plane

b. the weight of any water spilling over the top of a dam also acting downwards or, in some cases, the suction generated by this overflow water producing an uplift force component on the crest of the dam

c. the downward force component generated by the water pressures acting on the downstream face of a dam

d. water under pressure beneath a dam in drains or voids, cracks or fissures in the rock foundation or within cracks and joints within the dam monolith. This pressure will tend to produce an uplift or buoyancy force, which will act in the opposite direction to the weight of the dam.

3.48 It is the resultant of these forces that provides the effective normal force on the sliding interface. In turn, it is this resultant force that is multiplied by the friction coefficient ($\tan\phi$) to calculate the frictional component of the interface shearing resistance.

3.49 In summary, the shearing resistance of any potential sliding plane under or through a dam is generally considered to have two potential components – one due to cohesion and one due to friction. Friction will always be present, but the presence of a cohesive component depends very much upon the condition of the sliding interface. This is considered in more detail later in this report, particularly in relation to RCC and its use in the Dam.

Factor of Safety for shear sliding

3.50 The ratio of the computed shear strength of the interface to the calculated shear force assumed to act on the interface provides the factor of safety against sliding. The computed factor of safety is then compared with acceptance criteria to determine whether a dam's design is adequate with regard to sliding. Acceptance criteria are the minimum acceptable values of the factor of safety. Different values of the acceptance criteria are normally adopted for different load combinations, viz. *Usual*, *Unusual* and *Extreme*. These limiting values are always greater than 1.

3.51 The factor of safety approach allows for all the uncertainties in all the physical quantities that may have an influence on the safety of the dam. These include uncertainties about the strength parameters applicable to the potential sliding plane and those of the actual loadings that the dam may experience. The calculated values of the factor of safety are required to be greater than the specified acceptance values.

3.52 Various dam authorities and organisations, including ANCOLD, have published recommendations regarding what they believe constitute acceptable values of the factor of safety against shear sliding. As indicated above, different values are normally acceptable under different loading conditions and circumstances, such as the dam being at full reservoir level and flow overtopping the dam spillway corresponding to various recurrence intervals of floods or expected annual probabilities of their occurrence. It is generally accepted that higher values of the factor of safety are appropriate for lower flood frequencies and so-called normal operating conditions, i.e. for more frequent and usually more sustained loading conditions. Lower values of the factor of safety may be acceptable for less frequent and more extreme floods or seismic events or loading events of shorter duration.

Probabilistic analysis of dam failure

3.53 It is customary in dam safety guidelines to define minimum factors of safety that should be achieved in stability calculations. These calculations are performed using a deterministic model of the dam, assuming specific numerical values for the loads and strength parameters. In such cases the calculated factor of safety is a measure of the reserve strength of the dam with respect to whichever failure mechanism is being examined.

3.54 An alternative approach is probabilistic analysis. This is most often conducted when assessing the state of an existing dam and regularly forms part of a more comprehensive risk analysis. That approach considers not only the probability of dam failure but also, importantly, its consequences.

3.55 In a probabilistic analysis, any uncertainties in specifying the loads acting on the dam and the values of the relevant strength parameters critical to resisting those loads may be included. Doing so allows designers to predict the probability of failure occurring by the mode of failure being investigated.

3.56 A probabilistic analysis of this type requires more information than a deterministic one. For example, the likely variations in the strength parameters, such as cohesion and friction, need to be identified. This usually requires an appropriate selection of the statistical properties of the strength parameters and the loads. The relevant statistics include the mean (or average) value of that parameter, its standard deviation (i.e. a measure of how spread out the values of the parameter may be) and what is known as a 'probability density function'. That third component simply specifies the probability that a parameter value (e.g. cohesion) will fall within a given range of values, as opposed to taking on a unique value.

3.57 It can be seen that, as in the deterministic approach, the successful use of the probabilistic method is highly dependent upon the inputs. In the case of a probabilistic analysis, this means reliable values of the input parameters are required, i.e. the statistical properties of the loads and the strength parameters.

3.58 Once the statistical properties of the loads and strength parameters have been assessed and defined, the dam's probability of failure in a particular mode is calculated as follows:

a. A large number (typically up to 250,000) of sets of loading and strength parameters are 'sampled' at random, but within the bounds specified by the relevant probability density function, to examine a large number of possible scenarios involving different combinations of values of the input parameters (loads and strength parameters).

b. Stability analyses are performed for each one of this large number of scenarios so that a factor of safety is calculated for each scenario.

c. Statistical analyses are performed on the results of these stability analyses to determine the probability of failure. For example, if 10,000 different cases are considered and the statistics reveal that the factor of safety was 1 or less in 10

out of the 10,000 cases considered, then the probability of failure would be 10/10,000 or 0.001.

3.59 This method is often referred to as the 'Monte Carlo simulation technique'. It involves '*sampling at random to simulate artificially a large number of experiments and to observe the results'*.[8]

3.60 This technique is one of several often used by owners of existing dams to assess the current condition of the dam and to make an assessment of the probability and consequences of its failure, i.e. the risk of failure (see for example ANCOLD (2003)).[9] The term 'risk' is often understood as the product of the probability of something happening and the resulting cost or the consequences.

3.61 The designers of the Dam adopted the more conventional deterministic method of stability analysis and made estimates of the deterministic factor of safety with respect to sliding failure on horizontal planes through and beneath the dam. They also made checks on the likelihood of overturning failure and whether dam sections were likely to crack under the imposed loading. For this reason, the focus below is on the deterministic approach to stability analysis and the relevant guidelines that were (or are currently) available to assist engineers in making key decisions with respect to the stability and safety of the Dam.

Design guidelines

3.62 Dam construction and the practice of dam engineering and design have been carried out for millenia. The experience accumulated over this time has allowed dam engineering to evolve to the point where authorities and organisations in many countries have developed and published guidelines for dam design. For example, in the United States of America several authorities have produced documents to assist in the design of dams. These include the USBR and the US Army Corps of Engineers (**USACE**).

3.63 In Australia, ANCOLD has published a series of guidelines covering a variety of topics in dam engineering over many years. Some of these are discussed further in following sections of this report. No statute requires that they be complied with, nor have they taken the form of codes of practice. They are guides to good practice rather than a mandatory set of rules that must be followed. The 2013 ANCOLD Guidelines state:[10]

> *The purpose of these Guidelines is to provide the designer with the fundamental criteria for input to design of, and design acceptance criteria for, concrete gravity dams that are founded on rock.*

8 Melchers, R E, *Structural reliability analysis and prediction* (Wiley, 2nd ed, 1999).
9 ANCOLD, Guidelines on Risk Assessment (2003).
10 Exhibit 35, **ACD.001.0001**, .0008.

The Guidelines are not:

- *a textbook on concrete gravity dams and their design*
- *a design code or Australian Standard. Hence the mandatory 'shall' has been avoided in the text.*

1991 ANCOLD Guidelines – limit state design

3.64 The Australian guidelines current at the time the Dam was designed and constructed were ANCOLD's *Guidelines on Design Criteria for Concrete Gravity Dams* published in 1991 (**1991 ANCOLD Guidelines**).[11] The '*limit state design*' approach that they advanced is recorded as having '[*fallen] out of favour*' by the time the Dam was designed.[12] The 1991 ANCOLD Guidelines were replaced by the 2013 ANCOLD Guidelines. The latter adopted a factor of safety approach. That approach had been the one traditionally applied in practice. Both design approaches are directed to ensuring a margin of strength above anticipated load combinations to give a dam as designed an acceptable margin above those combinations. This amounts to conservatism that is built into the calculations to make reasonably certain that a dam will be stable under a range of anticipated conditions.

3.65 The 1991 ANCOLD Guidelines dealt, in section 6.0, with the topic of 'Stability, Strength and Serviceability'. Section 6.1 set out a range of load factors to be adopted in sliding stability calculations. In summary, they suggested an augmentation for factors which contribute to a dam's instability and the opposite for factors thought to contribute to stability. Consistent with what is later explained, this approach was not used for the Dam. This seems to have been consistent with engineering good practice at the time. For this reason, further consideration need not be given to the 1991 ANCOLD Guidelines.

2013 ANCOLD Guidelines – factor of safety design

3.66 The 2013 ANCOLD Guidelines contain the following statement in the Foreword:[13]

> *Within about ten years of the 1991 edition of the ANCOLD Guidelines on Design Criteria for Concrete Gravity Dams being published the Limit State design approach it proposed fell out of favour with Australian dam engineers. … A Working Group and a separate Expert Review Panel were formed in 2005 to review the Guidelines with the intention of preparing a document that was not only more in line with the current thinking but would also be widely adopted and used by practicing dam engineers.*

11 Exhibit 33, **ACD.003.0001**.
12 Exhibit 35, **ACD.001.0001**, .0005.
13 Exhibit 35, **ACD.001.0001**, .0005.

3.67 The membership of the working group included:[14]

a. Richard Herweynen and Timothy Griggs (both were designers of the Dam, the former having certified the construction of the Dam as a Registered Professional Engineer of Queensland (**RPEQ**))

b. David Ryan, who worked in the office of the Director, Dam Safety

c. Graeme Bell, a member of the first Technical Review Panel (**TRP**) for the Dam

d. Brian Forbes, who had a very brief involvement in a due diligence commenced by SunWater concerning the Dam

e. Peter Foster, who was a member of SunWater's second TRP and who gave evidence in this Commission, including about standards and guidelines for dam design.

3.68 Much if not all of the Dam's design occurred before the establishment of the ANCOLD working group for the 2013 ANCOLD Guidelines. The Dam was designed and built well before their publication. This does not mean, however, that those Guidelines are unimportant. As the Guidelines note, engineering practice quickly moved away from what the 1991 ANCOLD Guidelines proposed with respect to limit state design. Indeed the Preliminary Design carried out by SunWater during the proposal stage noted that at that time, the 1991 ANCOLD Guidelines were under review following criticism that the limit state method of design was difficult to apply.[15] It is a safe inference that practice had moved back towards a factor of safety approach by the time the Dam was designed. The Detail Design Report adopts a factor of safety approach for assessing shear sliding. That approach is said[16] to have been derived from the USBR (1976),[17] USACE (1995)[18] and ANCOLD (1991)[19].

3.69 This report will deal in some detail with the work that GHD has recently done to assess the stability of the Dam. For present purposes, it suffices to say that the approach of GHD was that any sliding stability assessment ought be undertaken with regard to the standards or guidelines as applicable at the time the assessment was carried out.[20] No single standard or guideline commands absolute authority because matters of engineering judgment fail to be applied.[21]

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14 Exhibit 35, **ACD.001.0001**, .0004.
15 Exhibit 96, **DNR.003.7930**, .7961.
16 Exhibit 24, **GHD.002.0001**, .0141.
17 **PDI.071.0001**.
18 Exhibit 248, **HER.003.0001**.
19 Exhibit 33, **ACD.003.0001**.
20 **TRA.500.003.0001**, .0071 ln 36-38.
21 **TRA.500.004.0001**, .0035 ln 22-28.

3.70 GHD's adoption of the 2013 ANCOLD Guidelines as the primary basis for the assessment of the Dam's sliding stability is considered appropriate for several reasons. First, those Guidelines are the most recent Australian guidance on the topic, and reflect the current state of knowledge and practice about the principles to adopt. Secondly, those Guidelines reflect the learning and practices which preceded the publication of the Guidelines, so the approach they adopt is one that can be taken as having had support before 2013. Thirdly, Mr Herweynen and Mr Griggs were involved in their preparation.

3.71 Mr Herweynen (the Dam's Principal Designer) and Mr Griggs (who reported to Mr Herweynen) (as members of the ANCOLD working group[22]) must have known that the limit state approach had fallen out of favour at the time they worked on the design of the Dam.

3.72 The 2013 ANCOLD Guidelines, in section 5.0 'Material Properties and Strength', provide guidance for the strength parameters to be adopted at the interface between RCC lift joints. That section applies to concrete generally. Materially, the 2013 ANCOLD Guidelines state:[23]

> *The critical failure mode in the dam mass will be along planes of weakness in the concrete, such as the lift joint. The test data is focused on testing along planes of weakness. Samples are described as 'bonded' if they are intact and as 'unbonded' if the sample is broken along the plane of weakness.*
>
> *…*
>
> *The **peak shear strength** is the peak strength on a bonded sample.*
>
> *The **sliding friction strength** is the peak strength on an unbonded sample.*
>
> *The **residual strength** is reached following large displacements.*

22 ANCOLD members may participate in the work of a variety of ANCOLD working groups. Technical working groups prepare reports and papers for publication by ANCOLD. They also assist ICOLD technical committees in preparing ICOLD Bulletins.

23 Exhibit 35, **ACD.001.0001**, .0025 – .0026 (emphasis added to last paragraph quoted only).

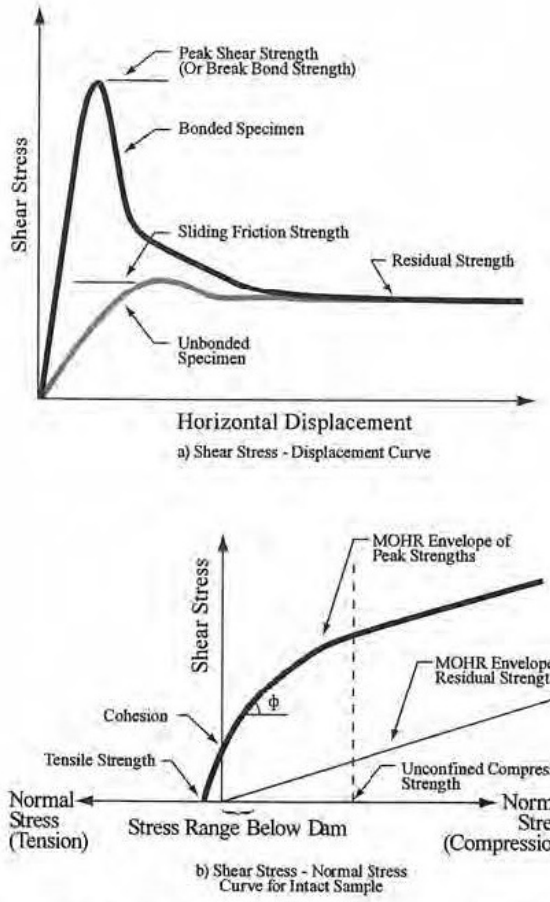

Figure 5.1: Shear strength terminology (reproduced from EPRI, 1992).

…

Good lift joint preparation, tight lift joints and good quality concrete may support the argument that the lift joint is bonded. However, in practice it is often difficult to determine whether a concrete lift joint is bonded or unbonded without obtaining intact samples by careful drilling, which could include horizontal drilling along lift joints. ***Therefore unless there is strong evidence to support bonded lift joints or investigations based on cored samples are undertaken, all concrete lift joints should be considered as unbonded****.*

3.73 Section 6.0 of the 2013 ANCOLD Guidelines concerns the acceptance criteria to be used in designing a dam. Section 6.1, 'Sliding Stability', states that:[24]

A sliding mode of failure will occur along the presumed failure surface when the net applied shear force (T) exceeds the resisting shear force (T_f) as defined by sliding factor (SF) below (Equation 6.1).

[24] Exhibit 35, **ACD.001.0001**, .0029.

$$SF = \frac{T_f}{T} = \frac{N\tan\phi + cL}{T}$$

(Equation 6.1)

Where

N = resultant of forces normal to the assumed failure surface. Note that many failure surfaces are on relatively steep slopes, in which case N will have a significant component from the horizontal loads and T will have a significant contribution from the horizontal loads.

ϕ = angle of internal friction along the failure surface, as appropriate (Section 5)

c = cohesion, as appropriate (Section 5)

L = the uncracked length of the base.

3.74 Section 6.4.1 of the 2013 ANCOLD Guidelines contains the acceptance criteria for sliding stability. Within that section, Table 6.1 (reproduced below) identifies the recommended minimum factors of safety against sliding for use on failure surfaces within the dam (here, principally, RCC lift joints). The 2013 ANCOLD Guidelines propose that peak and residual strength cases *'should be analysed'*. Which values of the peak strength parameters are adopted will depend upon the quality of the available site investigations and any laboratory strength data.[25]

Table 6.1: Recommended minimum Factors of Safety against sliding

Scenario	Load Case		
	Usual	Unusual	Extreme
Peak strength — c' and Φ' 'not well defined'[1,5]	3.0[3]	2.0[3]	1.5[3]
Peak strength — c' and Φ' 'well-defined'[1,5]	2.0[3]	1.5[3]	1.3[3]
Residual strength c' and Φ' 'well-defined'[1,5,8] (refer to Section 5.1, particularly Figure 5.1)	1.5[2,9]	1.3[2,9]	1.1[2,9]

3.75 Table 6.1 is accompanied by the following relevant notes about stability considerations within the dam and at the dam-foundation interface:[26]

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

25 Exhibit 35, **ACD.001.0001**, .0030.
26 Exhibit 35, **ACD.001.0001**, .0030.

2. *For these lower FoS to apply the residual strength c' would normally be expected to be zero. It is common practice to assume c'=O and Φ' = 45° for residual strength of concrete. This is consistent with EPRI (1992) data which has 90% of samples with a strength > c'=O, Φ' = 48° for the sliding friction shear strength (peak strength of unbonded specimens[)]. However, it needs to be noted that there are some cases where Φ' could be significantly lower, such as at AAR cracked surfaces, signs of open joints, at joints where there is evidence of calcite on the downstream face (implying leaching of the bedding surface), at surfaces where the lift joints were placed with a reasonably thick cement slurry bond layer, especially where little control may have been used during construction, and a surfaces subject to grinding. In these cases, and in all cases where the residual strength controls the analysis of stability, the residual strength should be determined by laboratory tests carried out on samples from the dam by appropriately qualified and experienced assessment personnel.*

2013 ANCOLD Guidelines – peak and residual strength analyses

3.76 How should the provisions of these guidelines be interpreted with respect to sliding stability and the use of both peak and residual shear strength parameters in analysis of stability? As noted above, the 2013 ANCOLD Guidelines indicate that both peak and residual strength cases *'should be analysed'*. However, it is open to interpretation as to how these analyses should be conducted, what assumptions ought be made, and what should be done with the outcomes of them.

3.77 Table 6.1 in the 2013 ANCOLD Guidelines contemplates that different types of analyses may be performed, adopting different values for the cohesion and friction strength parameters, denoted as *c'* and Φ', depending on the level of confidence in the selected strength values, i.e. whether they are *'well-defined'* or *'not well-defined'*. Explanations of these terms appear in the notes to Table 6.1 above.

3.78 Different acceptance criteria (i.e. different values of the minimum acceptable factors of safety) are given in Table 6.1. Which to use depends upon whether: (1) the analysis is conducted using peak or residual strength parameters; (2) how well the selected strength values are known or how well they are defined; and (3) the nature of the loading being considered (i.e. '*Usual*', '*Unusual*' or '*Extreme*'). In general, the acceptance criteria for peak strength conditions are more stringent than those for residual strength conditions. This reflects the fact that the residual case is considered the case where the lowest shear strength of the sliding interface is mobilised. It may also reflect the likelihood of greater uncertainty about the peak strength parameters in general.

3.79 A reasonable interpretation of these is that, at least in principle, stability analyses should be conducted for both peak and residual conditions and that all relevant acceptance criteria should be met. This means that the calculated factors of safety in each set of computations should not exceed the relevant acceptance limits for that set, i.e. the minimum acceptable factor of safety for the relevant case.

3.80 If this interpretation is correct, then it also needs to be recognised that reliable values of the strength parameters may not be readily available for both the peak and residual cases, and perhaps for neither.

3.81 For example, if the dam designer is confident that all lift joints within the dam are, or will be, bonded, it may be possible to estimate 'well-defined' values for the strength parameters of the bonded joints for both the peak and residual conditions. This would then allow sliding stability calculations to be performed and estimates of the factors of safety to be made for both the peak and residual conditions. These computed factors of safety could then be compared with the relevant acceptance criteria listed in Table 6.1. All acceptance criteria would need to be met for the design to be considered safe with respect to sliding.

3.82 There is a particularly important reason why larger values of the minimum acceptable factors of safety are specified for the stability analysis based on peak strength values. The graph in Figure 5.1 of the 2013 ANCOLD Guidelines indicates schematically that the bonded specimens typically display what is known as a *'brittle'* response to shearing. In this case the shear stress rises to a pronounced peak and then declines significantly beyond the peak as the shear displacement increases. Test data for bonded lift joints subjected to shearing support this indication of a brittle response, e.g. see Figures 12 and 13 of the 1999 paper by Dr Ernest Schrader.[27] Situations where the shear displacement corresponding to the peak is reached or exceeded must be avoided, because once the peak is exceeded the strength of the shearing interface may reduce markedly, thus accelerating the shear failure. Hence the greater caution for the peak strength case. In an attempt to avoid brittle failure of this type relatively large values of the factor of safety are recommended for the analysis of bonded lift joints when analysed using peak strength parameters.

3.83 If no or scant peak strength data are available for the lift joints, or if it is known that the lift joints are, or will be, largely unbonded, then greater reliance ought be placed on meeting the residual strength acceptance criteria. In this case it is important to recognise that the 2013 ANCOLD Guidelines require the residual strength parameters to be 'well-defined'. Unlike for peak strengths, there is no provision for a residual strength analysis using parameters that are 'not well-defined'. Furthermore, the Guidelines indicate that '"*Well-defined" means a sufficient number of tests have been done on concrete core from the dam and lift surfaces to give the strength parameters with reasonable certainty'.*[28] The Guidelines state specifically that residual shear strength testing should be undertaken if the residual strength condition is controlling the dam's stability.[29]

27 Exhibit 124, **PDI.040.0001**, .0016.
28 Exhibit 35, **ACD.001.0001**, .0030.
29 Exhibit 35, **ACD.001.0001**, .0026.

2013 ANCOLD Guidelines – RCC

3.84 Section 10 of the 2013 ANCOLD Guidelines bears the title 'Roller Compacted Concrete Gravity Dams'. The introductory section at 10.1 relevantly provides:[30]

> *In general, design principles for CVC and RCC gravity dams are similar. However there are some additional design considerations introduced by the RCC construction method. These include mixture proportioning, concrete properties, additional considerations during the design analysis, requirements regarding uplift in the body of the dam, uncertainties and variability in lift joint strength, seepage control measures, simplicity of design to maximise the speed of construction, and additional testing and quality control requirements to ensure adequate Factors of Safety against sliding.*
>
> *Section 10 highlights these differences and special considerations. Although some of these issues are mainly construction-related, they require consideration during the design phase.*

3.85 Section 10.3.3 is headed '*Shear strength*' and states:[31]

> *Shear strength is generally the most critical hardened property for RCC gravity dams. The total shear strength of bonded lift joints is the sum of cohesion plus sliding friction resistance across the lift joints. Shear resistance of unbonded lift joints includes only the sliding friction resistance between the lift surfaces.*
>
> *The shear strength along lift surfaces is always less than the mass concrete. Therefore, as for tensile strength, the strength at lift surfaces will govern the design. As with CVC, cohesion varies a great deal between lift surfaces, while the angle of internal friction is generally consistent.*
>
> *Factors that affect tensile strength at lift joint surfaces, as described for tensile strength, also affect shear strength. Therefore the methods to increase the tensile strength of RCC lift joints can also be used to increase the shear strength.*
>
> *Cohesion can vary from 0 to 10 % of the compressive strength. For preliminary design investigations a value of 5 % of the compressive strength may be used for lift joint surfaces that are to receive mortar bedding; otherwise, a value of 0 % should be assumed. For joints placed within the initial set time of the RCC, 10% may be adopted. The angle of internal friction can vary from 35° to 60° (see Section 5.1).*

30 Exhibit 35, **ACD.001.0001**, .0044.
31 Exhibit 35, **ACD.001.0001**, .0046 – .0047.

3.86 The 2013 ANCOLD Guidelines deal further with the choice of strength parameters to be used as inputs for a factor of safety analysis of dam stability:[32]

> *In the case of bonded lift joints, it is suggested the peak shear strength parameters be taken as c' = 0.95MPa and Φ' = 57° based on EPRI (1992) data. Adopting these values without site-specific test data requires the use of the 'not well defined' Factor of Safety acceptance criterion (Table 6.1). Where sufficient site-specific test data is available to give the strength parameters with reasonable certainty (e.g. 80% of the test results from a test regime involving a sensible number of tests exceed the assumed strengths) then the 'well defined' Factor of Safety acceptance criteria are proposed (Table 6.1).*

3.87 Appendix A of the 2013 ANCOLD Guidelines contains background information on the selection of strength parameters and acceptance criteria for use in sliding stability factor of safety calculations.[33] It quotes extensively from the US Federal Energy Regulatory Commission (**FERC**) Dam Safety Guidelines Chapter III – Concrete Dams[34] and includes the following information:[35]

> *With regard to safety factors, FERC places strong reliance on the no cohesion assumption-based analyses because of the difficulty in reliably defining cohesion.*

3.88 Chapter 5 of the 2013 ANCOLD Guidelines deals specifically with 'Material Properties and Strength'. Page 23 contains the following statements:[36]

> *There is great scatter in the test data available for concrete lift joints and the dam-foundation interface (EPRI 1992; Danay & Adeghe 1993; Dawson et al 1996) so thorough investigations with testing, if required, should be undertaken as part of the assessment of both existing and new dams.*
>
> *In these Guidelines, based on experience with existing dams (EPRI 1992), it has been recognised that the lift joints are sometimes of poorer quality than the dam body. Accordingly, if any tests are to be carried out for determining strength, lift joint testing should take precedence.*

3.89 These statements provide a clear warning to designers to exercise caution in the selection of strength parameters, particularly shear strength parameters pertaining to lift joints, when analysing the possibility of failure by shear sliding. These warnings are especially important for RCC dams in which the numerous lift joints provide planes of weakness. Various guidelines discussed later in this report manifest general agreement that intrusive sampling and shear strength testing of samples is probably the most reliable means of assessing the likely shear strength of lift joints.

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32 Exhibit 35, **ACD.001.0001**, .0026.
33 Exhibit 35, **ACD.001.0001**, .0060 – .0062.
34 FERC, *Engineering Guidelines for the Evaluation of Hydropower Projects*, Chapter 3, 'Gravity Dams' (2002).
35 Exhibit 35, **ACD.001.0001**, .0061.
36 Exhibit 35, **ACD.001.0001**, .0025.

2013 ANCOLD Guidelines – lift joint quality

3.90 On the topic of 'Lift joint quality', the 2013 ANCOLD Guidelines state:[37]

> ***10.3.4 Lift joint quality***
>
> *The general approach with RCC construction is to place the concrete such that it acts as a monolith without discontinuities after the concrete has set. However the layered method of RCC construction produces a series of closely spaced lift joints which, if not correctly treated, could result in weak or unbonded planes within the body of the dam.*
>
> *The strength of RCC lift joints may be increased by increasing the strength or cementitious content of the mixture, limiting the time between the successive layers to ensure the RCC is placed on a fresh joint surface, and using good lift joint surface treatment methods, e.g. the application of a bonding mixture such as bedding mortar between lifts.*
>
> *The allowable length of time between placements of layers depends upon the condition of the surface and the joint maturity (surface temperature x time since previous layer was compacted). When the time or maturity limits applicable to the site have been exceeded, the lift joint is considered to be a cold joint and specific lift joint treatment procedures are required. This includes wire brooming, placing mortar bedding over the surface or green cutting the top of the previous layer.*
>
> *In order to evaluate the acceptability of a lift joint, a Lift Joint Quality Index (LJQI) has been developed as a guide and has been used on some projects. This comprises a scoring system considering surface condition (segregation and tightness), rain damage, curing, maturity, surface flatness, method of RCC delivery, or other harmful conditions. The score achieved is then used to classify the lift joint in the range excellent to very bad. A surface treatment should then be specified based on the LJQI to ensure a lift joint with adequate strength to meet the design assumptions for lift tensile and shear strength. Nonetheless to achieve maximum joint shear resistance, placing lifts within the initial set time governs. The LJQI approach has been used principally with low cementitious, low-workability, RCC mix dams.*

3.91 As is discussed in detail later in this report, the LJQI was used during construction of the Dam and took on some significance in the Commission. The reference to the LJQI in the 2013 ANCOLD Guidelines is a recognition of its existence and of its use. The reference is not an endorsement of it. Nevertheless, the mention of the LJQI is far from a criticism. If anything, it gives the LJQI legitimacy for use during the construction of RCC gravity dams.

3.92 However, on any reading of the 2013 ANCOLD Guidelines it is not possible to interpret the use of the LJQI as a substitute for shear strength testing, in particular because of what is said in section 6.4.1. Furthermore, nowhere do the Guidelines

[37] Exhibit 35, **ACD.001.0001**, .0047.

endorse the LJQI as a basis for verifying that a dam has satisfied the acceptance criteria for sliding stability. In addition, a surface treatment may not be able to remedy all types and degrees of quality issues that the LJQI detects so as to ensure that a lift joint meets the design assumptions for shear strength.

Shear strength of lift joints

3.93 The Dam was constructed using RCC in layers about 300 mm thick. It therefore contains a large number of approximately horizontal lift joints between the many successive layers that made up the full height of the Dam wall. The Dam is no exception to the principle that the shear strength of the lift joints in a gravity dam constructed using RCC can be critical in determining the stability of the dam with respect to shear sliding. With the use of low cementitious RCC (**LCRCC**) it probably assumes even greater significance.

3.94 How is the shear strength of the lift joints in the Dam achieved? What influences or determines the cohesive and frictional components of the shear strength of the lift joints in the Dam?

3.95 The frictional component of the shear strength of a lift joint depends largely on the mineralogy of the large aggregate in the RCC and to a lesser extent on how well the aggregate is interlocked in the immediate vicinity of the lift joint surface. The latter is influenced to a significant degree by construction practices, with good joint preparation, tight joints and the use of bedding mix on the joints (described below) potentially increasing the degree of interlocking.

3.96 The cohesive component of the lift joint shear strength is highly dependent on the degree and strength of bonding of the aggregate particles that can be achieved between adjacent RCC lifts. The latter requires not only good quality RCC but also good joint preparation. Good placement technique includes avoiding segregation (also described below) of the top layer, particularly at its underside. The application of bedding mix to a joint normally improves the joint bonding and therefore its cohesion.

3.97 'Bedding mix' (also known as 'bonding mortar' or 'bedding mortar') is a material often placed between the foundation and the RCC or at lift joints between successive layers of RCC. A bedding mix is usually a high-slump, high-cement content mix of sand, cement, water, and perhaps set retarding admixture. In some cases it may also contain coarser aggregate than sand. As indicated, it is used to increase bonding and tensile strength of a lift joint and to improve water tightness by filling any voids that may occur at the lift joint.

3.98 Several different compositions of bedding mix were used in the construction of the Dam. Details of these mixes and their application in the Dam may be found in the Final Design Report[38] and in various Quality Control reports,[39] respectively.

3.99 Segregation is the term used to describe the separation of the constituent materials of the CVC or RCC mix. Good concrete (whether CVC or RCC) is one in which all the ingredients are distributed to make a homogeneous mixture. There are considerable

38 **ALC.002.001.0950**, .0976-.0977.

39 For example, **ALC.001.001.0658**, .0767; **SUN.128.002.0001,** .0040-.0041.

differences in the sizes and specific gravities of the constituent ingredients of CVC and RCC and therefore it is natural that the materials will exhibit a tendency to fall apart.

3.100 Segregation in CVC or RCC can have numerous causes, including:

a. a badly proportioned mix containing insufficient fine particle matrix to bind and contain the coarser aggregates

b. insufficiently mixed ingredients with excess water content

c. dropping of the mix from heights

d. discharging the mix from a badly designed mixer, or from a mixer with worn out blades

e. transporting the mix long distances by conveyor or haul trucks and dumpers

f. excessive vibration of the mix.

3.101 In the construction of an RCC dam, both 'hot' and 'cold' lift joints may be created. When an RCC lift layer is covered with the next lift layer before it reaches initial set of either mix, it is considered a fresh plastic joint or 'hot joint'. This type of joint normally produces a strong watertight joint for reasonably workable RCC mixes. If the period between placements of the consecutive layers is too long, a 'cold joint' will be developed. The period required for a cold joint to occur depends on various factors, but principally the ambient temperature at the time of layer placement. A cold joint potentially has relatively low bond strength and an increase in permeability. Once a cold joint develops, the application of bedding mix to the bottom layer may be required to achieve the required bond strength as well as watertightness.

3.102 In general, an RCC layer placed on a so-called 'hot' joint has greater potential to develop bonding across the interface and therefore greater cohesion, or cohesive component of shear strength, than for RCC placed on a 'cold' joint. However, as indicated, the cohesion and thus the shearing resistance of a cold lift joint can be significantly improved by the addition of a bedding mix between adjacent RCC layers. The bedding mix may contain coarse aggregate that is smaller than the largest aggregate used in the RCC mix and significantly more fines (including cement) than the RCC mix.

3.103 More than 8,600 t of bedding mix was used in the construction of the Dam.[40] According to calculations by Mr Herweynen,[41] this amount of bedding mix corresponds to an average coverage of 20% of all lift joints in the Dam. Given this recorded usage and estimated average coverage, it is reasonable to assume that bedding mix probably conferred some cohesive component of shear strength on some lift joints.

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40 Exhibit 38, **SUN.110.003.0001**, .0104.

41 Exhibit 244, **HER.001.0001**, .0042; Exhibit 247, **TRA.510.007.0001**, .0076; **TRA.500.013.0001**, .0016-.0017.

3.104 However, with respect to bedding mix, the following questions remain and are difficult to answer: how much cohesion is conferred, and on which areas of which lift joints? Unfortunately, there is only limited sampling and no shear strength testing of lift joints that could provide definitive answers to these questions. The Specification required the application of bedding mix adjacent to the upstream face of the Dam on every lift joint. The minimum width of the upstream bedding strip was 600 mm, but much wider strips were specified on cold joints.

3.105 The difficulty in taking bedding mix into account when assigning strength parameters of lift joints is knowing with a reasonable degree of certainty where it was placed. Simply assuming each and every lift joint had 20% coverage is not warranted and is potentially dangerous.

3.106 The actual distribution of bedding mix (rather than the average) is important with respect to estimating sliding stability. If the same coverage (either absolute or relative to total surface area) was applied on each and every lift joint, then a systematic assessment of its contribution to the sliding stability of the Dam may be possible. However, if, say, a single lift joint had only a narrow (e.g. 600 mm wide) strip of bedding applied to its upstream face, then the contribution of that strip of bedding mix to the overall shear resistance of the entire lift joint could be minimal.

3.107 This can be illustrated by a simple numerical example. Consider the case where a lift joint is located near the base of the Dam (where the overall width of the lift is about 36 m). For this case, a 600 mm wide strip of bedding mix with a cohesive strength of 2,400 kPa as assumed by the designers,[42] would provide a maximum effective cohesion for the entire width of the lift of approximately 40 kPa (= 2,400 x 600 / 36000). It is estimated that this effective cohesion, applied to the entire width of the joint, would account for a maximum contribution to the overall shearing resistance of approximately 1,440 kN/m (= 40.3 x 36). In the absence of uplift pressures and any downward component of the tailwater force and any hydraulic crest force being transmitted to that joint, the effective normal force at this location near the base of the Dam should be on the order of approximately 20,000 kN/m. Hence, if the friction angle of this joint is in the order of 40 degrees (which was similar to the design basis), the overall shearing resistance of the lift joint will be in the order of 1,440 + 20,000 tan(40) = 1,440 + 16,782 = 18,222 kN/m. In other words, in this case the contribution of the bedding mix to the overall shearing resistance of this lift joint would be about 7.9% (=1,440 / 18,222). This percentage is an example of what is meant by the potential contribution of the bedding mix to the overall shear strength of the lift joint being *'minimal'*.

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42 There is some uncertainty whether the design cohesion value for a good quality lift joint treated with bedding mix was 2,400 or 2,600 kPa. The Detail Design Report uses the former value in Table 5-2 but the latter in Table 5-4: Exhibit 24, **GHD.002.0001**, .0140, .0141. Mr Herweynen said that the value was 2,400 kPa: **TRA.500.013.0001**, .0019 ln 3-8, .0068 ln 32-35. Mr Griggs said that the value was the average of cohesion for an excellent and a poor quality treated lift joint ((2,800 + 2,000) / 2 = 2,400): **TRA.500.014.0001**, .0083 ln 27-30. There are documents that state that the design cohesion was 2400 kPa: see, for example, Exhibit 87, **DNR.005.4145**, .4408; Exhibit 9, **IGE.019.0001**, .0033. It seems likely that the correct design figure for cohesion of a good lift joint treated with bedding mix was 2400 kPa and that is value assumed in this report.

3.108 Furthermore, if that particular lift joint, with the minimum amount of bedding mix (600 mm width strip), happened to be near the base of the Dam (as assumed above) or in a more critical location of the Dam, it could provide a distinct plane of weakness on which sliding failure may be initiated.

3.109 The issue of the shear strength of the lift joints at the Dam has also been addressed at some length in the Paradise Dam Upgrade Options Assessment report of Aurecon, commissioned by Building Queensland.[43] It is worthwhile quoting directly from this report in regards to the questions posed immediately above:[44]

> *(i) [T]he shear strength of the lift joints is the most critical hardened property for RCC gravity dams affecting the sliding stability within the dam wall, more important than the shear strength of the mass concrete within the lift layers. In the case of Paradise Dam, the lift joint shear strength is also one of issues of most uncertainty. …*
>
> *(ii) The original design of Paradise Dam assumed that all the lift joints would be bonded and adopted the sum of cohesion and sliding friction resistance. It has been demonstrated in the Dam Safety Review (SunWater 2016a) and subsequent investigations, that this is not the as-constructed condition.*
>
> *(iii) The importance of understanding of the lift joint shear strength has been recognised by SunWater and it has been extensively investigated since 2014. As the lift joints in Paradise Dam are considered unbonded, the shear resistance should include only the sliding friction resistance between the lift surfaces. The investigations have appropriately focussed on estimating a representative friction angle only.*
>
> *(iv) The original design assumptions regarding lift joint shear strengths, made in lieu of testing, are considered reasonable. However, based on the records, it is not clear whether the lift joint shear strength was further investigated and confirmed through testing at the time of construction, which should have been done. The original assumptions now appear non-conservative.*
>
> *(v) The shear strengths adopted in the Dam Safety Review (SunWater 2016) are reasonable 'typical' shear strength values when referring to a data base of test results (e.g. EPRI 1992) in lieu of field assessments. However, it did not consider the actual condition of the concrete cores at the lift joint locations, i.e. it is on the low side for peak shear strength for bonded lift joints, and on the high side for sliding friction strength of unbonded lift joints. The residual shear strength is around* [a] *typical mean value.*
>
> *…*

43 **PDI.100.0001**.
44 **PDI.100.0001**, .0020-.0021.

(vii) The effect of honeycombing in the upper layer (top of lift joint) has not been considered in detail. Although some samples presented boney concrete above the lift joint, only sample RCC-S 2.3-3.3 included honeycomb concrete. Given the lack of paste (matrix) holding the course aggregate in place, the concrete above the lift joint could crush and might reduce the lift joint shear strength. However, this is not always the case and it depends on the degree of honeycombing. It was noted that the tests on samples with honeycombing on the joint, produced lower residual strengths, some less than 30°.

(viii) The proposed average residual shear strength of ϕ = 37-39.3° and c = 0 for up to 600 kPa normal stress, is considered appropriate for use in the options assessment, given limited available data. However, more certainty is required for the feasibility study and detailed design.

3.110 Consideration of what the designers stated the Dam would achieve in terms of its essential characteristics, particularly the shear strength of the lift joints, can be found in the Specification[45] and in the Detail Design Report.[46] These documents need to be understood against the background of various guidelines and standards in effect at the time, as well as what engineering good practice required. These matters are discussed in more detail in the Chapters that follow.

45 Exhibit 20, **DNR.004.4559** and Exhibit 21, **DNR.003.8385**.
46 Exhibit 24, **GHD.002.0001**.

Chapter 4 – RCC aspects of the Dam

Roller compacted concrete

4.1 The Paradise Dam (**the Dam**) was constructed using layers of Roller Compacted Concrete (**RCC**) about 300 mm thick. It therefore contains a large number of horizontal lift joints between the many layers. As with most relatively large RCC dams, the shear and tensile strengths of the lift joints between layers are critical to structural stability. The advantages and disadvantages of using RCC in construction and the important mechanical properties of it are discussed in the following sections.

RCC as a construction method

4.2 RCC is a method as well as a product. It combines rapid placement techniques from mass earthworks with a mixture that provides the strength and durability characteristics of concrete.[1]

4.3 Before the development of RCC technology, concrete dams were built in independent vertical monoliths that were constructed in a stair-step pattern across a valley. Each monolith had to be formed on all sides and poured in successive blocks between 1.5 m and 3 m in height. The method of construction was labour and time intensive.

4.4 The development of RCC technology began in the 1970s to reduce the cost of building concrete dams. RCC dams could be built by continuous two-dimensional construction in layers from one side of a gorge or valley to the other.[2] More rapid construction reduced construction costs.[3]

4.5 RCC contains the same general ingredients as Conventional Concrete (**CVC**), although the ratios of the materials used in it are different. It is mixed on site and delivered by trucks or conveyor where it is placed continuously from one abutment to the other in a series of horizontal shallow layers known as '*lifts*' (typically 300 mm deep). To achieve a void-free matrix, RCC is typically compacted using heavy vibrating rollers, although smaller compaction equipment can also be used where space is limited. To be placed and compacted effectively, it must be dry enough to support the weight of the construction equipment, yet workable enough to achieve the necessary compaction to prevent segregation and the formation of voids.

4.6 The advantages that RCC dams offer compared to CVC and earthen fill dams include:[4]

- *More rapid construction (with associated social and environmental benefits);*
- *Reduced cement utilisation (and in some cases cementitious materials);*

1 Exhibit 69, **ICO.002.0001**, .0023.
2 Exhibit 100, **TAG.001.0001**, .0003 [9] to .0004 [12].
3 Exhibit 69, **ICO.002.0001**, .0023.
4 Exhibit 69, **ICO.002.0001**, .0026.

- *Efficient use of equipment;*
- *Greater simplicity of construction;*
- *Reduced formwork;*
- *Simplified river diversion and management during construction (intentional overtopping has been successfully incorporated as part of the construction river management in the case of a number of major RCC dams);*
- *Horizontal construction increases the opportunity for the inclusion of an integrated coffer dam;*
- *An ability to use aggregates that may not be considered suitable for use in CVC dams; and*
- *Reduced direct and indirect implementation cost as a consequence of the above advantages.*

4.7 In seeking to take advantage of these benefits, the construction of dams with RCC has increased. More than 906 RCC dams have now been built or are due to be completed within the next 12 months. They are located in more than 70 countries around the world.[5] There are several in Australia, including Cotter Dam in the Australian Capital Territory and Wyaralong Dam near Beaudesert in Queensland.

Two main approaches for RCC dams

4.8 There are two primary approaches to building dams with RCC.[6] The first relies on an independent impervious barrier, usually fixed to the upstream face of the dam wall, to prevent permeation of water along the lifts joints. The second approach relies on the impermeability of the RCC itself and of the joints between lifts.[7]

4.9 The use of an independent membrane is most often associated with RCC that has a lower cementitious content. There are three categories of RCC based on the content of cementitious materials (i.e. Portland cement and mineral admixtures or supplementary cementitious materials, such as flyash):[8]

a. low cementitious RCC (**LCRCC**) with a cementitious content of less than 100 kg/m^3

b. medium cementitious RCC (**MCRCC**) with a cementitious content of greater than 100 kg/m^3 and less than 150 kg/m^3

c. high cementitious RCC (**HCRCC**) with a cementitious content of more than 150 kg/m^3.

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5 Database of all RCC dams throughout the world maintained by Malcolm Dunstan & Associates, accessed on 2 April 2020, <http://www.rccdams.co.uk/>.

6 Exhibit 69, **ICO.002.0001**, .0023.

7 Exhibit 69, **ICO.002.0001**, .0023.

8 Exhibit 69, **ICO.002.0001**, .0027.

4.10 LCRCC relies on the gradation of rock aggregates and uses only a small quantity of cementitious material. A low cementitious mix is not a fluid medium. It is frictional in nature, and its strength is obtained principally from the interlock of the aggregate particles. Voids in the mix are filled with graded fines and only the smallest voids are filled with cementitious paste.[9] Therefore, LCRCC usually has a higher permeability than HCRCC and must rely on additional measures to prevent seepage within the dam wall.[10]

4.11 HCRCC relies more on technology associated with conventional concrete. Its use of higher quantities of cementitious material generally results in all the aggregate particles being coated with paste. There is enough paste to fill all the voids with some left over, so that when the RCC is compacted by a vibrating roller, the excess paste rises to the surface of the lift to promote bonding with the next layer placed on top of it.[11] The rock aggregate sits in a matrix of cementitious material, which provides the strength.[12]

4.12 Rather than using an independent impervious barrier, such as an upstream membrane, the majority of new RCC dams adopt the second approach by relying on the impermeability of the RCC and especially HCRCC.[13] This trend is noted by International Commission on Large Dams (**ICOLD**): '*[s]ince the first generation of RCC dams, a general trend towards HCRCC types has been apparent*'.[14]

4.13 The Dam used LCRCC and an impervious membrane was fixed to the upstream face of the Dam where it was covered by precast concrete panels, presumably to provide protection from mechanical damage and sunlight.

LCRCC

Advantages

4.14 LCRCC is more economical than HCRCC because the ingredients in the mix cost less.[15] If the design of a structure is such that intimate bonding between lifts is not required, LCRCC is the most economical way to build.[16] An example is where the geometry of a dam is such that the surface area of a lift and the normal weight of the RCC lifts above it are large enough to develop frictional resistance that is sufficiently greater than the applied loads.[17] A dam with a larger footprint has, generally speaking, greater mass and bigger surface areas across which frictional resistance can develop, and therefore has higher sliding resistance.

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9 **TRA.500.002.0001**, .0004 ln 19-32; Exhibit 247, **TRA.510.007.0001**, .0008 ln 13-18.
10 **TRA.500.002.0001**, .0004 ln 40-43.
11 **TRA.500.002.0001**, .0005 ln 16-25.
12 Exhibit 247, **TRA.510.007.0001**, .0008 ln 20-25.
13 Exhibit 69, **ICO.002.0001**, .0024.
14 Exhibit 69, **ICO.002.0001**, .0027.
15 **TRA.500.002.0001**, .0022 ln 7-39.
16 Exhibit 48, **TRA.510.025.0001**, .0009 ln 35-39.
17 **TRA.500.007.0001**, .0013 ln 9-18.

4.15 Against the savings in cost of ingredients must be balanced the expense of installing an upstream membrane to provide watertightness[18] and the other 'add-ons' required because of the choice to use LCRCC.[19]

4.16 A further advantage of LCRCC is that the mix will produce less heat than an HCRCC mix. Heat can contribute to cracking of the mix as it cures. Thermal stresses in an LCRCC dam may be acceptable without the need for forced cooling,[20] for instance, by adding ice to the RCC mix.

4.17 LCRCC mixes were described in one Alliance[21] document as '*more like rubber, while [HCRCC] is more brittle, like cast iron or glass*'. The 'rubbery' nature of LCRCC is related to its modulus of elasticity. A low modulus means that the RCC can deform or 'stretch' or 'contract' or 'shear' more at a given load before it cracks in comparison to a mix with a higher modulus.[22]

4.18 A further benefit of LCRCC is the '*increased "creep" capability*' of that material. A document called '*Method Statement for RCC Overview and Trial Mix Specifications*' explained that:[23]

> *When the RCC is stressed by movement or strain due to attempted thermal or foundation movement, the level of stress gradually decreases with time. Traditional concrete and high cementitious content RCC have very little stress relaxation or creep, but lean RCC with lower strength and the type of gradation used by [the Hydro Tasmania consortium] has substantial creep and stress relaxation.*

Disadvantages

4.19 In an LCRCC mix, there is usually insufficient paste to both fill the voids and coat all the aggregate particles. The construction methodology relies upon compacting the RCC to achieve an unsegregated state.

4.20 LCRCC is prone to segregation because there is very little paste to hold the rock particles together. As explained in the previous Chapter, segregation is where particles in the RCC mix separate out because of differences in particle size, shape and density. Segregation in RCC can cause pockets of unconsolidated material within the RCC matrix. The figure below is an example of localised segregation in the surface of a lift joint:[24]

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18 **TRA.500.002.0001**, .0022 ln 14-18.
19 **TRA.500.002.0001**, .0022 ln 32-36.
20 Exhibit 22, **DNR.010.8266**, .8269.
21 Exhibit 18, **SUN.009.002.0020**, .0026.
22 Exhibit 22, **DNR.010.8266**, .8268.
23 Exhibit 22, **DNR.010.8266**, .8268 to .8269.
24 Exhibit 324, **LOJ.003.0001**, .0013.

Figure 4.1 – Area of segregation in lift surface

4.21 Segregation of the LCRCC mix can occur during construction, including when placing, spreading and rolling the RCC into layers.[25] Care in construction is required. The ICOLD acknowledges that the requirements of RCC dam construction are '*actually much more complex than is initially apparent*' and, as a result, recommends '*only allowing bids from suitably experienced contractors, particularly for large, or more complex RCC dams*'.[26]

4.22 The time during which RCC gains strength is generally longer for LCRCC than HCRCC. That presents difficulties for verification testing of shear strength in LCRCC. At an early stage, HCRCC is more robust and can withstand the torque and frictional force imparted by the drill bit cutting through the material to extract a core.[27] LCRCC has low early strength. It is also inherently weak at the lift joints because, unless bedding mix is applied, there is no extra paste to enhance bonding between layers. This combination makes it difficult to assess a core sample to determine whether lift joint bonds have been damaged during the drilling process or were debonded *in situ*. Not only does that mean that LCRCC requires more care and more effort to get reliable testing results,[28] it also means that LCRCC is susceptible to assertions that, because the core is said to have been damaged either in its taking or in its transportation after sampling, the testing shows a false positive in terms of the lift joints being unbonded. However, a person with sufficient experience may, with careful inspection, make an appropriate assessment.[29] There was a difference between the witnesses with knowledge on this topic about the extent to which such an assessment could be relied upon to assess the quality of lift joints.

25 **TRA.500.002.0001**, .0004 ln 32-38.
26 Exhibit 69, **ICO.002.0001**, .0029.
27 **TRA.500.002.0001**, .0022 ln 44 to .0023 ln 12.
28 **TRA.500.008.0001**, .0013 ln 39-47, .0015 ln 13-17.
29 **TRA.500.002.0001**, .0027 ln 15 – 27.

4.23 If LCRCC is well-compacted and there are no problems with porosity, it is possible to drill cores and obtain samples for testing.[30] Daryl Brigden (whose expertise included the material properties of RCC) said that coring in LCRCC or MCRCC '*to retrieve joints is best performed with the hole angled*'.[31] That technique gradually subjects lift joints to the torque applied by the drill bit. While coring can be done vertically, the torque of the drill bit can spin the core on top of the existing joint, whether it is bonded or not. That '*makes it very difficult to pick up the degree of bonding and the condition of the joint*'.[32] Angled coring assists with that problem. The core can be tested in a triaxial shear machine and the forces resolved to account for the angle of the lift joint to the axis of the core sample and the principal loading direction.[33]

4.24 There was evidence that block samples provide more reliable shear strength test results for LCRCC than core samples.[34] Block samples are typically cut from a trial embankment. They are tested in a large machine and the process is not difficult in principle.[35] Block testing, however, is relatively expensive to undertake and because of its sampling limitations cannot involve many lift joints, as with a vertical or diagonal core sample.

4.25 Hydro Tasmania submitted that it was not practicable to have conducted shear testing at the Dam for three reasons.[36] First, it is more difficult to shear strength test cores from LCRCC than from HCRCC. Four US-based RCC experts gave evidence concurrently: Dr Ernest Schrader, Dr Paul Rizzo, Timothy Dolen and Stephen Tatro. Of them, Dr Schrader, Dr Rizzo and Mr Tatro agreed that it was generally more difficult to test cores of LCRCC in shear,[37] and Mr Dolen said that it would be more difficult if the LCRCC was affected by problems with porosity.[38] However, none suggested that coring LCRCC for the purpose of obtaining reliable samples to shear test was impossible.

4.26 In recommending further testing on the Dam, two of the experts included further coring of the LCRCC: Mr Tatro[39] and Dr Rizzo.[40] The latter recommended that cores be subjected to shear testing. Mr Dolen put a desire for more testing in the context of incurring risks to the downstream population. However, he said that to provide a good statistical evaluation of the percentage of lift bonding (which Mr Tatro and Dr Rizzo said they wanted), cores could be tested.[41]

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30 **TRA.500.008.0001**, .0014 ln 19-27.
31 **TRA.500.002.0001**, .0014 ln 40-42.
32 **TRA.500.002.0001**, .0015 ln 4-6.
33 **TRA.500.002.0001**, .0015 ln 8-10.
34 **TRA.500.008.0001**, .0014, ln 4-10.
35 **TRA.500.002.0001**, .0026 ln 10-44.
36 **HYT.008.0001**, .0053 to .0054 [171].
37 **TRA.500.008.0001**, .0013 ln 39-47, .0014 ln 4-15, .0014 ln 31 to .0015 ln 17.
38 **TRA.500.008.0001**, .0014 ln 19-27.
39 **TRA.500.008.0001**, .0070 ln 6-25.
40 **TRA.500.008.0001**, .0070 ln 41 to .0071 ln 9.
41 **TRA.500.008.0001**, .0071 ln 41 to .0072 ln 3.

4.27 The second reason given by Hydro Tasmania that it was *'not possible'* to undertake shear strength testing with any reliability was because of a lack of equipment and people experienced in carrying out the testing.[42] The evidence relied on was Dr Schrader's response to a question whether it had been his decision not to subject the Dam's trial embankment to shear strength testing:[43]

> *You know, a lot of this is the best of my recollection, but I'm quite sure - and you could ask Bruce Embery or Mark Hamilton, but I think we had some talk about, 'It would be great to do shear tests. Are they normally done for a project this size and this height?' The answer is absolutely not. 'Would we be able to get results before we really progressed to the point that we're constructing the dam?' The answer is no.* ***'Would we be able to get them tested at a place and in a way that they would be credible?' The answer was probably not.*** *'Would they be very expensive?' The answer was probably yes. And so a decision was made, I think it was like a discussion amongst us, and shear testing was not done.*

4.28 Dr Schrader said he thought that there was '*probably not*' a testing facility that could reliably test the LCRCC. However, Mr Brigden said that the then Department of Main Roads (**DMR**) laboratory at Herston had '*specialised equipment for doing RCC trial mixes and testing*'[44] and was fully equipped to conduct all manner tests on RCC.[45] Mr Brigden was not challenged about this.

4.29 The third basis for Hydro Tasmania's submission is that it was not possible to '*undertake shear strength testing within the timeframes usually allowed*'.[46] It is not clear what is meant by this. In any event, shear strength testing could have been conducted at three different stages during construction.[47] The RCC mix design stage could have included shear testing samples of simulated lift joints between RCC layers. A core for testing could have been taken from the trial embankment once it had reached sufficient maturity. Testing at either of those stages would have returned results well before construction ended. Finally, a core could have been taken from the Dam wall proper. Even had the RCC only achieved sufficient maturity by the time the Dam was largely constructed, test results would have provided an objective and independent basis for verifying that the design shear strength values had been attained.

Inexperience of Alliance members with RCC

4.30 Few RCC dams had been built in Australia before 2003. The Copperfield River Gorge Dam (formerly Kidston Dam) was one of the first (1984).[48] It was privately built as part of the Kidston Gold Mine. Another early RCC structure was the Bucca Weir,

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42 **HYT.008.0001**, .0053 [171(b)].
43 **TRA.500.010.0001**, .0075 ln 36 to .0076 ln 1 (emphasis added)
44 Exhibit 48, **TRA.510.025.0001**, .0019 ln 39to .0020 ln 5.
45 Exhibit 48, **TRA.510.025.0001**, .0021 ln 19-23.
46 **HYT.008.0001**, .0053 [171(b)].
47 What is usually done to verify shear strength is discussed in detail in Chapter 5.
48 CSIRO, *Agricultural resource assessment for the Gilbert catchment*, 350, accessed 10 April 2020 <https://publications.csiro.au/rpr/download?pid=csiro:EP143222&dsid=DS4>.

[49] constructed in 1987. The Burton Gorge Dam was constructed in late 1992 to provide water supply for the North Goonyella coal mine.[50] Kroombit Dam, built in 1992, was the first public infrastructure dam constructed using RCC in Queensland. Kroombit Dam's reservoir has a capacity of 14,600 ML. It was built to ensure continuity of water supply to irrigators downstream of the dam and to replenish the region's aquifers.[51]

4.31 The Dam was the second public infrastructure RCC dam in Queensland, and was constructed more than ten years after Kroombit Dam. At the time of the Dam's construction, its head designer, Andreas Neumaier, wrote, in an article published about the Dam, as follows:[52]

> *When constructed, the Burnett dam will have some unique features, including:*
>
> - *The first high dam in Australia with dedicated upstream and downstream fishways for all migratory fish species and fish sizes;*
>
> …
>
> - *The largest volume RCC dam in Australia.*

4.32 There were few people in Australia at the time that the Dam was being designed and built who had knowledge of, and experience in, the construction of RCC dams, particularly as large as the Dam. This caused problems for the Dam's design and construction.

4.33 Only one dam had ever been built from LCRCC before in Australia; the Burton Gorge Dam in Queensland, which was completed in 1992. As Paradise Dam did, the Burton Gorge Dam incorporated precast concrete panels with a geomembrane on its upstream face.[53] There are aspects of LCRCC that require greater care and attention than when building with RCC generally. The workforce on the Dam had no prior experience of those difficulties and had to be trained on the job. The specialist skills required for the construction were lacking at the start.

4.34 None of the Dam's designers had designed an RCC dam before, let alone one using LCRCC. However, the Alliance had, as an advisor to it, Dr Schrader. He is a US-based world expert in RCC. Dr Schrader had considerable influence in the RCC-related aspects of the design and construction of the Dam and the Australian-based design team relied heavily upon him. Dr Schrader made important decisions and

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49 SunWater, Bundaberg Weirs and Barrages, accessed on 11 April 2020 <http://www.sunwater.com.au/weirs-and-barrages/bundaberg/>.

50 Database of all RCC dams throughout the world maintained by Malcolm Dunstan & Associates, 'Burton Gorge' accessed on 11 April 2020 <http://www.rccdams.co.uk/dams/burton-gorge/>.

51 SunWater, Kroombit Dam, accessed on 11 April 2020 <http://www.sunwater.com.au/dams/kroombit-dam/>.

52 Neumaier A, 'Burnett River: Australia's largest volume RCC dam', (2004) 4 *International Journal on Hydropower & Dams*.

53 Database of all RCC dams throughout the world maintained by Malcolm Dunstan & Associates, 'Burton Gorge' accessed on 2 April 2020 <http://www.rccdams.co.uk/dams/burton-gorge/>.

gave advice which had a bearing on the course and development of the Dam's design and construction.

4.35 The choice to use LCRCC for the Dam was made in part for financial reasons. The decision had important consequences.

Design of Paradise Dam

Tender stage

4.36 On 20 January 2003,[54] Burnett Water Pty Ltd (**Burnett Water**) expressed its intention to enter into an alliance agreement with private sector participants as part of a tender response for the design and construction of the Dam[55]. It did so by issuing an '*Invitation to Submit a Registration of Interest*' for the Burnett River Dam.[56]

4.37 On 14 February 2003, a consortium comprising Hydro Tasmania, SMEC Australia Pty Ltd (**SMEC**), Macmahon Contractors Pty Ltd (**Macmahon**) and Walter Construction Group Limited (**Walter**) (together, **Hydro Tasmania Consortium**) lodged a registration of interest.[57] Burnett Water issued the '*Burnett River Dam Stage 1 - Request for Proposals*'[58] on 17 March 2003. Burnett Water invited three consortia to respond.[59] The request included, as Appendix F, a preliminary design for Paradise Dam prepared by SunWater Limited (**SunWater**) (**Preliminary Design**).[60]

4.38 The Preliminary Design provided for the primary spillway to be constructed of RCC, although the cement content was not stated. The material properties of RCC stated in the Preliminary Design included:[61]

a. compressive strength at 90 days of 10 MPa

b. ultimate static tensile strength of 0 kPa, save for an allowable static tensile strength of 63 kPa for RCC with lift joints spread with bedding mix for the unusual and extreme static load cases

c. tensile strength under dynamic loading of 1600 kPa

d. ultimate cohesion of 0 kPa for areas of RCC lift joints without bedding mortar

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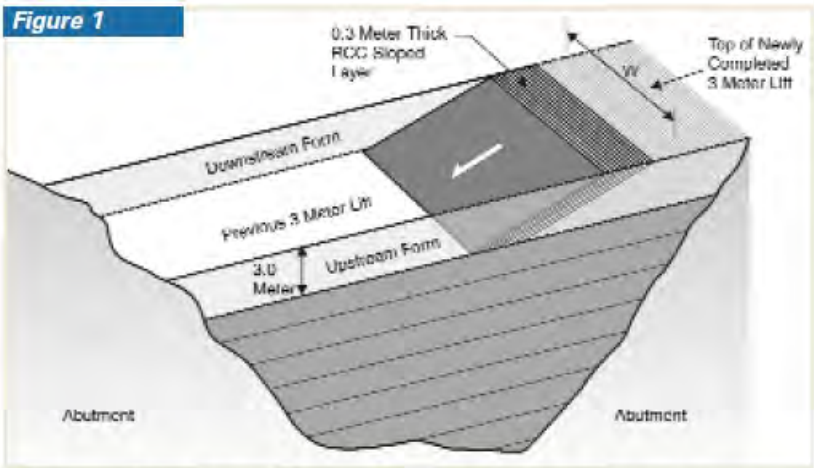
54 Exhibit 244, **HER.001.0001**, .0003 [12].
55 Exhibit 2, **ALL.155.008.0001**, .0002.
56 Exhibit 2, **ALL.155.008.0001.**
57 Exhibit 244, **HER.001.0001**, .0004 [14].
58 Exhibit 250, **SWA.500.001.2366**.
59 Exhibit 244, **HER.001.0001**, .0004 [15].
60 Exhibit 96, **DNR.003.7930** (Volume 1), **HYT.006.004.1338** (Volume 2), **HYT.006.004.0985** (Volume 3), **HYT.006.004.1233** (Volume 4), **HYT.006.004.0950** (Volume 5), **HYT.006.004.0913** (Volume 6).
61 Exhibit 96 **DNR.003.7930**, .7968.

e. cohesion of 200 kPa in areas with bedding mortar, the extent of which was at least one-third of the lift surface, or the width necessary to achieve the required factors of safety for sliding stability.

4.39 The Preliminary Design set out the results of the structural stability assessment of monolith blocks in the primary spillway. Assessments were made of the Dam's sliding factor of safety (based on friction only) and of the shear friction factor of safety (including cohesion and friction). The results showed that the desired sliding factor of safety was not achieved for all flood events considered in some planes. However, SunWater considered that, because the sliding factor of safety was achieved when account was taken of cohesion due to bedding mix, the Dam was stable with respect to sliding.[62]

4.40 Section 17.5 of the Preliminary Design headed '*Roller Compacted Concrete*' explained that the design allowed for the use of the '*sloped layer placement method*'. That method of construction places the RCC layers on a slope from one abutment to the other between formed upstream and downstream faces. The slope means that much thicker lifts (e.g. 3 m) are constructed out of the 30 cm layers as is depicted below:[63]



This figure shows how a 3.0-meter lift in an RCC dam can be constructed in 0.3-meter-thick sloped layers, which can be placed within the initial set time of the RCC in the previous layer.

*Figure 4.2 – Sloped layer method depicted in Mr Forbes's article. (Exhibit 307, **SME.001.0001**, .0002)*

4.41 The Preliminary Design described the sloped layer method as offering advantages including:[64]

- *more rapid RCC production rates;*
- *reduced construction costs;*

62 Exhibit 96, **DNR.003.7930**, .7971.
63 For example, Exhibit 307, **SME.001.0001**, .0002.
64 Exhibit 96, **DNR.003.7930**, .7992.

- *a significant reduction in area of construction joints requiring treatment;*
- *assistance in curing operations;*
- *progressive raising of formwork with less exposure to flood damage; and*
- *minimisation of area of RCC exposed to damage by rain.*

4.42 The sloped layer method was the subject of oral evidence. The Design Manager of the Alliance, Mr Neumaier, was not aware of the method at the time he worked on the Paradise Dam project. He first heard of it when working on an RCC dam in Malaysia afterwards.[65]

4.43 The Hydro Tasmania Consortium lodged its stage 1 proposal with Burnett Water on about 11 April 2003.[66] About a month later (on 9 May 2003),[67] Burnett Water issued its '*Burnett River Dam Stage 2 - Request for Proposal*'.[68] It invited two consortia from stage 1 to proceed to stage 2: the Hydro Tasmania consortium and a consortium involving Thiess and URS.[69]

4.44 The Hydro Tasmania Consortium lodged its stage 2 proposal with Burnett Water on 1 August 2003.[70] Burnett Water later announced that the Hydro Tasmania consortium was the successful tenderer.[71]

Contract award and Alliance Agreements

4.45 In October 2003, the Alliance was formed when Hydro Tasmania entered into an '*Alliance Agreement – Burnett River Dam*' with Burnett Water, SMEC, Macmahon and Walter (**2003 Alliance Agreement**).[72] The 2003 Alliance Agreement was dated 27 February 2004.[73]. After voluntary administrators were appointed to Walter, the 2003 Alliance Agreement ended. In around May 2005, it was replaced by an agreement between the same parties, save for Walter (**2005 Alliance Agreement**).[74]

4.46 Under Schedule 7 to each of the Alliance Agreements, SMEC and Hydro Tasmania were responsible for:[75]

> *The provision of resources to carry out all necessary design work for the design of the Works in accordance with the Alliance Agreement, including: design management, design validation and certification and the provision of specialist*

65 **TRA.500.015.0001**, .0011 ln 33 to .0012 ln 5.
66 Exhibit 251, **HYT.510.004.0001**.
67 Exhibit 244, **HER.001.0001**, .0005 [21].
68 Exhibit 252, **SWA.500.001.2068**.
69 Exhibit 244, **HER.001.0001**, .0005 [21].
70 Exhibit 244, **HER.001.0001**, .0006 [25]; Exhibit 231, **DNR.007.0477**.
71 Exhibit 244, **HER.001.0001**, .0006 [26].
72 Exhibit 244, **HER.001.0001**, .0006 to.0007 [27]; Exhibit 18, **SUN.009.002.0020**.
73 Exhibit 18, **SUN.009.002.**0020, .0025.
74 Exhibit 244, **HER.001.0001**, .0007 [28]; Exhibit 19, **ALL.144.002.0389**.
75 Exhibit 18, **SUN.009.002.**0020, .0089; Exhibit 19, **ALL.144.002.0389**, .0460.

design related services during the construction and commissioning of the Works and the provision of all necessary design, specification or other professional services not expressed to be provided by any other Other Alliance Participant.

4.47 'Works' were defined to mean the physical works and improvements to be completed under the Alliance Agreements that were to be handed over to Burnett Water at practical completion.[76]

Responsibilities during the detail design stage

4.48 The responsibilities of some members of the Alliance are set out in the '*Burnett River Dam Alliance – Design Management Plan (during design phase)*' dated 24 March 2004.[77] Mr Neumaier was the originator of that document.[78] His position was described as the '*Design Manager*'[79] and he was responsible for all aspects of the Dam's design.[80] His particular responsibilities included:[81]

a. managing and coordinating design consultants' activities and monitoring progress against the procurement programme and the design programme

b. obtaining verification from consultants and design and construct subcontractors that the documented design complied with all relevant items of the functional specification and statutory requirements

c. ensuring designers certified design by regularly inspecting the works during construction.

4.49 Mr Neumaier had no further involvement after mid-2004.[82] From that time, Richard Herweynen, of Hydro Tasmania, became responsible for any of the residual duties that may otherwise have been exercised by Mr Neumaier.[83]

4.50 Mr Herweynen was the '*Design Team Leader Dam*' and led the geotechnical and dam structure aspects of design.[84] His responsibilities included:[85]

a. preparing detailed specifications, schedules and drawings as appropriate for construction works

b. carrying out design review and verification of specifications, schedules and drawings.

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76 Exhibit 18, **SUN.009.002.0020**, .0032; Exhibit 19, **ALL.144.002.0389**, .0401.
77 Exhibit 297, **SUN.162.002.0149**.
78 Exhibit 297, **SUN.162.002.0149**, .0149.
79 Exhibit 297, **SUN.162.002.0149**, .0161.
80 Exhibit 297, **SUN.162.002.0149**, .0162.
81 Exhibit 297, **SUN.162.002.0149**, .0163 [2.4.2].
82 **TRA.500.015.0001**, .0003 ln 35-40.
83 **TRA.500.014.0001**, .0049 ln 30-34.
84 Exhibit 297, **SUN.162.002.0149**, .0161-.0162.
85 Exhibit 297, **SUN.162.002.0149**, .0164.

4.51 During the construction phase of the Dam, Mr Herweynen was the '*Design Coordinator*'.[86] He was responsible for ensuring that construction met the design intent.[87] Timothy Griggs assisted Mr Herweynen in that task as the '*Civil / Dam Design Engineer*'.[88] Mr Griggs was also the '*onsite design presence*'.[89] Mr Griggs had also been involved during the design phase. He had worked on, among other things, the dam stability analysis and the design of the primary spillway apron and was supervised by Mr Herweynen.[90]

4.52 Of the Detail Design Report, Mr Neumaier relevantly:

a. prepared the '*Executive Summary*'[91]

b. prepared and reviewed '*Section 1 – Introduction*'[92]

c. reviewed '*Section 4 – Hydraulic Design*'[93] and '*Section 7 – Outlet Works*'.[94]

4.53 '*Section 5 – Dam*' was prepared by Mr Griggs and Joanna Sheedy and reviewed by Mr Herweynen.[95] Dr Schrader prepared '*Section 6 – RCC Design*', which Mr Herweynen reviewed.[96]

4.54 Mr Neumaier was also involved in preparing the Specification for the Paradise Dam. He approved for issue:

a. '*Section 1.0 – General*', which Mr Neumaier also prepared[97]

b. '*Section 3.0 – Surface Excavation and Earthworks*' that Mr Herweynen prepared and Mr Neumaier also reviewed[98]

c. '*Section 5.0 – Protection and Support of Excavation*', which was prepared by David Starr of Golder Associates and reviewed by Mr Herweynen[99]

d. '*Section 11.0 – RCC Dam Construction*' and '*Section 12.0 – Guidelines for RCC & Concrete Inspection Quality Control and Quality Assurance*', which were both prepared by Dr Schrader and reviewed by Mr Herweynen.[100]

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86 Exhibit 115, **SUN.175.006.0009**.
87 Exhibit 244, **HER.001.0001**, .0039 [182(c)].
88 Exhibit 115, **SUN.175.006.0009**.
89 Exhibit 88, **DNR.005.4886**, .4890 and .4925.
90 Exhibit 287, **GRT.001.0001**, .0002 [9].
91 Exhibit 24, **GHD.002.0001**, .0004.
92 Exhibit 24, **GHD.002.0001**, .0015.
93 Exhibit 24, **GHD.002.0001**, .0056.
94 Exhibit 24, **GHD.002.0001**, .0255.
95 Exhibit 24, **GHD.002.0001**, .0121.
96 Exhibit 24, **GHD.002.0001**, .0222.
97 Exhibit 20, **DNR.004.4559**, .4561.
98 Exhibit 20, **DNR.004.4559**, .4585.
99 Exhibit 20, **DNR.004.4559**, .4620.

4.55 All those sections of the Specification were approved for construction by Mark Hamilton (Project Manager) and Steven Johnson (Construction Manager at the relevant time).[101]

Dr Schrader's role

4.56 Dr Schrader was initially engaged by Walter during the tender phase.[102] On 9 December 2003, he was retained by Hydro Tasmania to provide '*expert advice*' to Hydro Tasmania about:[103]

a. RCC mix design

b. RCC material properties

c. thermal analysis

d. proposed design details for the Dam

e. RCC specification and quality control

f. RCC construction methodology

g. Dam foundation requirements

h. other aspects of the project, as would be agreed between Dr Schrader and the Superintendent of the professional service contract (who appears to have been an employee of Hydro Tasmania).[104]

4.57 Dr Schrader was influential in decisions made by the designers about RCC-related aspects of the design and construction. The Alliance was reliant on his knowledge and skill, which Dr Schrader knew.[105]

RCC mix design

RCC mix developed for the Preliminary Design

4.58 As part of developing the Preliminary Design, Mr Brigden, of SunWater, visited the Dam site. He identified an area of basalt that might be used as aggregate in the RCC mix. SunWater arranged a trial blast of that material and then transported it to a quarry in Rockhampton where it was crushed according to a target grading design. The information from that crushing process was presented to tenderers during the Alliance selection process.[106]

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100 Exhibit 21, **DNR.003.8385**, .8441, .8498.
101 Exhibit 20, **DNR.004.4559**, .4585, .4620; Exhibit 21, **DNR.003.8385**, .8441, .8498.
102 Exhibit 244, **HER.001.0001**, .0010 [43(c)].
103 Exhibit 256, **HYT.505.004.0147**, .0147.
104 Exhibit 256, **HYT.505.004.0147**, .0149.
105 **TRA.500.010.0001**, .0007 ln 19-37.
106 Exhibit 48, **TRA.510.025.0001**, .0019 ln 12-35.

4.59 Mr Brigden went on to develop an RCC mix using the basalt from the site. That mix was subjected to basic testing (although Mr Brigden was unable to recall whether shear strength testing was carried out).[107] The results of that testing provided background and baseline parameters[108] for the Alliance selection process.

4.60 During stage 2, the two consortia tested their RCC mix designs in a laboratory at Herston that was operated by the then Department of Main Roads (**DMR**).[109] That laboratory had been sold by SunWater to DMR in 2002 on the proviso that all the specialised equipment that SunWater had developed for RCC trial mixes and testing would remain in place along with some select personnel. The laboratory was well equipped because SunWater had pioneered testing of RCC in Australia, in conjunction with Mr Forbes.[110] The laboratory was equipped to carry out numerous tests on RCC, including shear strength testing.[111]

4.61 Mr Brigden explained that design parameters for RCC were generally based on data from earlier projects, with guidance given by Australian National Committee on Large Dams (**ANCOLD**) and ICOLD. He considered it to be orthodox to use the RCC mix design stage to establish that selected design parameters were attainable with the particular mix under consideration.[112] Mr Brigden explained:[113]

> *[F]or each individual mix, in my experience, we have set out during the trial mix program to test for the parameters of shear strength, compressive strength, tensile strength, both indirect and direct, in some cases permeability, modulus, and that's done on - a trial mix program in my experience and the trial mix programs that I design will cover a range of cementitious, they'll cover a range of gradings, particularly in the fines side of things, and mixes of cement and pozzolan, various ratios, and of the suite, once you get the first indicative results, which is - generally compressive strength is the easiest to test and to do. RCC is a little different, because it's long term, because most designs are based around 365 days or 90-day compressive strength.*
>
> *We get around that by hot curing cylinders. We first tried that at Bucca [Weir], and you can develop a relationship between a hot-cured cylinder for 24 hours or a hot-cured for seven days, or both, with your long-term strength development once you have enough data, but that, in my experience, has always been part of the trial mix program where it's then - the trial mix program of the number of test trials that are done is then condensed based on the first indicators to say this is the likely range and then within that much smaller range. Because the rest of the testing is quite involved, quite expensive and time consuming, we would then do shear testing, moulding, cylinders with the joint in the middle, and mock up cold*

107 Exhibit 48, **TRA.510.025.0001**, .0020 ln 31-41.
108 **TRA.500.002.0001**, .0009 ln 10-21.
109 Exhibit 48, **TRA.510.025.0001**, .0019 ln 41 to .0020 ln 5; **TRA.500.002.0001**, .0009 ln 14-16.
110 Exhibit 48, **TRA.510.025.0001**, .0021 ln 18-23; **TRA.500.002.0001**, .0008 ln 38-41.
111 Exhibit 48, **TRA.510.025.0001**, .0021 ln 18-23.
112 Exhibit 48, **TRA.510.025.0001**, .0018 ln 31-43.
113 Exhibit 48, **TRA.510.025.0001**, .0017 ln 16 to .0018 ln 3.

joints, warm joints, hot joints, (indistinct), et cetera, and then test them in direct shear.

It can also be done by using a triaxial shear approach as well, but it's a little more involved. And the other things that we follow through from that is the tensile strength indirect and direct and, as I said, permeability based on cylinders. We have also moulded larger blocks in the past and included joints in those blocks.

RCC mix proposed by the Thiess and URS consortium

4.62 The Thiess and URS consortium proposed using an HCRCC mix to construct the Dam's primary spillway with 210 kg/m^3 of cementitious material, including Portland cement and flyash. The content of cementitious material was proposed by that consortium to be lowered to 120 kg/m^3 for the secondary spillway. The HCRCC mix offered the following '*key benefits*':[114]

1. *A strong bond will be achieved between individual lifts, giving a homogeneous structure with a low permeability.*

2. *The mix has some tensile capacity and this capacity has been utilised in the design, where appropriate, to optimize the section.*

3. *A high workability, which assists rapid placement of RCC.*

Alliance RCC mix design

4.63 During stage 1 of the tender process, the Hydro Tasmania consortium had anticipated using a high flyash mix. At that stage, the RCC mix proposal was for 60 kg of cement and 95 kg of flyash based on a 15 MPa design strength.[115] That strength was not far off the final design strength of 14 MPa.[116] The benefits of flyash were recognised as being to:[117]

- ***assist workability***
- *reduce heat generation*
- ***increase life of joints between lifts which could be important as exposure time is likely to be typically in excess of 12 hours and often greater than 24 hours due to the area of the placement surface and constraints of RCC placing due to the current design features.***

4.64 Emphasis has been added to the quote above because problems during construction of the Dam can be related back to the low workability of the mix ultimately chosen and the long exposure time. Most of the RCC lift joints in the Dam were cold joints.[118]

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114 Exhibit 81, **DNR.007.1087**, .1111.
115 Exhibit 251, **HYT.510.004.0001**, .0020.
116 Exhibit 24, **GHD.002.0001**, .0140.
117 Exhibit 251, **HYT.510.004.0001**, .0020 (emphasis added).
118 **TRA.500.006.0001**, .0037 ln 40-41.

4.65 After the Hydro Tasmania consortium had been selected as the successful Alliance partners, Dr Schrader and David Brett were involved in the RCC mix design. Mr Brett was retained by Hydro Tasmania to provide expert advice on RCC technology, including in relation to RCC mix design development.[119]

4.66 Dr Schrader developed the mix design program. Several mixes were tested before the decision to use LCRCC was made.[120] The design program involved testing on a range of mixes, including for compressive and split tensile strength of the RCC.[121] No shear strength testing was undertaken.

4.67 Mr Brett liaised with the DMR laboratory in Herston and developed the range of mixes that were to be tested based on the advice of Dr Schrader.[122] Mr Brett reviewed test results and prepared reports on the testing as it progressed.[123] Those reports show the following mixes to have been tested:[124]

Designation	Cement content (kg/m^3)	Fly ash content (kg/m^3)	Moisture content (%)	Washed 75µ fines content (%)
W4 80-00-4.5 (6.9)	80	0	4.5	6.9
W6 80-00-4.7 (6.3)	80	0	4.7	6.3
W6a 80-00-4.7 (6.3)	80	0	4.7	6.3
W7 70-00-4.7 (6.3)	70	0	4.7	6.3
W7a 70-00-4.7 (6.3)	70	0	4.7	6.3
W8 80-30-4.7 (6.3)	56	24	4.7	6.3
W8a 80-30-4.7 (6.3)	56	24	4.7	6.3
W9 80-60-4.7 (6.3)	32	48	4.7	6.3
W9a 80-60-4.7 (6.3)	32	48	4.7	6.3
W10 100-30-4.7 (6.3)	70	30	4.7	6.3
W10a 100-30-4.7 (6.3)	70	30	4.7	6.3
W11 80-00-4.7 (6.3) accelerated cure	80	0	4.7	6.3
W12 80-30-4.7 (6.3)	56	24	4.7	6.3
W13 120-00-4.7 (6.3)	120	0	4.7	6.3
W14 120-50-4.7 (6.3)	60	60	4.7	6.3
W15 80-00-4.7 (6.7)	80	0	4.7	6.7
W15a 80-00-4.7 (6.7)	80	0	4.7	6.7
W16 80-00-4.7 (6.7)	80	0	4.7	6.7
W16a 80-00-4.7 (6.7)	80	0	4.7	6.7
W17 80-00-4.7 (6.1)	80	0	4.7	6.1

119 Exhibit 257, **HYT.510.003.0049**.
120 **TRA.500.010.0001**, .0009 ln 20-26.
121 **DNR.020.019.0924**.
122 **PDI.091.0001**, .0002.
123 See, for example, Exhibit 156, **DNR.007.2295**, .2503.
124 Exhibit 156, **DNR.007.2295**, .2532-.2535.

Designation	Cement content (kg/m³)	Fly ash content (kg/m³)	Moisture content (%)	Washed 75µ fines content (%)
W17a 80-00-4.7 (6.1)	80	0	4.7	6.1
W18 80-00-4.7 (5.7)	80	0	4.7	5.7
W18a 80-00-4.7 (5.7)	80	0	4.7	5.7
W19 80-00-4.3 (6.3)	80	0	4.3	6.3
W19a 80-00-4.3 (6.3)	80	0	4.3	6.3
W20 80-00-5.1 (6.3)	80	0	5.1	6.3
W20a 80-00-5.1 (6.3)	80	0	5.1	6.3
W21 80-00-3.9 (6.3)	80	0	3.9	6.3
W21a 80-00-3.9 (6.3)	80	0	3.9	6.3
W22 80-00-5.5 (6.3)	80	0	5.5	6.3
W22a 80-00-5.5 (6.3)	80	0	5.5	6.3
W23 80-00-4.7 (3.3)	80	0	4.7	3.3
W23a 80-00-4.7 (3.3)	80	0	4.7	3.3
W24 80-00-4.7 (9.3)	80	0	4.7	9.3
W24a 80-00-4.7 (9.3)	80	0	4.7	9.3
W25 120-00-4.7 (6.3)	120	0	4.7	6.3
W25a 120-00-?? (6.3)	120	0	? [sic]	6.3
W25b 120-00-4.7 (6.3)	120	0	4.7	6.3
W26a 120-30-4.7 (6.3)	84	36	4.7	6.3
W26 120-30-4.7 (6.3)	84	36	4.7	6.3
W27a 120-60-4.7 (6.3)	48	72	4.7	6.3
W27 120-60-4.7 (6.3)	48	72	4.7	6.3
W28 100-00-4.7 (6.3)	100	0	4.7	6.3
W29 100-50-4.7 (6.3)	50	50	4.7	6.3

4.68 In a progress report on RCC testing dated 14 October 2003, Mr Brett stated that the compressive strength results showed '*the impressive strength gain of all mixes other than the two high flyash blends W 9 and W 27a*'.[125] When considering the compressive strength attained compared to cement content (and neglecting the effect of flyash), the results showed that cement was particularly efficient in the 55 to 80 kg/m³ range.[126] Mr Brett's recommendations for further testing included the following:[127]

> *A target mix design of 55 to 60 Kg/m3 seems feasible subject to appropriate workability and field performance. There does not seem to be any advantage in using flyash unless workability of the very lean mixes becomes an issue and flyash can be shown to economically improve this. It will be necessary to carry*

125 Exhibit 156, **DNR.007.2295**, .2505.
126 Exhibit 156, **DNR.007.2295**, .2506.
127 Exhibit 156, **DNR.007.2295**, .2513.

out additional testing to confirm the mix design and this would ideally be carried out in conjunction with trial crushing of similar or actual aggregates.

… The test program should include 50, 55, 60, 70 and 80 Kg/m3 cement contents, the latter two to give correlation of test results to the current series.

4.69 On 18 January 2004, Dr Schrader wrote to Mr Herweynen and Mr Johnson to provide an update on the material properties and thermal analysis from the trial program. Dr Schrader's executive summary stated:[128]

A mix with 60 kg^3 per cubic meter of Gladstone GP cement will result in RCC with properties that comfortably meet requirements for our design. This mix is recommended because it also results in the best thermal situation with essentially no risk of thermal mass cracking for the current design monolith joint spacing of 45 meters as well as for an increased spacing of 60 meters. It also is the most economical mix and it is simple.

4.70 A further progress report on RCC testing was prepared by Mr Brett on 22 January 2004.[129] By then, all testing to 180 days had been completed but no further results could be available until May 2004 when 365 day cylinders would be tested. Again, Mr Brett reported that cement was particularly efficient in the 55 to 80 kg/m^3 range.[130]

4.71 Mr Brett reported that two mixes had been made to evaluate the impact of '*time of compaction*' on density and compressive strength properties. 120 kg/m^3 of cement was used in both mixes, while one also had 50 kg/m^3 of flyash added.[131] That is nearly double the cement content ultimately adopted in the RCC mix for the Dam. The results of the testing revealed an '*unexpectedly long working time*' for the two mixes tested.[132]

4.72 The principal rock used for the aggregate component of the mix for the Dam was basalt from the diversion channel constructed through the right bank of the river. That was supplemented by quarrying additional basalt upstream of the diversion channel and by the 'Goodnight beds' from the Dam foundation.[133] During the mix design phase, testing was performed on those RCC aggregates to assess their potential for expansive breakdown. The aggregates were soaked in ethylene glycol and particle size gradation was assessed. No measurable breakdown was detected in any of the three aggregate types.[134]

4.73 In December 2003, concerns were raised by John Hunt, of Wagners Quarries Pty Ltd (the contractor engaged to operate the quarry, crushing facility, RCC mixing plant,

128 Exhibit 156, **DNR.007.2295**, .2297.
129 Exhibit 156, **DNR.007.2295**, .2517.
130 Exhibit 156, **DNR.007.2295**, .2519.
131 Exhibit 156, **DNR.007.2295**, .2524.
132 Exhibit 156, **DNR.007.2295**, .2525.
133 Exhibit 75, **PDI.037.0001**, .0003.
134 Exhibit 156, **DNR.007.2295**, .2525.

concrete batch plant and to manage the aggregate stockpile),[135] about the durability and reactivity of the Goodnight Beds material. Upon Mr Hunt raising concerns, Mr Brett stated that samples of the material tested to that point had performed well in trial mixes and had showed no breakdown in testing. However, the actual material exposed on site need to be reviewed.[136] Mr Brett recommended that Mr Hunt liaise with the site laboratory (once it was established) to establish a suitable testing regime.[137]

4.74 Mr Brett reported on RCC testing on blended cements products from Sunstate Cement on 3 March 2004.[138] Three trial mixes had been made up on 22 January 2004, all having 60 kg/m^3 of cementitious material. The first two used Sunstate products – one with 40% cement and 60% slag and the second with 75% cement and 25% flyash – while the third used cement from Queensland Cement Limited.[139] The results of compressive and tensile strength testing were available by March 2004 when Mr Brett concluded that either the slag blend or the flyash blend produced by Sunstate Cement would be suitable for RCC manufacture, although some additional testing would be required.[140] Mr Brett had previously expressed the view that using a blended cement would offer the following potential advantages:[141]

- *Lower cost*
- *Possibly improved workability*
- *Improved AAR [Alkali Aggregate Reaction] resistance*
- *Increased competition between suppliers.*

4.75 Mr Brett's interest in the blended mixes that Sunstate could provide was a point of contention with Dr Schrader. A peer review workshop was conducted about the RCC mix design on or around 19 January 2004. The resulting report records that:[142]

- *[David Brett] raised the economic benefit of admixtures:* ***Details required***
- *[Ernest Schrader] supports not using admixtures to cement:* ***Details required***

4.76 The proceedings of that RCC mix design workshop were summarised by '*K D Murray*', who worked for SRD Consulting and who was not said to have had any expertise in RCC. The report on the workshop relevantly stated:[143]

135 Exhibit 251, **HYT.510.004.0001**, .0096, 0101.
136 Exhibit 156, **DNR.007.2295**, .2499.
137 Exhibit 156, **DNR.007.2295**, .2498.
138 Exhibit 156, **DNR.007.2295**, .2538.
139 Exhibit 156, **DNR.007.2295**, .2540.
140 Exhibit 156, **DNR.007.2295**, .2544.
141 Exhibit 156, **DNR.007.2295**, .2557.
142 Exhibit 304, **GHD.043.0001**, .0003. (emphasis in original).

It was generally that the current trial mix program with 70 kg of cement per m3 was achieving strengths in excess of requirements and that it may be possible to reduce the cement content to 60 kg of cement.

It was suggested that consideration be given to commencing the RCC with 65 kg of cement and reducing to 60 kg once the RCC placement was in full production.

There was considerable discussion regarding the merits of using a slag cement and flyash in place of the general purpose cement currently being used in the trial mixes. The cost saving is of the order of $200K although there was some confusion about this. Some members of the Alliance believed that greater savings might be achieved.

Ernie Schrader is opposed to this on the following basis:

- *The current mix design is a robust mix that meets the required design criteria.*
- *This mix also has the flexibility to be able to deal with changes in inputs such as rock type and aggregate grading.*
- *Slag cement and flyash are the waste products of another process. If 60 kg of general purpose cement per m3 was required for the mix then it could be expected that in excess of 60 kg of a combined slag cement and flyash would be required. This would diminish potential cost saving.*
- *A combined slag cement and flyash mix would have smaller initial strength gains that would severely impact on the upstream panel placement.*

These concerns were noted in the workshop. However it is understood that further testing using the slag cement and flyash is proposed.

At the moment the quality of flyash available in Queensland is quite variable. This was an issue for the Thiess team whose mix design used flyash. In addition the experience with using waste products in the concrete mix at Burdekin Falls Dam was not good. The flow on cost penalties from the variation in the concrete mix was orders of magnitude greater than the cost saving achieved by using flyash from a cheaper source.

While it is acknowledged that the design team working on the Alliance has a good understanding of the issues involved, it is my recommendation that in accordance with Ernie Schrader's recommendation slag cement and flyash not be used. At the very least if the cost savings that can be achieved by using slag cement and flyash are considered significant, then a detailed assessment of the risks involved should be undertaken and clearly understood.

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143 Exhibit 305, **SUN.018.005.2929**, .2930 with highlighting in the document produced to the Commission removed.

4.77 Dr Schrader was opposed to using fly ash in the RCC mix. This is demonstrated by two memoranda he wrote to Mr Herweynen and Mr Johnson on 21 and 25 January 2004. In the first, '*Example of Poor Pozzolan Performance with Lean RCC & Basalt*', Dr Schrader said that poor performance of supplemental cementitious materials had '*typically been experienced when using basalt aggregate and gradations similar to what we have at Burnett River*'. It was '*essentially very probable that slag and/or fly ash will perform poorly in our lean RCC mixes*'.[144] In the second, Dr Schrader said that the results of the RCC mix design program for the Thiess and URS consortium supported '*our conclusions*' that flyash did very little with the aggregate materials for the Dam.[145]

4.78 On 28 January 2004, only six days after the Sunstate blended mixes were made up for testing, Mr Brett wrote a memorandum to Mr Herweynen saying the following:[146]

> *For your info I have reviewed Ernie Schraders memo re 'Poor Pozzolan Performance with Lean RCC and Basalt' and comment as follows. I realise that decisions have been made and I am comfortable with that if the decisions have been made on economic grounds. I don't necessarily disagree with the final outcome. However I do think that the case for alternative cements has not been fairly put or heard, particularly when Sunstate were asked to pay $10,000 for a test program and the results were not even considered.*
>
> *…*
>
> *None of the flyash mixes, or the slag mix in recent testing, showed any problematical behaviour. Performance of all materials was very similar. Vebe times where consistent with mix to mix variations much lower than the impact of added water for example.*
>
> *While Ernie has concentrated on strength, in fact, we currently are more concerned with too much strength and looking to reduce cement content. We have looked at test results and suggested that we are really only targeting around 10 MPa at 365 days. On Ernie's chart this would suggest going as low as 50 Kg but he is worried about going this low. It is likely we will end up 60-65 Kg as a practical level that gives a conservative result and improves workability, cohesiveness and helps reduce segregation. For all these benefits FA, and it seems slag, would perform just as well.*
>
> *…*
>
> *In summary, the tests show that strength of 30% flyash blends lags behind straight cement mixes by between 10%-30% getting within 10% at 180 days and potentially equalling over longer periods. On a strength for strength basis I can't see why we would need to allow any more than 10% more FA blend than GP based on this testing, assuming we were targeting a minimum strength, which we are not. On the other hand there is evidence from our testing that the FA seems to produce consistently higher densities in the cylinders. This is likely to be due to the 'lubricating effect of flyash.*

144 Exhibit 266, **DNR.020.019.1037**, .1039 to.1041.
145 Exhibit 266, **DNR.020.019.1037**, .1051.
146 Exhibit 266, **DNR.020.019.1037**, .1043 to .1045.

Our test program did not look at setting times but advice from Sunstate suggests the following:

Cement Type	*Initial Set*	*Final Set*
GP	*1.5-2 hrs*	*3 hrs*
GB (25% FA)	*2 hrs*	*3.5-3.45 hrs*

4.79 These documents reveal a difference of opinion between Mr Brett and Dr Schrader about using slag cement or flyash in place of some general purpose cement. Mr Herweynen's evidence was that Mr Brett was moved into a '*secondary role*' in early 2004,[147] about the time of the peer review and this disagreement. Mr Brett's last involvement with the Dam appears to have been in April 2004.[148] Mr Brett's recollection is that he was dismissed from the project because he had a difference of opinion with Dr Schrader.[149]

4.80 The Alliance's final progress report on RCC testing was dated 29 May 2004.[150] It was prepared by Dr Schrader (the earlier reports had been prepared by Mr Brett). By that time, all testing for the original test program had been completed. The report discussed compressive and tensile strength testing to which a range of RCC mixes had been subjected. Shear strength testing was not conducted as part of the RCC mix design program. Dr Schrader's final recommendation was that a mix with a straight cement content of 60 to 65 kg/m^3 be used.[151]

4.81 The final progress report mentioned the '*time of compaction*' testing that had been done on mixes using 120 kg/m^3 of cementitious material. However, because those mixes did not represent the lean mix with no flyash that Dr Schrader recommended, time of compaction testing would need to be '*re-run at the jobsite using precise materials and mixes for the dam*'.[152] Samples could be broken at an early age to allow results to be available quickly.

4.82 Dr Schrader's report stated that the option of using Sunstate blended products was not justified by '*[c]osts and the very limited test results*' available from the abbreviated test program to which those blended cements had been subjected.[153]

147 Exhibit 244, **HER.001.0001**, .0032 [148].
148 Exhibit 273, **DNR.020.019.1005**, .1014; **TRA.500.015.0001**, .0010 ln 28 to 32.
149 **PDI.091.0001**, .0003.
150 **SUN.126.002.0004**.
151 **SUN.126.002.0004**, .0023.
152 **SUN.126.002.0004**, .0020.
153 **SUN.126.002.0004**, .0006.

4.83 The outcome of the testing program was presented by Dr Schrader in the following way:[154]

The test results described have confirmed that

- *The target grading is appropriate and creates the basis of a successful mix.*
- *The mix is likely to be relatively insensitive to fines variation, but slightly high fines content could be beneficial.*
- *The probable best overall moisture content for the RCC will be about 4.7% to 4.9% at the time of compaction.*
- *Targeting a low modulus requires low cement content. On the order of about 60 to 65 Kg/m3. This cement content will result in a constructable mix with much more strength than required for the design.*
- *For the Alliance mixes and Burnett materials, fly ash does essentially nothing except add to cost and require an additional material to deliver, control, and mix.*

The specified RCC mix

4.84 Dr Schrader's recommendation to use LCRCC without slag or flyash admixtures was adopted by the Alliance. Section 11.3.2 of the Specification provided:[155]

S11.3.2 Mix Designs

The currently anticipated RCC mix design(s) are approximated below based on experience at other projects and preliminary information from mixes using aggregates similar to those expected to be used in this project. The proportions may adjust based on additional test data that becomes available at later ages, and on follow-up verification tests on samples of representative materials produced for construction. Weights are based on saturated surface dry aggregate. It is the intent to utilise a single RCC mix throughout the vast majority of the dam. However, if this cannot practically and economically meet the design criteria, zoned areas of different mixes will be used as necessary, for example with a higher cement content and strength at the top of the stilling basin apron.

The general criteria for mix designs(s) used in the dam will be:

(1) To provide adequate strength to meet structural design and durability criteria with normal or above normal factors of safely;

(2) To minimise internal heat rise from hydration and the subsequently developed stress or crack potential;

154 **SUN.126.002.0004**, .0023.
155 Exhibit 21, **DNR.003.8385**, .8445 to .8446.

(3) To maximise stress relaxation through creep and elastic properties;

(4) To provide a constructible mix; and

(5) To provide economy.

Although maximum compressive stresses are only on the order of about 2 MPa, friction between lift joints essentially provides sliding stability with no cohesion, and maximum tensile stress is on the order of about 0.1 MPa for minimal isolated areas, the estimated mix design given below for the vast majority of the dam is expected to result in strengths on the order of about 12 MPa at 1 year. This is much more than required for structural reasons. The cement content can therefore be reduced. This may be done in the field, perhaps to 65 kg, if the mix remains constructible, without segregation, and it results in suitable final in-place quality.

		APPX (SSD) kg/m³				
MIX	***PRIMARY USE***	***MAX AGG SIZE***	***CEMENT***	***FLY ASH***	***WATER***	***AGGREGATE***
1	*Mass (10 MPa)*	*51 or 63mm*	*65*	*0*	*120*	*2315*
2	*Top two lifts of the apron and auxiliary spillway, top lift of the dam, and special uses (25 MPa)*	*51 or 63mm*	*150*	*0*	*130*	*2220*
3	*Mass (10 MPa). If additional cohesion and workability are needed for localized areas with final delivery by truck.*	*51 or 63mm*	*85*	*0*	*135*	*2270*

It is expected that the mix will result in an immediate fresh mix modified VeBe time of about 20 to 30 seconds at a temperature of 20 C degrees. This is a relatively 'dry' RCC consistency. This information is provided to give a concept of the type of mix that will be used, but because of its subjective nature it will not be used as absolute field control.

Minor adjustments in exact mix proportions such as the added moisture required to obtain optimum compaction during cool/damp weather as compared to warm/wet weather shall be the Contractor's responsibility. Continual immediate visual routine monitoring of the behaviour of the mix at the time of placement shall be used as the basis for adjustments.

4.85 In an article '*RCC Construction & Quality Control for Burnett Dam*' by Dr Schrader, Mr Herweynen, Mr Griggs, and others, written when the Dam was nearing

completion, the test results of the RCC mix design were described as having indicated:[156]

- *The good performance of lean RCC mixes when they contained cement only, with no ash or slag.*
- *The poor performance of lean mass mixes when they contained fly ash.*
- *The 14 day accelerated cure results provided a good indication of the approximate 1 year strength for the lean mass mixes.*
- *The mix with a minimum cement content of 60 to 65 kg per m^3 of RCC complies with the strength requirements (based on calculated stresses in the dam) and is a practical and workable mix for the dam construction.*

4.86 LCRCC was considered attractive to the Alliance because it was the '*most economical mix*'.[157]

Further changes to the RCC mix

4.87 At the start of construction, the cement content was reduced from 65 to 63 kg/m^3 '*to better reflect the precision at the pugmills (making it range from 61 to 65 kg/m^3, approx.)*'.[158] The Final Design Report gave details of the final RCC mix designs:[159]

Table 6-1: RCC mix designs for Burnett Dam

Material	SpG (t/m^3)	MIX I-A	MIX I-B	MIX II	MIX III
		kg/m^3			
Cement	3.15	63	63	85	150
Ash	0	0	0	0	0
Water	1	120	129.1	129.2	132.5
Air		1.00%	1.00%	1.00%	1.00%
Site Agg		2,317	2,292	2,273	2,208
SUM		2499.8	2.726	2,487	2,490
Unit Wt	2.73	2,500	2,484	2,487	2,490
TAFD		2,525	2,509	2,512	2,515

Note: considering aggregate with SpG = 2.73 t/m^3 and abs =1.57%

4.88 Mix I-A was used for the trial section, monolith P in the secondary spillway, the main spillway at the foundation, and for the trial section of levelling layers for the aprons. Mix I-B was used for the remainder of the secondary spillway, the main spillway, the non-overflow section and levelling layers for the aprons (except for the trial section).

156 Exhibit 75, **PDI.037.0001**, .0002.
157 **DNR.020.019.0924**, .0924.
158 Exhibit 25, **DNR.001.0267**, .0292.
159 Exhibit 25, **DNR.001.0267**, .0293.

Mix II was used for some levelling sections on the secondary spillway aprons, while Mix III was used for the last layer on the main spillway and secondary spillway aprons.[160]

4.89 A mix of 60 to 65 kg/m^3 was at the very lean end of the RCC spectrum. Mr Dolen's evidence was that of the 906 RCC dams listed on the '*RCC Dams Database*' maintained by Malcolm Dunstan & Associates,[161] only 12 have a cementitious content less than 65 kg/m^3 and none less than 60 kg/m^3.[162]

4.90 The Specification referred to a '*modified*' VeBe time of about 20 to 30 seconds at a temperature of 20ºC. Vebe tests are a standardised method for assessing the consistency and workability of RCC.[163] The test involves measuring the time it takes for a sample of fresh RCC to take on the form of a cylindrical mould under vibration and surcharge. The longer the time, the less workable the RCC is. What was meant by the expression '*modified VeBe time*' in the Specification was not explained. Mr Dolen's view was that a VeBe time in the range of 20 to 30 seconds plus would be considered high for RCC dams constructed in the 21st century.[164]

Thermal analysis and impact of delays

4.91 The stage 1 proposal of the Hydro Tasmania consortium explained that the design and construction program did not allow for any RCC placement during the hot and wet months. RCC was to be placed in '*two seasons*' with completion of the first in November 2004 and the second in September 2005.[165] The proposed program incorporated a break in RCC placement from the start of December 2004 to the end of March 2005.[166] That break seems to have been designed to manage constructability issues with RCC placement. One type of those issues was '*ambient temperature constructability issues*', including the possibility of needing '*to cease placement until temperature falls*'. A preliminary management strategy identified during the tender phase was to '*[p]lace exclusively in winter*'.[167]

4.92 The programming of RCC placement activities impacted the analysis of the thermal performance of the different RCC mixes that were tested during the design phase. The thermal analysis was summarised in the '*Method Statement for RCC Overview and Trial Mix Specifications*' in the following terms:[168]

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160 Exhibit 25, **DNR.001.0267**, .0292.

161 Database of all RCC dams throughout the world maintained by Malcolm Dunstan & Associates, <http://www.rccdams.co.uk/>.

162 Exhibit 104, **GHD.006.0001**, .0010.

163 ASTM C 1170 – 91: Standard Test Methods for Determining Consistency and Density of Roller-Compacted Concrete Using a Vibrating Table <https://civilengineersstandard.com/wp-content/uploads/2018/11/C-1170.pdf> accessed on 21 April 2020.

164 Exhibit 104, **GHD.006.0001**, .0010.

165 Exhibit 251, **HYT.510.004.0001**, .0105.

166 Exhibit 251, **HYT.510.004.0001**, .0126.

167 Exhibit 251, **HYT.510.004.0001**, .0081.

168 Exhibit 22, **DNR.010.8266**, .8269 to .8270.

The thermal analysis was performed for the specific material properties of the … mix with 65 kg of cement and no fly ash. … The 65 kg mix is stressed to 58% of its strain capacity at the foundation and less than 50% of its strain capacity from a few metres above the foundation to the top of the dam. …

In order to be accurate, the thermal study must start with a very detailed and accurate model of the construction schedule, but it must also use dependable and detailed material properties. …

A requirement of the thermal analysis was to accurately model the time of placing of each RCC layer, and proper evaluation of the probable natural RCC placing temperature if no forced cooling is used. The placing temperature obviously will be higher for aggregates made and stockpiled in the hot part of the year, and cooler when aggregates are made and stockpiled in the cool time of the year.

Factors that were taken into account in determining the Schedule tor Construction ... :

- *The type of equipment (crushers, mixers and trucks or conveyors).*
- *The size of equipment (crushers, belt or truck).*
- *Average proven efficiency for the equipment with RCC.*
- *Days worked per week.*
- *Hours worked per day.*
- *Efficiency of crews working different schedules.*
- *Start-up inefficiency.*
- *Impact of obstacles such as gallery (if used).*
- *The rate at which forms or panels can be erected.*
- *The rate at which membrane can be welded.*
- *Moving from one location to another location.*
- *Constructing the Non-overflow wall faces at the abutment.*
- *Holidays.*
- *The reduction of placing rate in confined areas as the dam gets narrow (this has much more impact on truck delivery than with conveyor delivery).*
- *The effect of rainfall (much worse with trucks than for conveyors).*

All of these factors were taken into account in the thermal analysis, using proven detailed procedures specifically developed for RCC, and also using the site specific … detailed construction schedule. In order to achieve this, a day-by-day analysis was needed for both aggregate and RCC production.

A work schedule of 2 x 10 hour shifts per day, seven days per week, with crews working for ten days straight (then five days off), with three rotating crews was selected as best for the project.

The best overall solution for schedule, thermal stress, cost, risk and quality of RCC is a 30 inch belt all conveyor proven system by Rotec, fed by two Aran continuous mixers. The selected method also allows completion of the project soon enough to provide the opportunity to commence filling the reservoir in the 2004/05 wet season.

The conclusion of the thermal analysis was that an RCC mix with 65-75 kg/m^3 of cement does not require forced cooling. Team 1 has adopted an RCC mix with 70 kg/m^3 of cement for the [Target Cost Estimate].

4.93 Even before construction began on site, there were significant moves away from the construction program contemplated during the tender phase. By 18 January 2004, Dr Schrader understood that RCC placement in the main spillway was planned to be completed by 16 September 2004, which was a delay from planned completion on 22 August 2004. In a memorandum to Mr Herweynen and Mr Johnson, Dr Schrader provided an update on estimated material properties of RCC mixes being tested and on thermal stresses of mixes with 60, 65 and 75 kg/m^3.[169]

4.94 Dr Schrader's memorandum explained that, because average ambient temperatures in September were only 1ºC greater than in August, thermal stresses and strains were only slightly greater. However, if the scheduled slipped into October, peak temperatures would probably be 4 to 5ºC higher. Thermal stresses should still remain tolerable whether or not it was an '*unusually warm or cold year*'. The memorandum went on to say:[170]

In addition to temperature issues, a delayed schedule will push work into months with more rain and more lost time. The following table provides a short summary of the impacts of delayed schedule on temperature and rainfall.

	June	*July*	*August*	*September*	*October*	*November*
Shifts Lost to Rain	*5*	*4*	*4*	*4*	*6*	*7*
Average Ambient	*15*	*14*	*15*	*16*	*20*	*22*

169 Exhibit 146, **DNR.011.1361**.
170 Exhibit 146, **DNR.011.1361**, .1366.

4.95 By February 2004, the '*baseline construction program*' scheduled RCC placement in the:[171]

a. primary spillway from 27 July 2004 through to 3 November 2004

b. left abutment from 5 November 2004 to 8 December 2004

c. right abutment from 10 December 2005 to 15 January 2005

with impoundment planned for 16 February 2005.

4.96 At around the time the RCC mix design process was completed, Dr Schrader sent a memorandum dated 24 May 2004 to present the thermal analysis that reflected thermal strains and conditions for the delayed construction program. Of the placement schedule, Dr Schrader said:[172]

> *The current thermal schedule has RCC starting at the right abutment on 1 June, 2004, and continuing with no float to the end of all RCC placement on 16 December. Placement in the main spillway actually starts 22 July and reaches the spillway crest October 30 in the current thermal schedule.*

4.97 The updated analysis showed that thermal strains were slightly higher but still within the strain capacity of the RCC. However, Dr Schrader offered the following warnings about the program slipping into the warmer months:[173]

> *Note 1: Even though it may be acceptable to slip into warmer weather from the standpoint of thermal stress, warmer weather has a different impact that should not be overlooked. There is much less time available to deliver, spread, and compact our type of RCC in warm weather. This may lead to placing only at night on a one shift basis if temperatures get too high. Warmer weather also results in more cold joints with costly cleaning and lost time.*
>
> *Note 2: Even though it may be thermally acceptable if the placement slips from December into January-March (as a separate schedule by others indicates) thermal conditions will be worsened. More important, there will be a loss of efficiency and a new start-up after the Christmas break. This can be avoided by finishing RCC before Christmas. Most important, January-March is the rainy months. A detailed assessment indicates that there will be three times the amount of time lost due to rain compared to the dry months. In addition to having no productivity during rains, there are added costs and delays due to clean-up and extra cold joints stopping for rain if production slips into this time frame. Everyone should understand that each day test in June-September equates to about three days lost if the time needs to be made up in January March. Dragging into this time period with RCC can eat up what otherwise could be a profitable and efficient RCC project.*

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171 **DNR.011.1639**.

172 Exhibit 161, **DNR.011.1556**, .1559.

173 Exhibit 161, **DNR.011.1556**, .1558 to .1559, (with original bold type emphasis removed).

4.98 The warnings were prophetic. Rather than pausing RCC placement in the hotter and wetter months as had been planned during the tender stage, the opposite occurred. The peak period of RCC production and placement was from December 2004 to April 2005,[174] which overlaps with the planned break from December 2004 to March 2005. Instead of RCC placement avoiding the months that were expected to be warmer and wetter, that is when the majority of the work was done. Delays initially caused by unanticipated geological conditions had a '*compounding*' effect on RCC operations, which were less efficient because they were executed during the wet season.[175] That caused unnecessary extra work and productivity losses. The problems with RCC operations were summarised by Mr Hamilton in November 2004 as involving '*Delay upon delay; Vicious loop; Exacerbate cost overrun*'.[176]

4.99 The major contributors to the delayed RCC placement schedule appear to have included:

a. delays in foundation excavations due to unexpected geotechnical conditions in the foundation[177]

b. inability to attract labour to the project, including a senior experienced engineer to manage RCC[178]

c. industrial action[179]

d. delays in commissioning the Rotec conveyor.[180]

The choice of design shear strength values

4.100 Section 5.5 of the Detail Design Report was headed '*Stability Analysis*'. It set out the assumptions adopted in the analysis of the Dam's stability, including as to uplift, load types, material properties and acceptance criteria.[181] Material properties were stated in section 5.5.4. Tables 5-4 and 5-5 set out the friction angle (tan ϕ') and cohesion (c') under static loading for untreated lift joints and lift joints treated with bedding mix. The friction angle and cohesion parameters were different depending upon whether the lift joint quality was '*poor*', '*good'* or '*excellent'*.[182]

174 Exhibit 38, **SUN.110.003.0001**, .0030.
175 **SUN.018.019.6859**, .6882
176 **SUN.018.019.6859**, .6882.
177 **SUN.018.014.1152**, .1152.
178 **SUN.018.014.1152**, .1152.
179 **SUN.018.014.1152**, .1155.
180 **SUN.018.014.1152**, .1155.
181 Exhibit 24, **GHD.002.0001**, .0137 to .0141.
182 Exhibit 24, **GHD.002.0001**, .0141.

Table 5-4: Untreated Lift Joints (Static Loading)

SHEAR STRENGTH PARAMETER	PROBABLE VALUE	LIFT JOINT QUALITY INDEX		
		Poor	Good (Adopted Values)	Excellent
Friction angle, Tan ϕ'	1.1	0.7	**0.85**	1.0
Cohesion, c′	500 kPa	250 kPa	**325**	400 kPa

Table 5-5: RCC Mass & Lift Joints with Bedding Mix (Static Loading)

SHEAR STRENGTH PARAMETER	PROBABLE VALUE	LIFT JOINT QUALITY INDEX		
		Poor	Good (Adopted Values)	Excellent
Friction angle, Tan ϕ'	1.2	0.8	**0.90**	1.0
Cohesion, c′	3600 kPa	2000 kPa	**2600 kPa**	2800 kPa

Dr Schrader's advice about shear strength values

4.101 Tables 5-4 and 5-5 were prepared by Mr Griggs with heavy reliance on the input of Dr Schrader. Dr Schrader advised the Alliance on the design shear resistance values to use. During stage 2 of the design and based on the results of the RCC mix testing to that point, Dr Schrader had advised the Alliance on probable shear strength parameters based on an RCC mix with 65 kg/m^3 of cement.[183]

4.102 Relevant to how Dr Schrader arrived at the recommended design inputs, a document titled '*Method Statement for RCC Overview and Trial Mix Specifications*' (the date of which is not clear) said:[184]

> *The testing program has been designed to provide key data specific to the materials intended to be used at the … Dam project. This will not only provide direct results for design but, more importantly allow correlation with data from a large number of past RCC projects to which [the Hydro Tasmania consortium] has access. This will allow accurate prediction of all necessary design parameters.*

4.103 Dr Schrader explained that he had used an orthodox approach to deriving design values for the Dam, which involved the process of correlation referred to in the quote immediately above:[185]

> *[T]ypical recommendations of industry guides and codes to use 45 degrees as a base assumption for friction and some percentage or function of compressive strength as a value for cohesion are adopted. At times, and almost always for projects in which I am involved, initial values from industry guides are then adjusted by careful consideration of shear values that have been adopted and/or determined by actual tests at other projects with similar mixes, aggregates, gradations, and properties.*

183 Exhibit 88, **DNR.005.4886**, .5140.
184 Exhibit 22, **DNR.010.8266**, .8269.
185 Exhibit 109, **SCE.019.0001**, .0001.

4.104 In his statement, Dr Schrader referred to the various industry standards that supported assuming an angle of friction of 45º when there was no site specific test data.[186] That was the design angle of friction that he had recommended and that the designers adopted for an 'excellent' quality lift joint, whether treated with bedding mix or not.

4.105 During hearings, some attention was directed to whether cohesion ought to have been considered in assessing the stability of the Dam. Dr Rizzo's evidence was that he only relies on friction (and does not have regard to cohesion) when designing an RCC dam.[187] Mr Dolen's view was that it was '*impractical*' to calculate cohesion derived from the upstream portion of lift joints treated with bedding mix if the remainder of the lift joint was unbonded.[188] Dr Schrader disagreed saying that the calculation was done '*all the time*'.[189] Mr Tatro was also familiar with that approach being used.[190]

4.106 The fact that experts could not agree whether prudent engineering ought to include cohesion in stability calculations (whether at all or attributable to a treated portion of a lift) directs attention to the industry standards that Dr Schrader referred to as supporting the conservatism of the design values for cohesion that he recommended.

Industry standards relevant to Dr Schrader's advice

4.107 The **first** of the industry standards that Dr Schrader's statement referred to was the 2013 ANCOLD Guidelines, which did not exist at the time the Dam was designed. As is set out earlier in this report, those Guidelines say that common practice was to assume *c*'=0 and *ø*'= 45 for residual strength of concrete.[191] However, it cannot have been relevant to the advice that Dr Schrader gave to the Alliance given that it came into effect years later. The 1991 ANCOLD Guidelines were in effect at the time and provided:[192]

> *For roller compacted concrete (RCC):*
>
> - *ultimate tensile strength = 0 (unless shown otherwise by tests)*
> - *peak effective cohesion = 0.02 f'_c MPa (unless shown otherwise by tests)*
> - *peak effective coefficient of friction = 1.0*
>
> *It should be noted that reasonable tensile and cohesive strengths are more easily obtained by the use of an high paste content in RCC than a lean variety.*

186 Exhibit 109, **SCE.019.0001**, .0001 to .0006.
187 **TRA.500.008.0001**, .0022, ln 40-42.
188 **TRA.500.008.0001**, .0035 ln 6-13.
189 **TRA.500.008.0001**, .0032 ln 18-19.
190 **TRA.500.008.0001**, .0036 ln 3-11.
191 Exhibit 35, **ACD.001.0001**, .0030.
192 Exhibit 33, **ACD.003.0001**, .0014.

(f'_c was the characteristic compressive strength of the concrete in MPa typically at age 90 days for the dam design).[193]

4.108 SunWater reviewed the design of the Dam as part of its due diligence in around October 2004.[194] A draft report (which was never finalised) was prepared by Russell Paton who worked for SunWater at the time and was part of the due diligence team.[195] In that report, Mr Paton observed:[196]

> *The adopted cohesion value appears to be particularly high c' = 325 kPa (no bedding mortar) and c' = 2600 kPa (bedding mortar).*
>
> *The ANCOLD Guidelines (ref 3) for roller compacted concrete (RCC) allows a peak effective cohesion = 0.02 f'c (unless shown otherwise by tests) = 0.02 x 14 = 280 kPa.*

4.109 The **second** industry standard in effect at the time was: the '*Roller-Compacted Mass Concrete: ACI 207.5R-99*' reported by American Concrete Institute (**ACI**) Committee 207 (**ACI Report**). It came into effect on 29 March 1999. Contributors to that report included Dr Schrader, Mr Dolen, Glenn Tarbox and Mr Tatro, all of whom gave evidence.[197] Table 3.4 of the ACI Report (reproduced below) presented data on the shear performance of drilled cores from RCC dams. The table was said to present '*[t]ypical shear test values for parent RCC and bonded and unbonded joints*'.[198] Dr Schrader referred to data in table 3.4 as justifying the values recommended for the Dam and demonstrating that his advice was conservative. Dr Schrader stated that the table showed:[199]

a. peak friction angles ranged from 33º to 76º (where Dr Schrader said that the lower bound result was anomalous because the residual friction angle for that RCC mix was higher at 45º)

b. residual friction angles ranged from 40º to 53º (although the table shows that the highest angle was 55º)

c. peak cohesion values ranged from 586 kPa to 3,861 kPa (where both results related to joints with no bedding mix)

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193 Exhibit 33, **ACD.003.0001**, .0014.
194 Exhibit 54, **SUN.016.014.1266**, .1268.
195 **TRA.500.006.0001**, .0042 ln 43 to .0043 ln 25.
196 Exhibit 54, **SUN.016.014.1266**, .1270 to .1271.
197 American Concrete Institute, 'Roller-Compacted Mass Concrete: ACI 207.5R-99', ACI Committee 207 (1999), accessed on 12 April 2020 <http://dl.mycivil.ir/dozanani/ACI/ACI%20207.5R-99%20Roller-Compacted%20Mass%20Concrete_MyCivil.ir.pdf>, p 1.
198 American Concrete Institute, 'Roller-Compacted Mass Concrete: ACI 207.5R-99', ACI Committee 207 (1999), accessed on 12 April 2020 <http://dl.mycivil.ir/dozanani/ACI/ACI%20207.5R-99%20Roller-Compacted%20Mass%20Concrete_MyCivil.ir.pdf>, p 13 (3.2.3).
199 Exhibit 109, **SCE.019.0001**, .0002.

d. residual cohesion values ranged from 69 kPa to 1,379 kPa (with Dr Schrader describing a lower test result of 0 kPa as an 'outlier')

e. core compressive strengths from 10 MPa to 39 MPa (although the lowest compressive strength in the table was 9 MPa).

Table 3.4—Shear performance of drilled cores of RCC dams

Dam/ project	Mix type/ ID	Cement, lb/yd³ (kg/m³)	Pozzolan, lb/yd³ (kg/m³)	*w/cm*	NMSA, in. (mm)	Joint type	Age, days	Core compressive strength, psi (MPa)	Peak cohesion, psi (kPa)	Shear ϕ, deg	Residual shear cohesion, psi (kPa)	Residual shear ϕ, deg	Vebe consis-tency, sec	Bonded joints, %	Joint maturity
Cuchillo Negro	130C100P	130 (77)	100 (59)	0.99	3 (76.20)	B	750	2530 (17)	225 (1551)	58	—	—	—	—	—
	130C100P	130 (77)	100 (59)	0.99	3 (76.20)	P	750	2530 (17)	360 (2482)	52	—	—	—	—	—
	130C100P	130 (77)	100 (59)	0.99	3 (76.20)	NB	750	2530 (17)	100 (689)	62	—	—	—	—	—
Elk Creek	118C56P	118 (70)	56 (33)	1.00	3 (76.20)	P	90	1340 (9)	225 (1551)	43	—	—	21	—	—
	118C56P	118 (70)	56 (33)	1.00	3 (76.20)	B	90	1340 (9)	125 (862)	49	—	49	—	58	—
Galesville	RCC1	89 (53)	86 (51)	1.09	3 (76.20)	NB	415	2080 (14)	110 (758)	67	80 (552)	40	—	24	500 deg hr
	RCC1	89 (53)	86 (51)	1.09	3 (76.20)	B	415	2080 (14)	330 (2275)	52	70 (483)	43	—	76	—
	RCC1	89 (53)	86 (51)	1.09	3 (76.20)	P	415	2080 (14)	380 (2620)	33	95 (655)	45	—	—	—
Upper Stillwater	RCCA	134 (79)	292 (173)	0.39	2 (50.80)	NB	365	5220 (36)	450 (3103)	53	30 (207)	49	17	80	—
	RCCA	134 (79)	292 (173)	0.39	2 (50.80)	NB	545	5590 (39)	560 (3861)	76	20 (138)	53	17	—	—
	RCCA85	134 (79)	291 (173)	0.37	2 (50.80)	P	120	3870 (27)	300 (2068)	55	30 (207)	42	29	60	—
	RCCA85	134 (79)	291 (173)	0.37	2 (50.80)	NB	730	6510 (45)	440 (3034)	48	20 (138)	46	29	60	—
Victoria	113C112P	113 (67)	112 (66)	0.80	2 (50.80)	P	365	2680 (18)	280 (1931)	64	40 (276)	47	730	—	—
	113C112P	113 (67)	112 (66)	0.80	2 (50.80)	B	365	2680 (18)	230 (1586)	69	10 (69)	44	—	—	—
	113C112P	113 (67)	112 (66)	0.80	2 (50.80)	NB	365	2680 (18)	170 (1172)	62	200 (1379)	48	—	—	—
Willow Creek	175C	175 (104)	0	1.06	3 (76.20)	NB	200	—	185 (1278)	65	—	—	—	57	500 deg hr
	175C80P	175 (104)	80 (47)	0.73	3 (76.20)	NB	200	—	186 (1279)	63	—	—	—	54	500 deg hr
	80C32P	80 (47)	32 (19)	1.61	3 (76.20)	NB	200	—	115 (793)	62	—	—	—	58	500 deg hr
Zintel Canyon	125CNA	125 (74)	0	1.50	2.5 (63.50)	NB	345	1510 (10)	85 (586)	56	10 (69)	40	14	—	—
	125CNA	125 (74)	0	1.50	2.5 (63.50)	B	345	1510 (10)	200 (1379)	54	10 (69)	40	14	65	—
	125CNA	125 (74)	0	1.50	2.5 (63.50)	P	345	1510 (10)	290 (1999)	56	0	55	14	—	—

Joint type: B = bedding concrete or mortar; NB = no bedding; and P = parent concrete.

Figure 4.3 – Shear performance of drilled core in RCC Dams from the ACI Report[200]

4.110 The ACI Report states that:[201]

> *The unconfined shear strength of an unjointed section of RCC has varied from 16 to 39% of its compressive strength. The unconfined shear strength of conventionally placed concrete, as determined by direct shear tests generally ranges from approximately 20 to 25% of its compressive strength, but a conservative value of approximately 10 percent is often used in design. The*

200 American Concrete Institute, 'Roller-Compacted Mass Concrete: ACI 207.5R-99', ACI Committee 207 (1999), accessed on 12 April 2020 <http://dl.mycivil.ir/dozanani/ACI/ACI%20207.5R-99%20Roller-Compacted%20Mass%20Concrete_MyCivil.ir.pdf>, p15 (3.2.3).

201 American Concrete Institute, 'Roller-Compacted Mass Concrete: ACI 207.5R-99', ACI Committee 207 (1999), accessed on 12 April 2020 <http://dl.mycivil.ir/dozanani/ACI/ACI%20207.5R-99%20Roller-Compacted%20Mass%20Concrete_MyCivil.ir.pdf>, p15 (3.2.3).

coefficient of friction within the mass has been usually taken to be 1.0 (ø = 45 deg) for RCC if no project specific tests have been conducted.

4.111 Dr Schrader stated that friction of an angle of 45º was in line with the ACI Report.[202] He also stated that his initial design estimate for cohesion fell comfortably within the range of 16% to 39% of the compressive strength of the RCC. However, the 16 to 39% range was the ratio of cohesion-to-compressive strength for parent (unjointed) RCC. It was not appropriate to draw a comparison between that range and Dr Schrader's cohesion values for the lift joints. Instead, the ACI Report stated that, for initial design purposes, a value of cohesion of 5% of the design compressive strength (i.e. 0.05 x 14 = 700 kPa) was generally used.[203]

4.112 The design compressive strength for the RCC in the Dam was 14 MPa. Based on Dr Schrader's advice in May 2004,[204] the Alliance adopted design values for cohesion of 2,400 kPa for a 'good' quality lift joint treated with bedding mix and 325 kPa without bedding.[205] Those values are about 18% and 2% of the design compressive strength of the RCC. Dr Schrader's estimate of cohesion for a treated lift joint far exceeded the conservative assumption of 5% in the ACI Report.

4.113 The **third** industry standard referred to by Dr Schrader was an 'Engineer Manual' published on 15 January 2000 by the US Army Corps of Engineers titled '*Roller-Compacted Concrete: EM 1110-2-2006*' (**USACE RCC Manual**).[206] Dr Schrader quoted excerpts from the following passage from a section headed 'Shear strength' in the Manual in explaining the design parameters he recommended for the Dam:[207]

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202 Exhibit 109, **SCE.019.0001**, .0002.

203 American Concrete Institute, 'Roller-Compacted Mass Concrete: ACI 207.5R-99', ACI Committee 207 (1999), accessed on 12 April 2020 <http://dl.mycivil.ir/dozanani/ACI/ACI%20207.5R-99%20Roller-Compacted%20Mass%20Concrete_MyCivil.ir.pdf>, p20 (4.3.2).

204 Exhibit 164, **SUN.010.002.0356**, .0360.

205 There is some uncertainty whether the design cohesion value for a good quality lift joint treated with bedding mix was 2,400 or 2,600 kPa. The Detail Design Report uses the former value in Table 5-2 but the latter in Table 5-4: Exhibit 24, **GHD.002.0001**, .0140, .0141. Mr Herweynen said that the value was 2,400 kPa: **TRA.500.013.0001**, .0019 ln 3-8, .0068 ln 32-35. Mr Griggs said that the value was the average of cohesion for an excellent and a poor quality treated lift joint ((2,800 + 2,000) / 2 = 2,400): **TRA.500.014.0001**, .0083 ln 27-30. There are documents that state that cohesion was 2,400 kPa: see, for example, Exhibit 87, **DNR.005.4145**, .4408; Exhibit 9, **IGE.019.0001**, .0033. It seems likely that the correct design figure for cohesion of a good lift joint treated with bedding mix was 2,400 kPa and that is value assumed in this report.

206 US Army Corps of Engineers, 'Roller-Compacted Concrete: EM 1110-2-2006' (2000), accessed on 12 April 2020 <http://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-2006.pdf>.

207 US Army Corps of Engineers, 'Roller-Compacted Concrete: EM 1110-2-2006' (2000), accessed on 12 April 2020 <www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-2006.pdf>, p4-6 to 4-7 (emphasis added).

(2) Lift joint shear strength (from cores). The shear strength at the lift joints is generally the critical value for design. RCC shear strength for lift joints can be lower than for CMC and may be more variable on some projects. Cohesion varies a great deal from lift surface to lift surface, while the shear friction angle is usually quite consistent. ***Cohesion generally varies based on the amount of paste, cementitious content, and lift joint preparation and exposure****. Cohesion can be improved by correcting these problems and by application of a bedding mortar or concrete. Shear friction angle is relatively unaffected by factors affecting cohesion and is more dependent on the aggregate type and shape. McLean and Pierce (1988) found that use of ø = 45 deg for preliminary design was generally conservative, while use of c = 0.1 f'_c was unconservative, due partly to the natural variation of all strength properties. For unbedded lift joints, c / f'_c has varied from 0.03 to 0.06. For bedded lift joints, c / f'_c has varied from 0.09 to 0.15. Friction angle for bedded and unbedded lift joints has been essentially unchanged.* ***Evaluation of shear strength from cores requires caution when interpreting results since joint core recovery can vary dramatically depending on drilling and extraction procedures. Core specimens tested are invariably the best samples, while unbonded or poorly bonded RCC generally debonds during coring or extraction and is not tested further. Hence, the percent joint recovery in a core testing program must be considered when evaluating test results and determining RCC lift joint shear strength design properties.*** *This can be done by reducing the cohesion by a suitable factor representing the percent bonded lifts based on the percent bonded lift joint recovery, similar to that applied for the determination of lift joint direct tensile strength (bond). Bonded lift joint recovery has varied from 2 to 38 percent for projects with unbedded lift joints, while bonded lift joint recovery for projects with bedded joints has varied from 65 to 85 percent.* ***A preliminary design value of c = 0.05 f'_c is recommended for lift joint surfaces that are to receive a mortar bedding; otherwise, a value of 0 should be assumed. A value of ø = 45 deg can be assumed for preliminary design or for small projects, for both parent and lift joint shear strength. Design values should also take into account the expected percentage of the joint which will be adequately bonded, as indicated by the testing of cores from test sections and later from the completed structure.*** *Assumed values must be verified for final design by tests performed on samples prepared in the lab and on cores taken from test fills. At a number of RCC projects, joint shear tests, at different confining pressures, have been performed on a series of large blocks of the total RCC mixture cut from test placements compacted with walk-behind rollers or small to full-scale roller compactors. Shear strength under rapid loading may or may not behave like rapid load tensile strength. Until testing of RCC shear specimens under dynamic loading conditions has been accomplished, designers should use values of shear strength conducted using the normal load rate.*

4.114 Dr Schrader's recommended design value for friction of 45º was consistent with the Manual. However, the Manual recommended preliminary design values for *cohesion* of:

a. 5% compressive strength for a bedded lift joint, i.e. 700 kPa (0.05 x 14 MPa) compared to Dr Schrader's recommended 2,800 kPa

b. 0 kPa for an unbedded lift joint, compared to Dr Schrader's recommended design value of 400 kPa.

Dr Schrader's database

4.115 Dr Schrader's approach for developing the design values for the Dam was to rely on industry guides for starting assumptions and to adjust them in light of shear strength values from other projects with similar RCC mixes.[208] In developing shear strength values, Dr Schrader considered data from five other dams: Nordlingaalda, Mujib, Willow Creek, Urugua-I and Burton George.[209] These projects all used a design friction angle of 45º.[210]

4.116 Because cohesion can be correlated to compressive strength, Dr Schrader compared the compressive strength predicted for the Dam to other projects built with LCRCC with similar basalt aggregate, gradation, paste and workability.[211] The results of that comparison are below:[212]

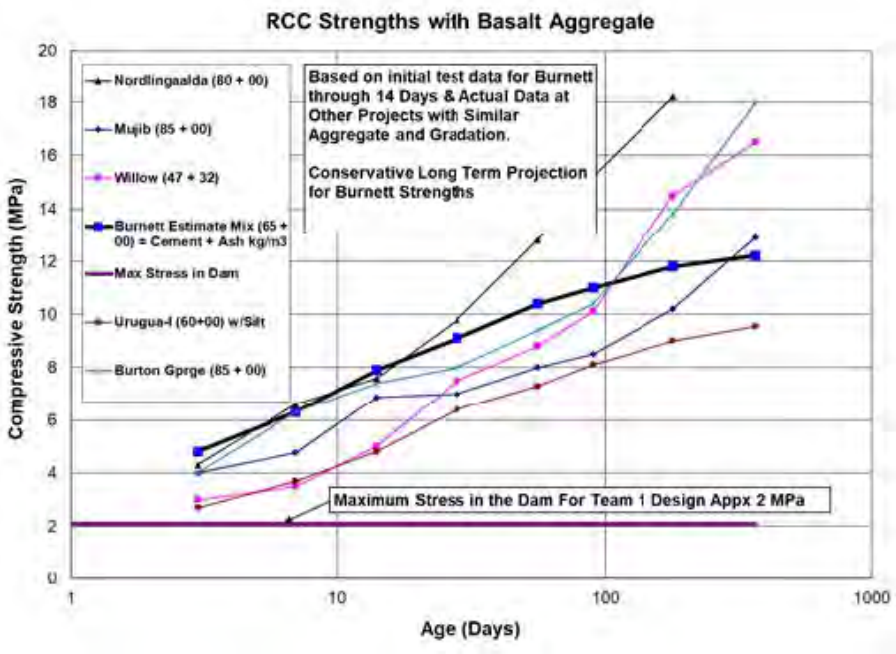


Figure 4.4 – Dr Schrader's graph of comparative compressive strengths from LCRCC dams.

208 Exhibit 109, **SCE.019.0001**, .0001.
209 Exhibit 109, **SCE.019.0001**, .0004.
210 Exhibit 109, **SCE.019.0001**, .0004.
211 Exhibit 109, **SCE.019.0001**, .0003.
212 Exhibit 109, **SCE.019.0001**, .0003.

4.117 Of the five comparator projects referred to in the graph above, Dr Schrader said that:[213]

> *Willow had actual tests performed on large shear blocks. At only 90 days the blocks with no bedding typically had results of 44 to 55 degrees for fiction angle with cohesion of 900 KPa to 1000 KPa. These were typical bi-axial tests of lift surfaces with maturities on the order of 350 to 600 C degree hours compared to the Paradise cold joint criteria … of 500 degree hours. A tri-axial test that uses three dimensional load conditions (instead of biaxial) as well as uplift pressure within the lift joint was also done. It showed expected improved friction angles of about 60 degrees. Tri-axial testing is most representative of real conditions in the dam but these tests are very complicated and almost never done due to difficulty, cost, and the fact that very few labs in the world have the ability to do concrete samples. Therefore, the more conservative bi-axial shear testing is usually done.*
>
> *Urugua-I dam (appx 88 m high, 750,000 m3) one of the first high RCC dams. It used a lean mix with typically 60 or 65 Kg of cement per cubic meter, no fly ash. It was very similar to the Paradise mix except that it had somewhat less paste and it was less workable, with larger (76 mm) coarse aggregate that had a tendency to segregate. The fiction [sic] and cohesion values at 700 Degree-hr. maturity (more than the 500 used at Paradise) and no bedding were established at 60 degrees and 900 KPa. The values adopted for design were 45 degrees fiction and 800 KPa cohesion.*
>
> *Burton Gorge dam (Australia, 30 m high) was also a lean mix (typical 85 Kg cement no ash). It used the following design basis (no tests) for shear properties of different quality joints: With bedding, cohesion 2000 KPa and fiction 46 degrees. Clean joints with no bedding, cohesion 500 KPa and friction 45 degrees. Conservative for design with no bedding, cohesion 250 KPa with friction 41 degrees. As with most smaller and medium sized dams, no shear tests were performed.*
>
> *Nordlingaalda, not yet built, used a design basis lean mix with 65 kg cement and no ash, no bedding, cohesion 310 KPa and friction 45 degrees.*

4.118 And:[214]

> *Mujib dam used similar (and worse) crushed basalt aggregate and a similar gradation to Paradise. It originally was designed and tendered as a high cementitious and high ash RCC mix but was changed to a lean mix (typical about 85 kg) with no ash because the fly ash would have to be imported, because the lean mix was judged quite acceptable (without a change in friction angle), and due to cost savings (value for money). It originally had an upstream membrane but that was eliminated after initial construction because the quality of*

213 Exhibit 109, **SCE.019.0001**, .0004.
214 Exhibit 109, **SCE.019.0001**, .0005.

lift jojnts was judged to be very good and water tight due to excellent inspection by the full time on-site RCC engineer Jose Lopez (our primary on-site RCC & QC Engineer at Paradise).

The friction angle was not changed with the change of mix type. For design basis the 1 year compressive strength of the lean mix was 10 MPa (basically similar to Paradise). The design basis estimated values used for cohesion and friction with (no bedding) were 500 KPa and 45 Degrees. With Bedding the values were 2500 KPa and 45 Degrees.

Dr Schrader's handwritten database

4.119 Dr Schrader also discussed the personal handwritten database that he maintained with shear strength test results. He considered that information at the time the estimates for the Dam values were provided.[215] That database included:[216]

> *35 proper and credible determinations of [friction] and cohesion of various RCC mixes. Typically, each determination used at least three individual large samples of the same mix and conditions, with each samples tested at a different confining load. So, the total number of samples was on the order of a hundred.*

4.120 The data was affected by a range of variables, including compaction equipment, lift joint maturities, different types of RCC mixes and aggregates, testing from cores and blocks. The friction angles recorded in the database ranged from 35º to 71º, with an average of 51º.[217] Cohesion values were between 440 kPa to 2,441 kPa, with an average of 1,125 kPa. It was not clear whether those cohesion values were for treated or untreated lift joints (or both). For LCRCC, cohesion was about 700 kPa, whereas it was about 2,000 kPa for HCRCC.[218]

Dr Schrader's advice

4.121 Based on that exercise of correlation to the database, the final estimated material properties for this Dam were provided by Dr Schrader on 29 May 2004.[219] That day, Dr Schrader sent an email to Mr Herweynen and Mr Lopez updating the material properties for mixes with 60, 65, 75 and 85 kg/m^3 of cement. Estimates were given of the '*probable*' and '*design basis*' shear resistance values for excellent quality lift joints at one year with and without bedding for a range of tested mixes.[220] The following values for the shear resistance (MPa) of a 60 kg/m^3 mix were stated:

a. for untreated (but not cold) joints:

215 Exhibit 109, **SCE.019.0001**, .0004.
216 Exhibit 109, **SCE.019.0001**, .0004.
217 Exhibit 109, **SCE.019.0001**, .0004.
218 Exhibit 109, **SCE.019.0001**, .0005.
219 Exhibit 164, **SUN.010.002.0356**.
220 Exhibit 164, **SUN.010.002.0356**, .0360.

i. probable values of '*0.5 + 1.1 N*', where 0.5 means cohesion of 0.5 MPa (or 500 kPa) and 1.1 is tan ϕ, which equates to ϕ of 47.7º;

ii. design values of '*0.4 + 1.0 N*' (i.e. ϕ of 45º);

b. for joints treated with bedding mix:

i. probable values of '*3.6 + 1.2 N*' (i.e. ϕ of 50.2º);

ii. design values of '*2.8 + 1.0 N*' (i.e. ϕ of 45º).

4.122 Although Dr Schrader also provided values for a 65 kg/m^3 mix, the design values in Tables 5-4 and 5-5 of the Detail Design Report are the ones for a 60 kg/m^3 mix.

4.123 The table in Dr Schrader's email went on to include the table that presented the '*percent of 'probable' values recommended for design without confirming full scale tests*'.[221] The table was reproduced as part of Table 6-3 of the Detail Design Report.[222]

PERCENT OF "PROBABLE" VALUES RECOMMENDED FOR DESIGN WITHOUT CONFIRMING FULL SCALE TESTS						
AGE (DAYS)	LIFT JOINT TENSION		LIFT JOINT SHEAR			
	EXCELLENT[1]	POOR[2]	COHESION		TAN PHI	
			EXCELLENT[1]	POOR[2]	EXCELLENT[1]	POOR[2]
7	40 %	10 %	40 %	10 %	80 %	65 %
28	60 %	25 %	60 %	25 %	85 %	70 %
90	70 %	40 %	70 %	40 %	90 %	75 %
180	80 %	50 %	80 %	50 %	95 %	80 %
365	80 %	55 %	80 %	55 %	95 %	80 %

4.124 Dr Schrader gave the following explanation of how the information in the table of material properties was to be used by the designers:[223]

> *[W]hen I developed the shear properties that I provided to Richard, I did find my files where I had the estimated properties and then there was a number of revisions, and I provided the shear to him, saying, 'This is probably what it is', and then underneath, I said, 'But if I was the designer, this is what I would use for a value', and it was less than what I said it probably was, which is typically what I do if I don't have any test results. So I'm very confident with 'probable', but it's an important factor, so I cut it down some.*
>
> *Then in that table of properties that I provided that had the shear, it said, 'This is for excellent conditions', and then I provided a separate table that said, 'For poor conditions, use these factors to reduce even further'.*

221 Exhibit 164, **SUN.010.002.0356**, .0361.
222 Exhibit 24, **GHD.002.0001**, .0229 to .0231.
223 Exhibit 126, **TRA.510.023.0001**, .0019 ln 36 to .0020 ln 5.

4.125 Dr Schrader said that his advice was based on the testing to which the RCC mixes had been subjected. However, no shear strength testing was done as part of the trial mix program. The shear resistance design and probable parameters were not supported by objective verification. He also said that the cohesion and friction values were based on information from a database that contained information from other projects around the world with similar kinds of mixes.[224] No spreadsheet of that description was produced to the Commission. Dr Schrader was asked when he gave evidence about the spreadsheet that underpins the Lift Joint Quality Index (**LJQI**) (discussed below). He was unable to recall what spreadsheet he had used for the Dam.[225]

4.126 Hydro Tasmania adopted Dr Schrader's recommended values as the design values for the Dam. The only exception was the percentage reduction for tan ϕ' of poor lift joints. Due to an oversight by Mr Griggs, that figure remained at 65% as had been earlier advised by Dr Schrader.[226] Adoption of that value was a more conservative approach than the 80% reduction that Dr Schrader had recommended on 29 May 2004.[227]

The Specification

4.127 The Specification for RCC construction in the Dam was a '*hybrid*' of a method statement and a conventional specification.[228] That is because RCC is a method and a product.

4.128 Section 11 of the Specification concerned RCC and (with some very small adjustments) was taken from a draft dated 27 June 2003 that had been prepared by Dr Schrader.[229] The Alliance was described as having '*unequivocally adopted Dr Schrader's Specification from the outset as a means of effectively managing the RCC placement and achieving certainty with respect to the quality and timeliness of RCC placement*'.[230]

4.129 Dr Schrader's draft, section 3.2 '*Mix Designs*' (which became, relevantly unchanged, section 11.3.2 of the final Specification[231]) stated:[232]

> *[F]riction between lift joints essentially provides sliding stability with no cohesion, and maximum tensile strength is on the order of about 0.1 MPa for minimal isolated areas, the estimated mix design given below for the vast majority of the dam is expected to result in strengths on the order of about 12 MPa at 1 year. This is mush [sic] more than required for structural reasons. The cement content*

224 **TRA.500.010.0001**, .0016 ln 2-9.
225 **TRA.500.010.0001**, .0069 ln 47 to .0070 ln 23.
226 As set out in Exhibit 88, **DNR.005.4886**, .5140.
227 **TRA.500.014.0001**, .0084 ln 3-15.
228 Exhibit 21, **DNR.003.8385**, .8445.
229 Exhibit 23, **ALC.002.001.1176**.
230 Exhibit 22, **DNR.010.8266**, .8270.
231 Exhibit 21, **DNR.003.8385**, .8446.
232 Exhibit 23, **ALC.002.001.1176**, .1176 to .1177.

can therefore be reduced. This can be done in the filed, perhaps to 65kg, if the mix remains constructible, without segregation, and it results in a suitable final in-place quality.

4.130 This statement is analysed elsewhere in this Report in the context of whether the design team (including Dr Schrader) considered that the Dam achieved stability '*essentially'* with friction alone.

4.131 The Specification itself was never changed. Revision 0 remained unaltered for the entire project. It recognised that the selection of the particular RCC mix had consequences for other things: that the methodology, placement rates, and selection of appropriate equipment were 'interrelated' with the RCC mix. The choice of LCRCC that Dr Schrader had recommended, and that the Alliance adopted, meant that the quality of its laying was critical to later being reasonably satisfied that adequate shear strength had been achieved at the lift joints.

Adoption of Dr Schrader's advice by the designers

4.132 The designers relied on Dr Schrader's advice on the appropriate design values.[233] Mr Neumaier was aware that Dr Schrader had a database '*developed on years of experience in the field of RCC, and that relate[d] to RCC dams, particularly low cementitious RCC, being built all over the world*'.[234] Mr Neumaier accepted that it would be usual for the Design Manager, although relying on others, to reach a state of personal satisfaction with the design parameters that were used.[235] Mr Neumaier could not '*definitely recall*' having reached such a state in relation to this Dam.[236]

4.133 Mr Herweynen also understood that Dr Schrader's advice was based on '*a large database of shear strength parameters'* that had been developed *'over the course of his more than 30 years' experience with RCC*'.[237] He said that Dr Schrader used a '*large spreadsheet*' to correlate previous data and to define expected shear strength values for the Dam.[238] Mr Herweynen, however, did not understand everything in that spreadsheet.[239] He did not know of any authoritative material that could have been used to verify Dr Schrader's inputs.[240] Asked by Senior Counsel Assisting whether more should have been done to verify Dr Schrader's value for cohesion, this exchange took place:[241]

> *Q. We will come to how one might calculate them and what we do. Did it cause you to be unsettled, as a designer, that you were adopting values*

233 **TRA.500.015.0001**, .0005 ln 3-12.
234 **TRA.500.015.0001**, .0005 ln 27-32.
235 **TRA.500.015.0001**, .0005 ln 44 to .0006 ln 2.
236 **TRA.500.015.0001**, .0006 ln 4-5.
237 Exhibit 244, **HER.001.0001**, .0033 [155].
238 Exhibit 244, **HER.001.0001**, .0033 [156].
239 Exhibit 247, **TRA.510.007.0001**, .0021 ln 33-34.
240 Exhibit 244, **HER.001.0001**, .0033 [157].
241 **TRA.500.013.0001**, .0022 ln 2-46.

that relied upon one man's - albeit the expert's - assessment of matters which went to the fundamentals of the shear strength of your dam?

A. That is not necessarily unusual in a dam engineering project.

Q. Do I take it that it did not, for that reason, cause you to be unsettled?

A. I was not unsettled at that time, no.

Q. Your assessment of what was unusual or not is based upon the concrete gravity dam that you were involved with?

A. That's correct.

Q. And what else, in terms of your assessment of what is usual?

A. As I said, I had done other safety reviews on concrete dams as well within our Hydro Tasmania portfolio.

Q. But never an RCC dam?

A. Not RCC, but, as I have already said, we had Dr Schrader and we had David Brett.

Q. Did you not think, because it was your first RCC dam, that there might be some problem, something special about RCC dams because of the number of layers, that meant you should independently verify, check, the values being given to you for cohesion?

A. I was comfortable at that stage to adopt Dr Schrader. He had done many RCC projects, and I felt comfortable with that.

Q. It didn't concern you at the time that you might have to sign off on the design parameters or the design intent having been achieved without having seen, other than through the spreadsheet Dr Schrader showed you and relying upon him, how these values were arrived at?

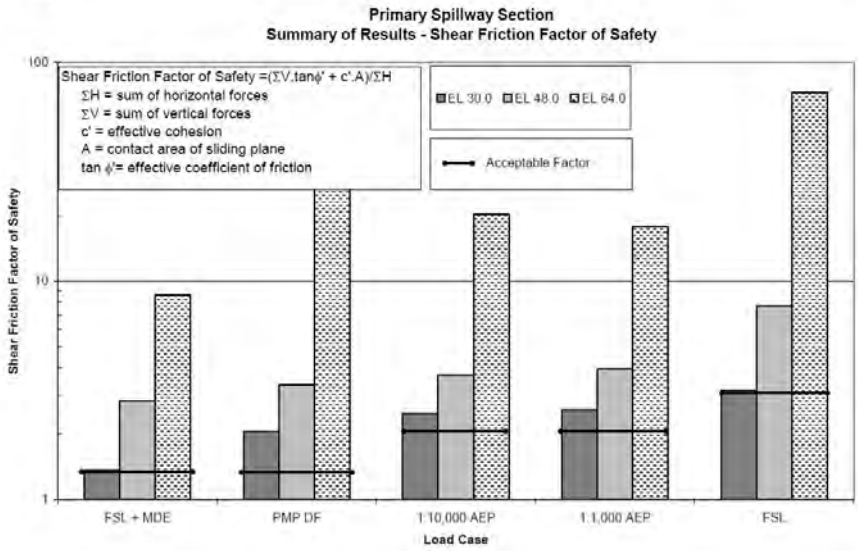
A. Well, just to be clear, there was more process than just me just adopting these particular ones. This was a basis of design that was put forward, signed off by Andreas, another person with extensive dam experience - so he signed off on these parameters. He accepted those parameters, too. It wasn't just myself.

4.134 Mr Griggs, who also relied on the advice of Dr Schrader, did not analyse or verify independently the inputs provided by him.[242]

[242] **TRA.500.014.0001**, .0084 ln 21-35.

4.135 Based on Dr Schrader's estimates, the summary of the stability analysis was presented in Figure 5-13 of the Detail Design Report, which follows.[243] More detailed results were set out in Appendix K.[244] They showed, for instance, that for the normal load case at Full Supply Level (**FSL**) with 50% uplift, the shear friction factor of safety was lowest at the base of the Dam and increased with the existing level.[245]

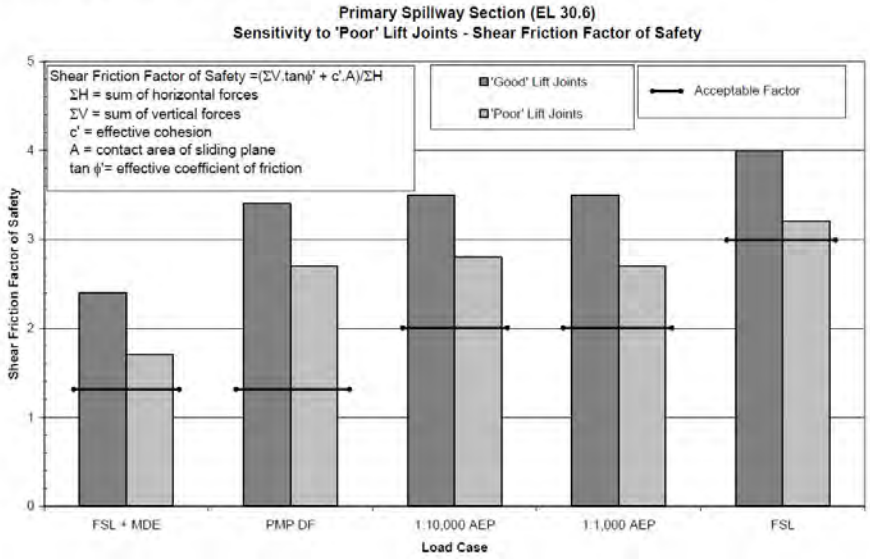
Figure 5-13: Primary Spillway – Summary of Results – Shear Friction Factor of Safety



4.136 A sensitivity analysis was performed on the stability of the Dam based on untreated poor lifts joints with a friction angle of 35º and cohesion of 250 kPa.[246] Figure 5-16 of the Detail Design Report (below) shows the results for a lift joint in the Dam at EL 30.6 m.[247] A shear friction factor of safety of 3.2 was obtained under the normal load case with the reservoir at FSL and 50% uplift.[248] That compared with a minimum shear friction factor of safety of 3.0 for the normal load case.[249]

243 Exhibit 24, **GHD.002.0001**, .0152.
244 Exhibit 24, **GHD.002.0001**, .2354 to .2369.
245 Exhibit 24, **GHD.002.0001**, .2356.
246 Exhibit 24, **GHD.002.0001**, .0141.
247 Exhibit 24, **GHD.002.0001**, .0156.
248 Exhibit 24, **GHD.002.0001**, .2358.
249 Exhibit 24, **GHD.002.0001**, .0141.

Figure 5-16: Primary Spillway – Sensitivity to 'Poor' Lift Joints – Shear Friction Factor of Safety – EL 30.6m



4.137 Dr Schrader was not the 'designer of record' of the Dam. But he was closely involved in its design, especially in respect of the shear strength parameters and properties of the RCC mix. The designers relied heavily on his advice, as he had expected.

Sliding factor of safety calculation

4.138 An assessment was undertaken by Mr Griggs on 7 November 2003[250] in which he compared the stage 2 design of the Hydro Tasmania Consortium with that of the Thiess and URS consortium. Assuming no cohesion, the Thiess and URS design was shown to achieve sliding factors of safety of 1.5 for the usual load condition, 1.3 for unusual, and 1.0 for extreme. By contrast, the design of Hydro Tasmania and SMEC achieved sliding factors of safety of 1.0 only for the usual load case and less than 1.0 for the unusual and extreme cases. Mr Griggs considered that a calculated value of the sliding factor of safety less than 1 may not be a concern, as long as bedding mix was used and the factor of safety for sliding failure (i.e. based on cohesion also) was acceptable.[251] This is consistent with the conclusion of SunWater in the Preliminary Design[252] referred to above.

Peer review of the design

4.139 The detail design was subject to peer review. The Design Management Plan required that for each design package verification was to be:[253]

> *[C]arried out by suitably qualified and independent persons, to ensure that the following requirements [were] met*
>
> *(i) All regulatory requirements*

250 Exhibit 88, **DNR.005.4886**, .5022.
251 **TRA.500.014.0001**, .0085 ln 31-34.
252 Exhibit 96, **DNR.003.7930**, .7971.
253 Exhibit 297, **SUN.162.002.0149**, .0169.

(ii) Safety considerations

(iii) Functional and operational requirements

(iv) Compatibility of materials

(v) Anticipated environmental and local conditions

(vi) Achievement of specified tolerances

4.140 In a memorandum that Mr Neumaier wrote dated 27 January 2004, the independent reviewers engaged to review the design work were listed as:[254]

a. Mike Fitzpatrick (in respect of dam engineering)

b. Patrick MacGregor (regarding geology and geotechnical engineering)

c. Eric Lesleighter (for hydraulic structures).[255]

4.141 None of those individuals considered RCC mix design or RCC-related aspects of dam stability. Mr Fitzpatrick was briefed with documents relating to RCC, including papers about shear strength and lift quality of RCC.[256] However, his report '*Burnett River Dam, Peer Review of Dam Safety, 19-23 January 2004*'[257] does not discuss shear strength aspects of the RCC design.

4.142 Mr Fitzpatrick said that the method of stability analysis was appropriate. He remarked that the results demonstrated that the criteria were met.[258] Mr Fitzpatrick's report does not offer any *critique* of the stability analysis or the shear strength values adopted in the design. There is no evidence that he did any work independently to verify the stability analysis or otherwise assess the appropriateness of the inputs. The report presents a bare conclusion rather than a reasoned assessment on that topic.

4.143 Mr Neumaier and Mr Herweynen both testified that Jack Linard had conducted the peer review of RCC aspects of the design.[259] Mr Neumaier was asked by Counsel Assisting about Mr Linard's involvement. This exchange ensued:[260]

Q. Who was the peer reviewer for the RCC mix design, to your knowledge?

---

254 Exhibit 232, **DNR.010.0929**, .0929. The brief described the peer reviewers' work as '*Each reviewer is required to prepare a review report stating the issues which have been covered by the review and provide recommendations of how, in their opinion, the proposed design could be improved*'.

255 The peer review of hydraulic structures is considered in Chapter 6.

256 Exhibit 303, **DNR.020.021.6529**, .6632.

257 Exhibit 232, **DNR.010.0929**, .0937.

258 Exhibit 232, **DNR.010.0929**, .0941 to .0943.

259 **TRA.500.015.0001**, .0006 ln 15-33; **TRA.500.013.0001**, .0023 ln 23.

260 **TRA.500.015.0001**, .0006 ln 15-33.

A. That would have been Mr Jack Linard.

Q. Do you ever recall seeing from him any document which expressed the results or analysis that he undertook as part of his peer review?

A. I recall having discussions taking place between Mr Linard and Mr Schrader and Mr Herweynen on the merits of this particular type of RCC. Does that answer your question?

Q. It does in part, thank you. Is it possible, I want to suggest to you, that Mr Linard was in fact the peer review with respect to design and not RCC mix design specifically?

A. Well, the boundaries are fluid. Mr Linard is a very well-known dam and RCC dam expert in his own right, and it would have been out of character for Mr Linard to confine his review to things that are other than RCC.

4.144 Mr Neumaier's evidence was that it would have been out of character for Mr Linard not to review RCC design issues. His evidence falls short of demonstrating that Mr Linard did check. Moreover, the documentary evidence indicates that Mr Linard's review was restricted to foundation excavation and preparation, foundation grouting, diversion works and inlet/outlet works.[261] The Detail Design Report refers to Mr Linard as having provided '*[a]ssistance with the design of the outlet works and some of the RCC dam construction aspects*'.[262]

Dr Schrader's attendance at peer review workshops

4.145 Peer review workshops and meetings took place during January 2004. Mr Neumaier said that Dr Schrader '*would not*' have been a member of the peer review panel because that would be a conflict.[263] Peer review was supposed to be independent.[264] The documents refer to Dr Schrader as a peer reviewer who was involved in workshops including to review the RCC mix design.[265] A report on the peer review workshops was prepared by K D Murray and stated that:[266]

> *During the week of 19 to 23 January 2004, the Alliance carried out a Peer Review of aspects of the Burnett River dam design and construction. The reviews consisted of a number of formal workshops and meetings. SRD Consulting facilitated each workshop. Minutes of all workshops and meetings were taken.*

261 Exhibit 232, **DNR.010.0929**, .0958.
262 Exhibit 24, **GHD.002.0001**, .0018.
263 **TRA.500.015.0001**, .0006 ln 47 to .0007 ln 11.
264 **TRA.500.015.0001**, .0010 ln 14-17.
265 Exhibit 305, **SUN.018.005.2929**, .2929.
266 Exhibit 305, **SUN.018.005.2929**, .2929.

The following peer reviewers were in attendance during the week:

Ernie Schrader

Jack Linard

Mike Fitzpatrick

Patrick McGregor

4.146 Within K D Murray's report, the RCC mix design workshop was summarised.[267] That account is consistent with another document '*Expert & Peer Reviews Workshop, RCC – Mix Design & Laboratory*', which records Dr Schrader as an active participant in the RCC mix design peer review.[268]

4.147 Dr Schrader was probably inaccurately described as a '*peer reviewer*' in the summary prepared by K D Murray. More likely, Dr Schrader participated in peer review workshops but did not '*peer review*' his own work.[269]

4.148 Even if it is accepted that Dr Schrader was not a '*peer reviewer*', the intended output from a peer review process is independent verification that the Dam's design (i.e. the fundamental aspects of design) met, among other things, safety considerations. There is utility in the main designer or advisor attending a peer review workshop. That is likely to facilitate informed discussion about the design under scrutiny. However, it is a different matter if the designer or advisor's attendance affects the integrity of the process, intended (as it is) to produce independent verification.

Peer review of RCC mix design

4.149 Mr Brett was involved as a peer reviewer of the RCC mix design. The record of the workshop on 19 January 2004 shows that Mr Brett '*raised the economic benefit of admixtures*' and that Dr Schrader did not support using them.[270] Dr Schrader's position prevailed.

4.150 On 28 January 2004, Mr Brett wrote a memorandum to Mr Herweynen about admixtures. Mr Brett said:[271]

> *I realise that decisions have been made and I am comfortable with that if the decisions have been made on economic grounds. I don't necessarily disagree with the final outcome. However I do think that the case for alternative cements has not been fairly put or heard, particularly when Sunstate were asked to pay $10,000 for a test program and the results were not even considered.*

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267 Exhibit 305, **SUN.018.005.2929**, .2930.
268 Exhibit 304, **GHD.043.0001**, .0005.
269 **SMC.001.0001**, .0010 [21]; **HYT.008.0001**, .0017 [42]-[43].
270 Exhibit 304, **GHD.043.0001**, .0005.
271 Exhibit 266, **DNR.020.019.1037**, .1043.

4.151 Saying that he did not '*necessarily disagree with the final outcome*' might be viewed as a weak endorsement of the final mix design. It does not equate to an independent verification.

4.152 In a memorandum of 28 January 2004 that recorded the decision to use a mix without admixtures, Mr Herweynen said there was '*no doubt that Ernie Schrader [felt] stronger about not adopting a blended cement for Burnett Dam then David Brett does*'.[272]

4.153 Not long after the peer review workshops, Mr Brett was moved into a '*secondary role*' on the project.[273] At the time, he had expressed concern about whether the Alliance had given sufficient consideration to using admixtures in the RCC mix. It is not clear whether, had he been asked, Mr Brett would have provided written verification in accordance with the Design Management Plan requirements. It is possible that he would have. He may have taken the view that there was little point in disagreeing with Dr Schrader. This sense of resignation is suggested by an email of 1 April 2004. Mr Brett, in expressing certain reservations about trials that Dr Schrader had proposed, wrote, '*I suspect no point arguing though, unless you want to use lots of CAPITALS and !!!!!!!!!!! too*'.[274]

4.154 Dr Schrader wielded a measure of influence over the design team. Mr Herweynen said:[275]

> *[W]e obtained RCC experts - at the very start of this project, we had three RCC experts on board. … Dr Schrader, by the end, became, you could say, the expert of the experts.*

4.155 Dr Schrader participated in the review workshop as the designer of the RCC mix. That may not be a problem in principle. It may make good sense to have the designer in attendance at a peer review workshop to explain the rationale of the approach taken and assumptions adopted. However, given Dr Schrader's influence, it is not known whether his presence discouraged the workshop attendees from engaging in healthy sceptical debate about his RCC mix.

4.156 All the peer review workshops were facilitated by SRD Consulting. Using an independent facilitator is one possible strategy for trying to provide balance to the review process. Another approach would be to exclude the person whose design is being reviewed from the workshop once any questions have been asked and answered. As Mr Neumaier said, Dr Schrader had a conflict. The natural human inclination is to defend a stated position and it is understandable that Dr Schrader would probably have sought to do the same thing. Therefore, to encourage wholesome debate about the appropriateness of Dr Schrader's design, he might have been excused from the workshop when that became the focus of it.

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272 Exhibit 266, **DNR.020.019.1037**, .1038.
273 Exhibit 244, **HER.001.0001**, .0032 [148].
274 Exhibit 273, **DNR.020.019.1005**, .1014.
275 **TRA.500.013.0001**, .0009 ln 13-30.

4.157 Serious consideration needed to be given to the use of admixtures in the RCC. Using admixtures had been identified during the tender stage as a strategy to manage constructability issues with segregation, compaction difficulties, and inter-layer cohesion.[276] It is possible that further investigations of RCC mixes using blended cement might have demonstrated that the constructability advantages it offered justified its use on the project. However, there is no certainty that further investigations would have had that result. That is because of the results of testing on the trial mixes on 22 January 2004 that included only the following:[277]

a. a blended mix with 40% cement and 60% slag

b. a blended cement with 75% cement and 25% flyash

c. a straight general purpose cement mix.

4.158 The results of Vebe test results on each of these mixes were close enough to conclude that the field performance of each would be similar.[278] That was one of the expressed bases upon which Mr Herweynen decided that there was no benefit in using a blended cement.[279]

4.159 In summary, the RCC mix design peer review was deficient because there appears to be no document recording that Mr Brett, following his independent review, verified that the RCC mix met all requirements including safety considerations, functional and operational requirements, and compatibility of materials (as the Design Management Plan required of the design verification process[280]).

Peer review of the Dam's stability

4.160 Aside from Mr Brett's involvement with RCC mix design, no other RCC-related aspects of design appear to have been subject to peer review. In particular, the design strength parameters on which Dr Schrader advised the Alliance and that were fundamental to the Dam's sliding stability were not scrutinised by an independent expert in RCC. This was despite there being appropriately qualified people who might have performed that task, including:

a. Mr Tarbox, who was later involved with the Dam as a member of the Technical Review Panel that reviewed GHD's recent work.

b. Mr Forbes, whose experience included working on Kidston Dam (the first dam built from RCC in Australia),[281] working with SunWater to pioneer RCC testing in Australia,[282] and writing about RCC technologies, including improved methods of

276 Exhibit 251, **HYT.510.004.0001**, .0081.
277 Exhibit 266, **DNR.020.019.1037**, .1047.
278 Exhibit 266, **DNR.020.019.1037**, .1048.
279 Exhibit 266, **DNR.020.019.1037**, .1038.
280 Exhibit 297, **SUN.162.002.0149**, .0169.
281 **TRA.500.002.0001**, .0003 ln 41-46.
282 Exhibit 48, **TRA.510.025.0001**, .0021 ln 18-23; **TRA.500.002.0001**, .0008 ln 38-41.

construction to aid in bonding, by the time that Paradise Dam was being designed.[283]

c. Mr Linard, who was retained by SMEC and had previously worked with SMEC on projects involving RCC.[284] Although Mr Linard reviewed some aspects of the design, he was not shown (on the documents before the Commission) to have reviewed Dr Schrader's advice about shear strength design values.

4.161 Both SMEC and Hydro Tasmania relied upon Mr Linard's involvement and submitted the peer review process was appropriate.[285] Mr Neumaier's evidence was that Mr Linard was the peer reviewer for the RCC mix design.[286] When asked if he recalled seeing a document from Mr Linard that expressed the results of a peer review, Mr Neumaier recalled only discussions about the merits of the LCRCC mix and said it would have been out of character for Mr Linard not to review RCC aspects of the design.[287]

4.162 Even had Mr Linard discussed RCC-related aspects of the design with members of the design team, that is well short of the manner in which all other peer reviewers documented their views and conclusions. A written, signed document clearly marked as a review is engineering good practice. Such an approach ensures the reviewer's opinions are recorded, and that the author can be held accountable for them.

4.163 Hydro Tasmania submitted that it was '*particularly noteworthy*' that K D Murray's summary of the RCC mix design workshop recorded that '*it is my recommendation that in accordance with Ernie Schrader's recommendation slag cement and flyash not be used*'. The person making '*my recommendation*' was said to be Mr Linard.[288] That is improbable. The minutes of the RCC mix design peer review workshop on 19 January 2004 do not record Mr Linard as attending. The initials of attendees are '*ES,SJ,RF,AD,DB,RH,AN*'.[289] Moreover, it is a reasonable inference that Mr Linard, if he had been a peer reviewer of the RCC mix design, would have produced a written review. Mr Linard's written review did make mention of RCC but only to the extent that RCC related to the particular aspects of the Dam's design that were the subject of his review.[290] He did not address any aspect of the Dam's sliding stability, for example.

4.164 The Dam's stability assessment (and the associated shear strength values of the RCC adopted in the design) were of fundamental importance to the Dam's safety. A suitably qualified and independent person was required to ensure that safety considerations were addressed and all requirements met.[291] There was no peer

283 Eg. Exhibit 307, **SME.001.0001**.
284 Exhibit 244, **HER.001.0001**, .0011 [43](c).
285 **SMC.001.0001**, .0009 [19]; **HYT.008.0001**, .0016 to .0017 [40]-[41].
286 **TRA.500.015.0001**, .0006 ln 15-17.
287 **TRA.500.015.0001**, .0006 ln 22-33.
288 **HYT.008.0001**, .0016 to .0017 [41(b)].
289 Exhibit 304, **GHD.043.0001**, .0003.
290 Exhibit 232, **DNR.010.0929**, .0960, .0961.
291 Other benefits of peer reviews are considered in Chapters 6 and 7.

review of the stability assessment. This meant the Alliance did not obtain independent verification from a suitably quality expert of the stability assessment which they undertook for the Dam.

4.165 Mr Neumaier was the Design Manager. His responsibilities included obtaining verification from others that the documented design complied with all relevant items of the functional specification and statutory requirements. This verification extended to ensuring an appropriate and independent peer review of fundamental aspects of the design, including in respect of RCC-related aspects of the Dam. Peer review was required as part of that verification.[292]

4.166 The peer review of RCC-related aspects of the Dam's design was deficient. To the extent that there was a review of the RCC mix design, questions remain about whether it was sufficiently independent of the project's main RCC advisor, Dr Schrader. In any event, Mr Brett did not provide written and signed verification that regulatory, safety and operational requirements had been met by the final RCC mix adopted. Moreover, the Dam's stability, including shear strength design parameters, was overlooked in such peer review of the design as was undertaken.

Lift Joint Quality Index

4.167 The LJQI assumed prominence, not least because of its relevance to the shear strength parameters that were adopted in the detail design stage by the Alliance. That is evident from the headings to Tables 5-4 and 5-5 of the Detail Design Report.

4.168 Mr Herweynen denied that the LJQI was a design input and said it was an element of quality control.[293] The LJQI was not an input to the design. However, there is a correlation between Dr Schrader's LJQI and the design parameters. Dr Schrader based the design and probable values for friction angle and cohesion on a lift joint classified as having 'excellent' quality.

Origins of the LJQI

4.169 The originator of the LJQI was Dr Schrader: he devised and maintains its datasets. Those datasets are not publicly available, with the exception of some limited data published in articles by Dr Schrader. An article describing the LJQI was published by Dr Schrader in 1999 entitled '*Shear strength and lift joint quality of RCC*' in Issue 1 of the *International Journal on Hydropower & Dams*.[294] That article is Appendix H to the Detail Design Report.[295]

292 Exhibit 297, **SUN.162.002.0149**, .0169.
293 Exhibit 247, **TRA.510.007.0001**, .0026 ln 18-20.
294 Exhibit 124, **PDI.040.0001**.
295 Exhibit 24, **GHD.002.0001**, .2206-.2215.

4.170 In the article, Dr Schrader expressed the opinion that:[296]

> *If an RCC dam were to fail from shear, or fail at all, it would probably be by sliding along one of the lift joints or layer to layer interfaces. This is the weakest part of an RCC dam.*
>
> *For the same dam cross section, lift joint shear strength is normally of more concern with lower cementitious mixes.* ***With all other things being constant, increasing the cementitious content of RCC increases the lift joint quality and strength.***

4.171 Related to those remarks are the later made points that '*cohesion is almost directly proportional to the cementitious content*'[297] and that it had been demonstrated that '*a properly designed and placed bedding or mortar mix between layers of RCC substantially improves lift joint shear strength*'.[298]

4.172 In explaining the basis of the LJQI, Dr Schrader wrote that '*[s]ubstantial testing of lift joints reported for a variety of projects provides useful data for … specific mixtures and conditions*'. Fourteen studies were cited, four of which were Dr Schrader's own projects.[299] Dr Schrader warned against the use of '*one publication or one set of results based on a set of conditions at one particular project as an absolute basis for what will occur at another project*'[300] and went on to say:[301]

> *When site-specific tests cannot be conducted, an approximate lift joint shear strength can be developed using information from other projects in combination with knowledge of the mixture, aggregates, and other materials proposed for the new project, but absolute values should come from testing of the specific mixture and conditions in question.*

4.173 This approach is suggestive of the guidelines at the time (discussed later in Chapter 5) that recommended a conservative design approach unless site specific testing warranted otherwise.

4.174 Dr Schrader's LJQI system was based on the results of tests from earlier projects, supplemented by field experience and destructive testing of trial sections. The concept was described as similar to geotechnical and geological systems which establish estimated rock mass quality and mass modulus values based on the numerous factors to be taken into consideration.[302] The article described the LJQI in the following terms:[303]

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296 Exhibit 124, **PDI.040.0001**, .0004 (emphasis added).
297 Exhibit 124, **PDI.040.0001**, .0007.
298 Exhibit 124, **PDI.040.0001**, .0014.
299 Exhibit 124, **PDI.040.0001**, .0004.
300 Exhibit 124, **PDI.040.0001**, .0004.
301 Exhibit 124, **PDI.040.0001**, .0004 to.0005.
302 Exhibit 124, **PDI.040.0001**, .0022.
303 Exhibit 124, **PDI.040.0001**, .0021.

Lift Joint Quality Index

In addition to maturity, lift joint quality depends on a number of other factors. Example test results for some of these, such as rain and surface damage, have been included in the Figures. Other factors include surface segregation, cure, surface flatness, and the RCC delivery procedure. For example, trucks tend to damage the surface through spillage, tracking, and tyre treads, whereas conveyors do not have these problems. Guidelines in the specifications for RCC projects have tried to address these issues, but most of the ultimate decisions have been left to the inspector. The problem is that the inspector is usually not a designer who understands what is needed for stability in the various parts of the dam, nor is he very concerned with the cost to the contractor for unnecessary over-inspection. Also he is not usually a materials engineer who understands the effect that different aspects of joint quality have on lift joint strength. As a result, RCC dams have been subject to multiple problems related to lift joints, ranging from insufficient quality in critical areas to 'over-inspected' non-critical areas which wasted both money and time.

4.175 A definition of the index itself then followed:[304]

Table 2 - RCC Lift Joint Quality Index (LJQI) Guidelines

	Rating	LJQI (sum of points)
1	Excellent	> +1
2	Good	+1 to -1
3	Fair	-1 to -3
4	Poor	-3 to -6
5	Vary bad	< -6

Factor	Points	Condition/Description
Surface segregation	+2	Absolutely no segregation
	0	Minor non-concerned areas with < 1 m^2 of segregation. Total areas of segregation < 0.1 per cent of lift surface. Stones partially embedded in mortar.
	-2	Areas of segregation 1 to 2 m^2. Total areas of segregation 0.1 to 0.4 per cent of surface. Stones mostly embedded.
	-7	Multiple areas of segregation. Areas of segregation > 2 m^2. Total areas of segregation > 0.4 per cent of lift. Stones typically not embedded. All segregation and suspect areas not totally cleaned/removed.
Rain	+1	No rain (or total rain protection provided).
	0	No apparent rain damage.
	-1	Some minor rain damage, but cleaned and treated.
	-4	Obvious rain damage, not fully cleaned.
	-8	Rain damage with trucks driving on surface with slurry/paste causing soft surface, not totally cleaned and treated.
Cure	+1	Surface never dries.
	0	Essentially 100 per cent moist cure except for 0-60 minute surface drying allowed for cleaning just before RCC placement. Surface re-wetted prior to RCC placement.
	-2	Frequent periods with large areas > 100 m^2 allowed to dry in hot weather.
	-4	Dry and hot when covered with next RCC.
	-7	Never cured.

304 Exhibit 124, **PDI.040.0001**, .0022.

Maturity	+12	Next lift placed before the 'set time' (approximate time allowed for compaction) of the RCC. Typically 40 minutes for dry mix, no-retarded, little pozzolan mix and warm temperature. Typically 2 to 10 hours for heavily retarded, high ash/pozzolan, cold mix with wet consistency.
	+1	Less than 30 per cent of allowed cold joint maturity.
	0	Within specifications.
	-1	Exceed specification limits by 50°C-hour without special treatment.
	-2	Exceed specification limits by 50 to 150°C-hour without special treatment.
	-5	Exceed specification limits by > 150°C-hour without special treatment.
Surface tightness and condition	+1	Uniform tight surface with no loose fine sand grains, no dozer marks, no over-compaction, no surface cracking.
	0	Generally tight surface with dozer marks and minor loose surface sand grains blown off.
	-2	Obvious areas where dozer has damaged the surface (even if re-compacted). Obvious loose sand grains partially removed by cleaning.
	-6	Obvious dozer tracks not blown clean. Obvious and extensive sand grains loose at the surface at the time of placing RCC.
Surface flatness	0	Roller drum contacts at least 80 per cent of the surface (not more than 20 per cent bridging low spots). All RCC surface contacted by roller with at least one pass.
	-2	Roller bridges more than 20 percent while going over high points, but all RCC surfaces contacted by roller with at least one pass.
	-7	Roller leaves any area of RCC surface without compaction.
Delivery	+2	All conveyor delivery system with no spillage. No contamination from return belt.
	0	All conveyor system with spillage and contamination promptly cleaned.
	-1	Truck delivery. Careful continuous cleaning.
	-4	Spillage from trucks, constant damage to surface from tyres and turns, tracking of damp material on to RCC, even with general attempts to clean.
Other	-8	Re-blading the surface after initial compaction.
	-8	Adding a thin (less than 10 cm) layer of RCC.
	-6	Major oil or fuel spill > 3 m^2 not thoroughly cleaned.
	-8	Bedding mix dries or sets before compacting RCC.

Uses of the LJQI

4.176 The LJQI was designed to be used by inspectors on site to assign a value for each lift joint indicating the relative quality of the work. That system assigns a numerical 'point' to each of the factors influencing lift joint quality. Positive (+) points are associated with better quality, and negative (-) points with lower quality. The score assigned was the sum of the points in each of the categories. Dr Schrader's article explained that if the LJQI score was greater than 0, the quality of the lift was better than the basis of design. The work was acceptable. If the sum of the points was less than 0, the lift joint had a quality worse than the basis of design.

4.177 The graphs of Figure 16 in the article show how the LJQI was used to evaluate whether (and what percentage of) design strength values had been achieved. The graphs correlate the LJQI score with the relevant percentage of the design values for tensile strength, cohesion and friction that can be expected. The example given by Dr Schrader in the article was that for a dry consistency mix without retarder, an LJQI of -2 would have '*a probable cohesion that is 90 per cent of the design basis with a probable friction angle that is 98 per cent of the design basis*'.[305]

305 Exhibit 124, **PDI.040.0001**, .0025.

4.178 Significantly, Figure 16 does not indicate the absolute values of the strength parameters of the lift joints, only the relative changes to those design values that are independently suggested if the LJQI score for the lift joint has a value other than 0.

4.179 It was intended that the designers of dams could specify the appropriate minimum LJQI in particular areas of the RCC dam that would '*result in a safe but useable structure, perhaps at lower quality than desired, with more risk of seepage or maintenance, or with a lower factor of safety*'.[306]

4.180 Even if the system was not used in its entirety, Dr Schrader considered that the LJQI provided a detailed guide to help assess the inter-related factors that dictate the quality of RCC lift joints.[307] Using the LJQI ensured that there was a record that '*every square metre of every lift'* had been inspected and by whom.[308]

4.181 It was observed by other engineers with experience in RCC that the LJQI was a useful part of a quality management system during construction.[309] However, whether it could be applied consistently was questionable because of the visual inspections evaluating the different categories required. In that respect, Mr Brigden described the LJQI as a:[310]

> *[V]ery, very useful tool, provided there was consistency, and I imagine that there would need to be some fairly intensive training of the observers on the bank as to what constituted each of these different conditions, because we can both look at a surface and both have a completely different opinion, so I'd like to see that there is some training and some consistency behind it, in which case it's probably a useful tool. I've never - I've never used anything like this, I haven't been exposed to it before, but it is* ***- it is a measure of trying to calibrate people's eyes, and in roller compacted concrete construction, the very first test is the eye****, because by the time that you have got a density result or a moisture result, you've probably placed another hundred to 500 or to a thousand cubic metres, and so it's critical that you have people as observers on construction, day and night, all the time, with very, very keen eyesight.*

4.182 The evidence shows that the LJQI was closely connected to perceptions of whether the Dam achieved its design shear strength values.[311]

4.183 An article prepared by Mr Lopez, Mr Griggs, Roberto (sometimes known as Robert) Montalvo and Mr Herweynen and Dr Schrader about the Dam described the quality control program. Two of the aims of the program during construction of the Dam were stated in the following way:[312]

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306 Exhibit 124, **PDI.040.0001**, .0022.
307 Exhibit 124, **PDI.040.0001**, .0022.
308 Exhibit 126, **TRA.510.023.0001**, .0023 ln 23-37.
309 **TRA.500.002.0001**, .0030 ln 31-35.
310 Exhibit 48, **TRA.510.025.0001**, .0033 ln 14-29 (emphasis added).
311 Exhibit 38, **SUN.110.003.0001**, .0101.
312 Exhibit 75, **PDI.037.0001**, .0009.

- *Evaluation of lift joint quality index (LJQI) for each RCC surface layer and its effects on the dam stability.*
- *Verify that RCC, bedding mix properties fulfil design parameters*.

4.184 The article went on to explain how the LJQI was used on the project:[313]

> *The lift joint quality index (LJQI) was a very important quality aspect considered throughout the construction of Burnett Dam.*
>
> *It provided guidelines related to the evaluation for acceptance of lift joints and gave a criterion to follow during the dam construction.*
>
> *The LJQI enabled decisions to be made on:*
>
> - *Required width of bedding mix to be used (according to the lift maturity)*
> - *Cleaning & treatment requirements*
> - *Overall quality assessment of the bond between two RCC lifts that define safety factors against stability of the dam*

4.185 The LJQI was used on site daily to assess the quality of lift joints. Members of the quality assurance team completed forms titled '*RCC Lift Joint Quality Index (LJQI) Guidelines – Rating*' (**LJQI Scorecard**) to document their inspections and evaluations of quality. The Specification stated that:[314]

> *The Lift Joint Quality Index (LJQI) system will be used to evaluate the acceptability of lift joints. This will assure that all workers and inspectors are using the same standard of acceptability. In general, the lift joint quality index should average about "0" for each lift in each monolith block. The Engineer may occasionally allow an average LJQI value of -3, based on the particular location, consideration of design stresses and stability for that area, and other factors[.] Lifts with a value of -3 to -5 must be evaluated by an Engineer familiar with the design requirements for that location to determine if the lift is acceptable or needs to be removed and replaced. No lift joint in any monolith will be allowed to have with an LJQI below -5*

4.186 Using the scores allocated by those inspectors, it appears that the LJQI was also used, as will be explained, to estimate the design cohesion and friction angle values achieved during construction. However, Dr Schrader and Mr Dolen were of the view that the LJQI assessments during construction were no substitute for shear strength testing in a laboratory. Dr Schrader said:[315]

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313 Exhibit 75, **PDI.037.0001**, .0013.
314 Exhibit 21, **DNR.003.8385**, .8467.
315 **TRA.500.009.0001,** .0059 ln 6-13.

> *No, I don't think it's a substitute for laboratory - for testing. What it is is either you have an estimated value or you have test results and you have a value, and it's a way, then, to say how am I going to assure that I do inspections to get that in the dam, or what quality of inspection am I going to get, so what kind of factor do I apply to reduce that estimated or tested property? I don't think this is a substitute for it.*

4.187 Mr Dolen's evidence was:[316]

> *I don't think it is a substitute for it. I think there are other sources of data available for typical construction that would give you representative numbers to use for estimating purposes if you are doing preliminary design and such. If you have a major structure or something such as this steep slope, in no way would I allow that.*

4.188 Mr Brigden agrees that the index was no substitute for shear strength testing.[317]

4.189 The shear strength of the Dam was not verified in a way that engineers could agree shows the Dam to have achieved the values used in its design stability assessment.

Subjective and complicated assessments

4.190 A number of people interviewed or who gave oral evidence considered the assessments required by the LJQI to be subjective ones.[318] Dr Schrader accepted that there was some subjectivity in the application of the LJQI by the inspector tasked with scoring a lift joint.[319] An example is the criteria against which surface flatness were assessed, which required an inspector to evaluate whether the drum of the vibratory roller:

a. bridged over the lift surface not more than 20%, while at the same time confirming that the entirety of the lift surface was contacted by the roller drum at least once (for a score of 0)

   or

b. bridged more than 20% while passing over high points, while also ensuring that the entirety of the lift surface was contacted by the roller drum at least once (for a score of -2)

   or

c. left any area of the RCC surface uncompacted altogether (for a score of -7).

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316 **TRA.500.009.0001**, .0059 ln 15-21.

317 **TRA.500.002.0001**, .0030 ln 5-21.

318 Exhibit 48, **TRA.510.025.0001**, .0033 ln 14-29; **TRA.510.015.0001**, .0014 ln 9-10; Exhibit 302, **TRA.510.021.0001**, .0011 ln 22-32; **TRA.500.006.0001**, .0036 ln 14-17.

319 **TRA.500.010.0001**, .0069 ln 37-45.

4.191 RCC production peaked during March 2005. In February and March 2005, RCC was placed simultaneously in multiple locations: the primary spillway, the secondary spillway and the primary spillway apron.[320] During that period, the following equipment was used for RCC compaction:[321]

Compaction equipment			Characteristics			Dynamic Force (kg/mm)		# of passes	Comments
Supplier	Model	Description	Operating mass (ton)	Centrifuge force Hi/Lo (kN)	Drum width (mm)	Roller	Specification		
Dynapac	CA 512 D	Single smooth drum roller	15.6	300 / 238	2,130	14.4	8.5	8-10	On site since 30 July 04
Dynapac	CC 132	Double smooth drum roller	3.3	33	1,200	2.8	3.5	8-12	
Wacker	DS 720	Rammer	70 kg	100joules/ stroke	320 mm long x 280 mm wd		800 kg/blow (1.7 kg/cm2 area)		Wacker
Dynapac	LH 700	Vibrating plate	0.768	95	660 x 1050	14.4		6-8	
Ingersoll Rand		Single smooth drum roller							
Stavostroj	VP 2400	Pneumatic Tyre Compactor	14.09	-	1,986		-	4	On site since 20 January 2,005

4.192 The RCC Inspectors would have been required to assess the quality of lift joints in the various locations, as well as observe the operations of different compaction equipment in each of those locations.

4.193 When asked how that assessment was to be made, Bruce Embery (the Construction Manager) said that an inspector would need physically to walk the lift and observe the drum of a roller across the entirety of the lift surface under consideration.[322] At least at certain times, the three drum rollers listed in the table above operated simultaneously on an RCC lift.[323] This raises the question whether the RCC Inspectors could have assessed surface flatness by observing the drum of all three rollers across the entirety of the lift while also conducting the many other checks that were required to be done by the quality assurance (**QA**) system that the Alliance established.

320 **SUN.110.001.0174**, .0235.
321 **SUN.110.001.0174**, .0236.
322 Exhibit 114, **TRA.510.018.0001**, .0013 ln 27 to .0014 ln 37.
323 **SUN.110.001.0174**, .0300.

4.194 During an interview, Ben Brampton recollected that:[324]

> *It's a pretty intense process. It's a very complex sequence of activities that need to happen in very confined time frames, and there are so many moving parts and things that could change, and you needed to manage them all the time. It was always a full-on operation. I'd never done anything like that before. Still haven't, not in that aspect, where you have heaps of different variables firing at you.*

4.195 Mr Embery did not reject the suggestion that the system was complicated and there would have been hundreds of inspection points for an RCC Inspector to sign off on each RCC lift. He said that that was '*typical of construction projects these days, unfortunately. There is too much paperwork for the wrong people'*.[325] Even had the practical challenge of assessing surface flatness across multiple RCC fronts with a range of compaction equipment been overcome, deciding whether the drum of each of the three rollers bridged more or less than 20% across high points in the surface would be a subjective evaluation dependent on the inspector's powers of observation.

4.196 Those observations aside, the LJQI made no provision for areas where obstructions and the fragility of the PVC membrane meant that the drum rollers could not be used. Instead, small compaction equipment was used in those areas, including the wacker packer and vibrating plate listed in the table above. The criteria for 'surface flatness' did not accommodate assessment of compaction in those areas, which included the upstream face, critical though it was to the stability and impermeability of the Dam.

4.197 Another example of subjective assessment is provided by the criteria for evaluating surface tightness and condition. There are only vague differences between the criteria for:[326]

a. a score of -2 where there were '*[o]bvious areas where dozer* ***has damaged the surface (even if re-compacted)****. Obvious loose sand grains* ***partially removed by cleaning'***; and

b. a score of -6 where there were '*[o]bvious areas* ***dozer tracks not blown clean****. Obvious* ***and extensive*** *sand grains loose* ***at the surface at the time of placing RCC'***.

4.198 The emphasis has been added above to point out the differences between the criteria for the two negative scores. The score allocated depended on an inspector's evaluation whether (1) dozer tracks had merely '*damaged the surface*' or, more seriously '*were not blown clean*', and (2) obvious loose sand grains were '*partially removed by cleaning*' or were '*extensive ... at the time of placing RCC*'. The distinctions are far from clear and a degree of subjectivity is inherent in deciding which of the two scores should apply.

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324 **TRA.510.015.0001**, .0005 ln 21-27.
325 **TRA.500.009.0001**, .0117 ln 5-9.
326 Exhibit 124, **PDI.040.0001**, .0022 (emphasis added).

4.199 Dr Schrader said that he had never tested the LJQI for operator dependency. He was also aware that different inspectors would look at the same lift and independently arrive at different conclusions. Dr Schrader's evidence indicated that inconsistencies in approach were smoothed by onsite training.[327] He accepted that these kinds of assessments suffered from the weakness that they were only as good as the judgements upon which the scoring was based.[328]

4.200 Dr Schrader accepted that there is subjectivity in the application of the LJQI in terms of the scoring on site.[329] He pointed out, however, that it was a preferable approach to having no structure at all to the assessment; that is, the LJQI provided a documentary framework for inspections.[330] Mr Dolen agreed that the categories in the LJQI were important for inspecting an RCC lift surface and that the LJQI served '*as a good check list for inspectors*'.[331]

4.201 Whether the LJQI could be consistently used and applied remains a question. As Mr Brigden said, the LJQI was a '*measure of trying to calibrate people's eyes*'.[332] Mr Dolen said that he was '*not comfortable with a rating index which could change from day to day, inspector to inspector, daytime to nighttime, and season to season*'.[333]

LJQI overlooks the base of the lift joint

4.202 Mr Brigden said that the LJQI was not sufficient to replace shear strength testing in terms of giving sufficient certainty about a dam's sliding stability. The LJQI looks at the top surface of a layer but not the bottom of the layer above, which if badly segregated with no paste, would be a weakness in the structure.[334] Mr Brigden said:[335]

> *It's obviously based on a rather large database that has been developed. I am not privy to that database, but in my opinion I would be reticent to assume design parameters based on trends and databases when we're talking about natural materials, particularly when particle shape comes into it, the hardness of the stone, the mineralogy of the stone, the conditions, the freeze-thaw, the degree of submergence. It's very hard to get apples with apples, in my opinion.*

4.203 In identifying that the LJQI does not assess the base of the upper layer forming a lift joint, Mr Brigden's evidence echoed aspects of Mr Dolen's, who supposed that the Dam's designers had placed weight on the values from Dr Schrader's 1999 paper.[336] Mr Foster also felt that the LJQI drove the designers' assessment of what the shear

327 **TRA.500.010.0001**, .0073 ln 4-26.
328 Exhibit 126, **TRA.510.023.0001**, .0027 ln 35-40.
329 **TRA.500.010.0001**, .0069 ln 37-45.
330 Exhibit 126, **TRA.510.023.0001**, .0023 ln 34-37.
331 Exhibit 104, **GHD.006.0001**, .0024 [88].
332 Exhibit 48, **TRA.510.025.0001**, .0033 ln 22-23.
333 Exhibit 104, **GHD.006.0001**, .0024 [88].
334 **TRA.500.002.0001**, .0029 ln 42 to .0030 ln 3.
335 **TRA.500.002.0001**, .0030 ln 12-21.
336 **TRA.500.009.0001**, .0046 ln 18-22.

strength was.[337] As Mr Dolen observed however, the LJQI did not address the full picture of a mixture that needed to be fully compacted all the way to the bottom of a lift.[338]

4.204 In giving evidence, Dr Schrader conceded that the LJQI does not take into account the bottom of the upper lift. That deficiency had not previously occurred to him.[339] The omission is a serious shortcoming because the extent of bonding between lift joints depends not only upon the quality of the top of the preceding lift but also on the quality of the lower part of the upper lift. It is between those surfaces that bonding is to develop.

Adding points for application of bedding mix

4.205 When a low LJQI score was initially obtained, the practice at the Dam was to treat the lift with bedding mix to improve the score.[340]

4.206 An article about quality control on site explained the practice:[341]

> *Each LJQI's evaluation was made before adding bedding mix on the RCC surface affected by segregation or rain. Once the bedding mix was placed on the affected RCC surface to correct these conditions, a new evaluation of the real LJQI was made.*
>
> *When LJQI decreased to -4 due to segregation on the RCC surface, bedding mix was placed on these areas, increasing the LJQI from -4 to 0.*

4.207 Records were kept of the LJQI scores before and after bedding mix was applied. The article provides the example that: '*in the primary spillway section bedding mix increased the LJQI assessment up to 25% from good to excellent*'.[342]

4.208 The manner in which points were added to reflect the application of bedding mix was not consistent throughout construction and the rationale for different approaches remains unexplained. In July 2004, an LJQI Scorecard with a score of -4 included a notation that the next layer had added bedding mix to compensate, although no adjustment was made to the score.[343] At some stage before September 2004, a decision was made to add points to the LJQI score if bedding mix was applied. The reason for that change is not documented, although the change itself can be seen in the third to sixth bi-monthly reports titled '*RCC Quality Control Report*' (**RCC QC Report**) prepared during construction. In some of those reports, the application of bedding mix was said to have increased the LJQI score from -4 to +5.[344] In others,

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337 **TRA.500.004.0001**, .0016 ln 27-36.
338 **TRA.500.009.0001**, .0046 ln 18-22.
339 **TRA.500.009.0001**, .0048 ln 1-7.
340 **TRA.500.014.0001**, .0076 ln 34-36, .0090 ln 20-23.
341 Exhibit 75, **PDI.037.0001**, .0013.
342 Exhibit 75, **PDI.037.0001**, .0014.
343 **SUN.112.003.0186,** .0187.
344 Exhibit 101, **SUN.110.002.0158**, .0207; **SUN.110.001.0001**, .0085; **SUN.110.001.0174**, .0265.

adding bedding mix increased the LJQI score from -3 to +5.[345] It is not apparent why the increase in score was different or how an increase of some 8 or 9 points was arrived at.

4.209 RCC QC Reports numbered 7, 8 and 9 contained the following table explaining the addition of points for bedding mix:[346]

Factor	Points	Conditions/descriptions	Points added
Surface Segregation	0 or -1	Minor non-connected areas with <1 m^2 of segregation. Total areas of segregation <0.1% of lift surface. Stones partially embedded in mortar.	2
	-2	Areas of segregation 1 to 2 m^2. Total areas of segregation 0.1% to 0.4% of surface. Stones mostly embedded.	4
Rain	-1	Some minor rain damage, but cleaned or treated	2
	-4	Obvious rain damage, not fully cleaned	4

Table 19. RCC treatment with bedding mix – Points added to determine actual LJQI

4.210 The addition of points for bedding mix was not consistent. The practice was not based upon anything stated in the Specification. The explanation of points that were added was not described in a consistent way in the RCC QC Reports. For example, on 8 March 2005 an otherwise -3 lift joint was increased by 4 points because bedding mix was applied.[347] This also occurred during April 2005.[348] However, in April and June 2005, 10 points were added to lifts in the primary spillway apron.[349] LJQI Scorecards in August 2005 show that only 4 points were added for lifts in the left abutment.[350]

4.211 The process of adding points to the LJQI score because bedding mix had been applied was a process of double dipping. This is evident in the criteria for the lower scores within the category for 'maturity' which received:

a. a score of 0 where the lift joint maturity '*within specifications*'

b. a score of -1 where the lift joint maturity exceeded the '*specifications limit by 50°C-Hr without special treatment*'

c. a score of -2 where the lift joint maturity exceeded the '*specifications limit by 50 to 100°C-Hr without special treatment*'

d. a score of -6 where the lift joint maturity exceeded the '*specifications limit by >150°C-Hr without special treatment*'.

345 **SUN.110.002.0279**, .0351.
346 **SUN.110.001.0949**, .1030; **ALC.001.001.0658**, .0742; Exhibit 38, **SUN.110.003.0001**, 0083.
347 Exhibit 118, **SUN.112.001.0277**. This LJQI related to the primary spillway apron.
348 **SUN.021.006.2667**, **SUN.021.005.7902** and **SUN.021.005.7935,** .7945 in areas of the primary and secondary spillway.
349 **SUN.112.001.0141**, **SUN.112.001.0133**, **SUN.112.001.0122** and **SUN.112.001.0116** all related to the primary spillway apron.
350 **SUN.112.002.0403**, **SUN.021.005.8244** and **SUN.021.005.8344.**

4.212 The negative scores were applicable to cold joints of worsening degrees; however, those points were only given if cold joints were left '*without special treatment*'. The effect of applying bedding mix was built into the LJQI criteria. Instead of a cold joint with maturity of greater than 150ºC-Hr receiving a score of -6, the addition of bedding mix meant that the lift was '*within specification*' and received a score of 0 instead. Applying bedding mix, therefore, added 6 points to the score, which accords with Dr Schrader's closing submission that six points were usually added for bedding mix.[351] This approach to scoring the maturity criterion is demonstrated by completed LJQI Scorecards. There are many examples of surfaces being given a score of 0 for maturity where the accompanying RCC Placement ITP records that a cold joint had formed.[352]

4.213 The addition of further points if bedding mix was applied doubled up on the points that were already built into the LJQI. It also masked the underlying quality issues that the LJQI was designed to detect. It is the probable reason that there appear to be no non-conformance reports (**NCRs**) raised because of a low LJQI score.

Conclusions about the LJQI

4.214 As is discussed in Chapter 3, the LJQI was referred to in the 2013 ANCOLD Guidelines. That lends support to the use of the LJQI during construction of an RCC dam. The Guidelines explain that based on the LJQI score that a lift receives, a surface treatment should be specified to '*ensure a lift joint with adequate strength to meet the design assumptions for lift tensile and shear strength*'.[353] In this way the 2013 ANCOLD Guidelines indicate that the LJQI is one part of measures to ensure the design parameters are met. The LJQI is not, in and of itself, the measure to provide such assurance. In addition, treating the lift surface may not remedy all of the construction problems that the LJQI detects. Applying a specified surface treatment will '*ensure*' that the design assumptions are met.

4.215 By allocating a number to lift joint quality, the LJQI system risks creating the impression that it is more reliable than it really is.[354] LJQI scores were not based on objective measurements or laboratory test results. With the exception of lift joint maturity (which was a measurable function of time and temperature), an LJQI evaluation depended on observations by the RCC Inspectors. Assigning a numerical value to something that is subjective tends to suggest that the value is more objective than is actually the case.

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351 **SCE.035.0001**, .0018.

352 See, for example, **DNR.020.014.5810**, **SUN.021.006.1769**, **SUN.021.006.0127**, **SUN.114.001.0057**, Exhibit 117, **DNR.020.014.4624**, Exhibit 119, **SUN.112.002.0368**.

353 Exhibit 35, **ACD.001.0001**, .0047.

354 **TRA.500.009.0001**, .0055 ln 23-39.

Trial embankment

Draft trial embankment procedure

4.216 Dr Schrader's draft Specification made provision for a trial embankment in section 19:[355]

> *Prior to 15 April 2004, and proper to the start of production RCC in the dam, the contractor shall complete an acceptable trial placement at least eight lifts high and containing at least 800 cubic meters of RCC, including each mix to be used in the dam. The location of the test section shall be approved by the owner. It may be an area of the right abutment that has very low levels of stress and does not require the same quality as the main spillway. The trial section shall include all of the techniques and materials to be used in construction of the dams such as the precast panels, joint cleaning, compaction, density testing, bedding mix, facing mix, etc. At least two conveyor transfers shall be included in the trial section, using equipment representing the equipment to be used in the dam, but the full length of conveyor necessary for the dam does not need to be used. The trial sections shall serve as a practice, training, and orientation area, and it may be used to help evaluate the practical effectiveness of various construction methods and pieces of equipment. It will also serve as a practice area for inspection. The Engineer and Contractor will closely monitor activities during construction of the test fill and provide an informal critique and review session afterward for all those involved, including supervision, inspection, engineering, and craft personnel. Within two days after placing the test fill RCC, the contractor shall excavate a trench through the RCC test section using an excavator. The work will be accomplished under the guidance of the Engineer. The contractor will have available for the Engineers use in evaluating the test section a water truck, hose, compressor, blow pipe, and two laborers with hand shovels for an estimated time of 4 hours. If it is not part of the dam, the test section shall remain in place until completion of the dam, and it may be used for additional later testing*

4.217 With reference to the specified requirement to build a trial embankment, Mr Brett prepared the '*Proposal for RCC Trial Embankment*' dated 25 November 2003.[356] The document was emailed by Mr Brett to Mr Herweynen and Dr Schrader, among others, that day.[357]

4.218 Nearly $200,000 was budgeted to build the trial embankment.[358] Owing to this expense, Mr Brett proposed that the embankment be incorporated into the Dam wall at the far end of the right abutment. The embankment was to be 40 m long and 7 m wide, tapered in height from 0 m at one end to 5 m high at the other end, with a

355 Exhibit 23, **ALC.002.001.1176**, .1200
356 Exhibit 273, **DNR.020.016.5720**.
357 **DNR.010.0359**.
358 Exhibit 273, **DNR.020.016.5720**, .5723.

volume of about 800 m^3 of RCC. The dimensions of the embankment proposed by Mr Brett are shown below:[359]

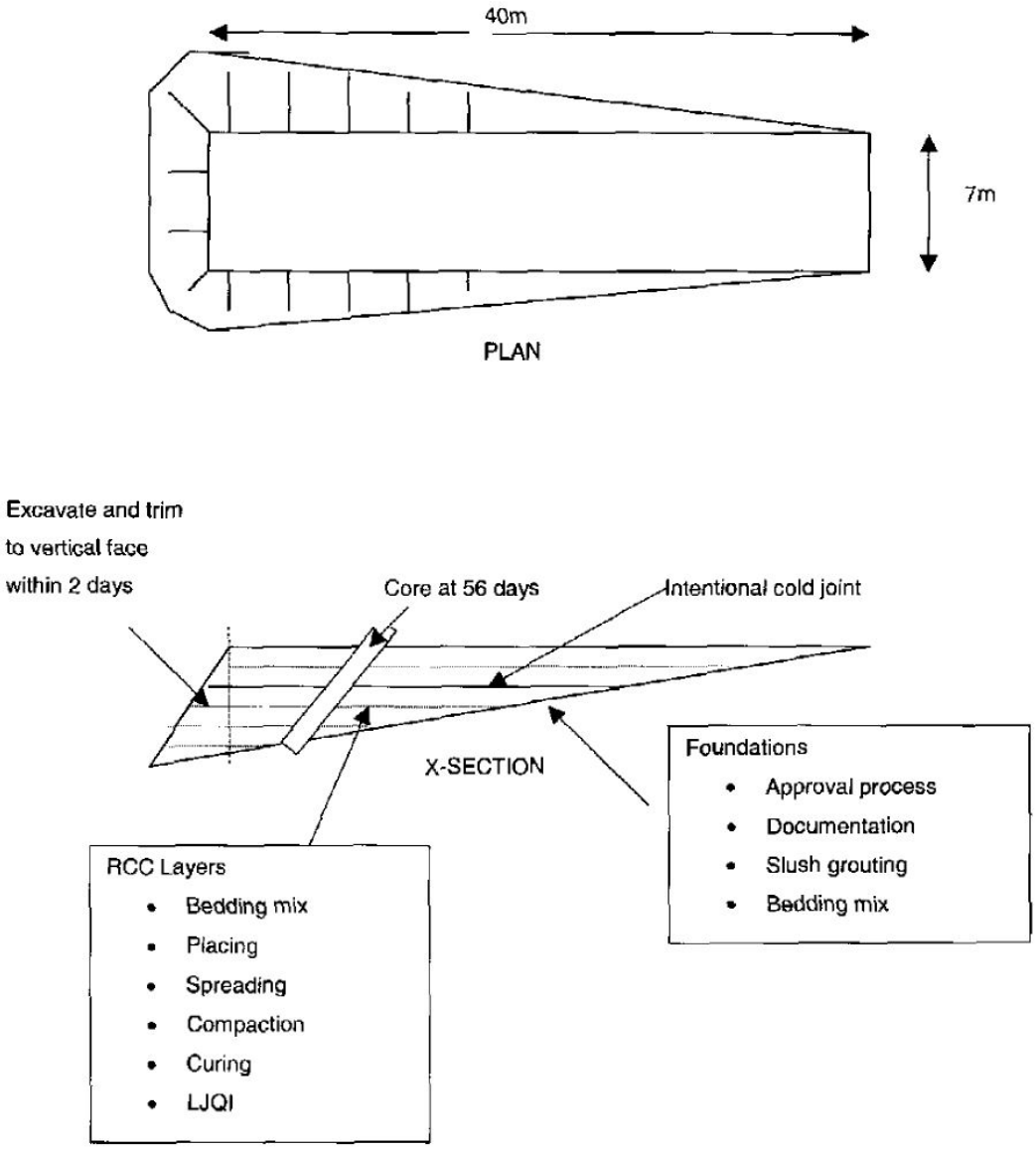

*Figure 4.5 – Trial embankment proposed by Mr Brett. (Exhibit 273, **DNR.020.016.5720**, .5724)*

4.219 Verification of the quality of RCC placement was proposed to be undertaken in two ways. First (and in accordance with Dr Schrader's draft Specification), two days after completing the embankment, the end would be trimmed to a vertical face perpendicular to the dam axis. The exposed cross section would permit visual inspection of the RCC layers. The cut face would form a monolith joint.

359 Exhibit 273, **DNR.020.016.5720**, .5724.

4.220 The second method of verification was to take a core:[360]

> *Fifty-six days after construction a diamond core will be taken through the embankment to evaluate joint bonding, compressive and tensile strengths. This work will allow confirmation of design parameters prior to placement of RCC in the critical areas of the upper dam levels.* ***This is considered to be one of the most critical aspects of the trial embankment work from a design perspective.***

4.221 Mr Brett's proposal to take a core would facilitate an evaluation of joint bonding; however, he did not recommend that shear strength testing be undertaken.

4.222 Despite Mr Brett's proposal to take a core, and his view that that task was a critical aspect of the trial, no corehole was taken from the trial embankment. By the time the trial embankment was constructed in June 2004, Mr Brett's involvement in the project had ceased.

Final specified procedure

4.223 Section 19 of Dr Schrader's draft Specification was adopted without any relevant alteration as section 11.19.1 of the Specification.[361] Those requirements (as distinct from those in Mr Brett's proposal) were followed when the trial embankment was built between 5 and 30 June 2004.[362] A report about the process – '*RCC Trial Section Construction, Report No. 1, July 2004*'[363] – summarised the experience and the test results.

4.224 The trial section was used for a number of purposes:

a. The labourers and engineers became familiar with RCC-related construction methods, including segregation control, lift joint treatment, treatment of edges of unfinished RCC layers, and treatment after rain damage.[364]

b. It was used to train site engineers in the use of the LJQI system and the procedures and tests required for quality control of the works.[365]

c. The operators of the pug mills and RCC placement foremen were provided with knowledge about how to adjust RCC moisture for different placement conditions.[366]

d. The behaviour of different mix designs was assessed to finalise the optimum mix for the Dam.[367]

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360 Exhibit 273, **DNR.020.016.5720**, .5725 (emphasis added).
361 Exhibit 21, **DNR.003.8385**, .8479.
362 Exhibit 50, **SUN.114.003.0001**, .0009.
363 Exhibit 50, **SUN.114.003.0001**.
364 Exhibit 50, **SUN.114.003.0001**, .0010.
365 Exhibit 50, **SUN.114.003.0001**, .0011.
366 Exhibit 50, **SUN.114.003.0001**, .0011.

4.225 The details of the trial embankment, including relevant construction notes, were included in the construction drawing set. Excerpts from those drawings below illustrate the general arrangement of and cross-section through the trial embankment:

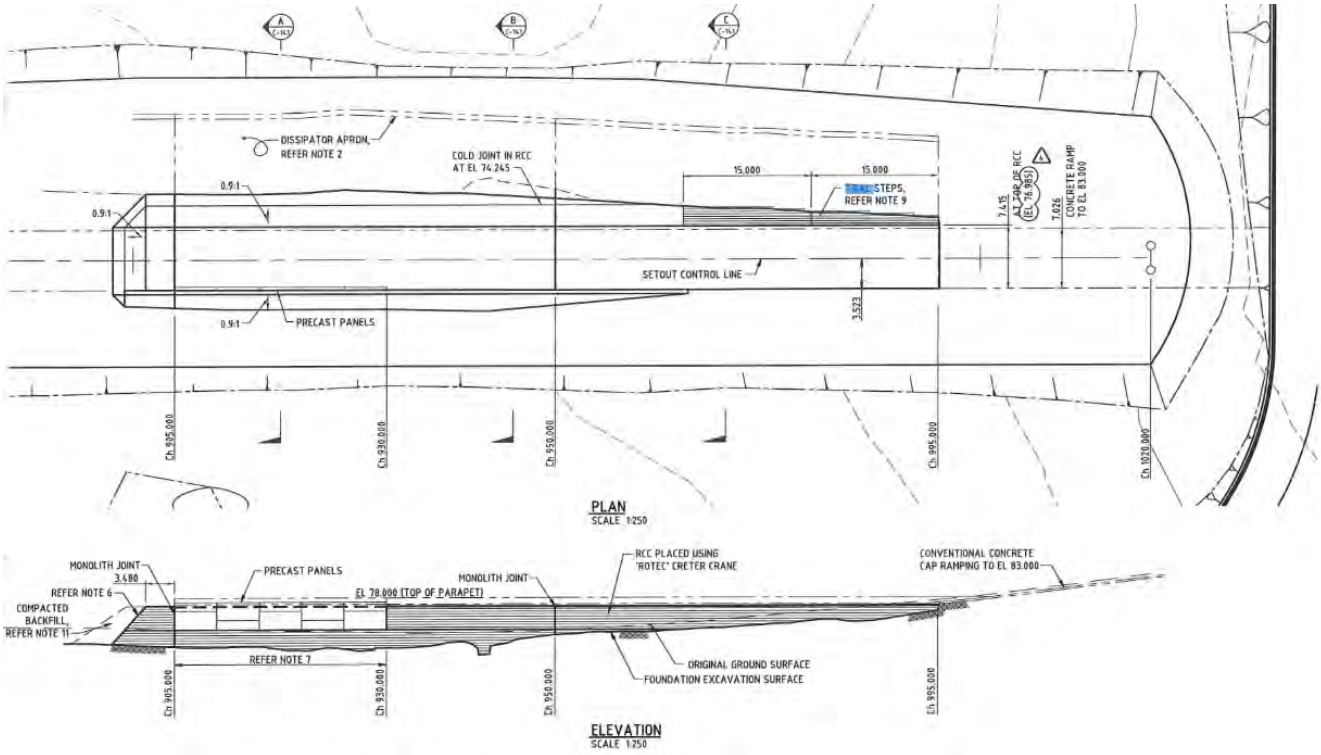

Figure 4.6 – Plan and elevation of trial embankment[368]

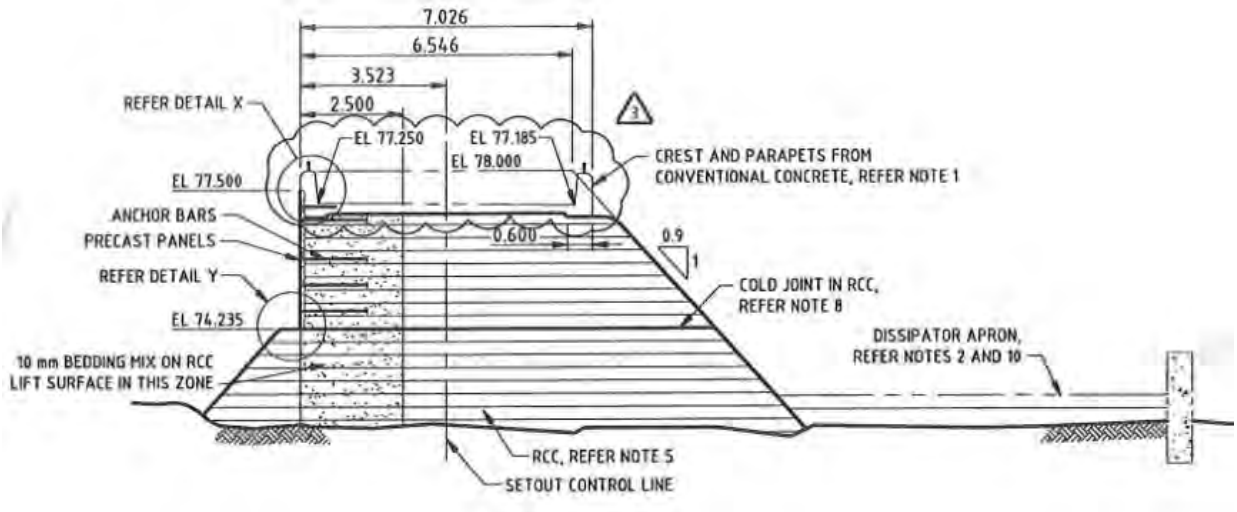

*Figure 4.7 – Section through trial embankment looking downstream. (**DNR.006.0001**, .0099)*

367 Exhibit 50, **SUN.114.003.0001**, .0011.
368 **DNR.006.0001**, .0098.

*Figure 4.8 – Overview of trial embankment during construction. (**MCM.011.0001**, .0001)*

4.226 The trial embankment was a trapezoidal section divided into two stages. The section from the foundation to EL 74.235 m (layer no. 8) had sloped upstream and downstream faces. The second section from EL 74.235 m to approximately EL 76.96 m had a vertical upstream face and a sloping downstream face.[369] A cold joint was forced at the lift between the two sections in order to trial the procedure for treatment of such joints.[370]

4.227 RCC was mixed at one of the pug mills on site and transported by 15 t trucks to an Auger max hopper located 30 m upstream of the trial embankment. The hopper loaded a mobile conveyor system (Creter crane) that delivered RCC to the trial embankment where it was placed with an elephant trunk attachment.[371] Heavy machinery was used to spread and compact the RCC.[372]

4.228 The trial embankment was used to practice a range of construction techniques including:

a. treating the surface of the RCC, including curing, application of bedding mix, and surface cleaning[373]

b. generating monolithic joints using L-shaped plastic joint formers[374]

369 Exhibit 50, **SUN.114.003.0001**, 0012.
370 **DNR.006.0001**, 0098, note 8.
371 Exhibit 50, **SUN.114.003.0001**, 0023 to .0028.
372 Exhibit 50, **SUN.114.003.0001**, .0031 to .0032.
373 Exhibit 50, **SUN.114.003.0001**, .0035 to .0038.

c. checking RCC field densities using a double probe nuclear densimeter[375]

d. monitoring the weather with a local weather station located next to the site offices[376]

e. evaluating the LJQI for each lift surface[377]

f. sampling RCC during placement for testing in the site laboratory.[378]

Observations from trial embankment

4.229 Sampling and testing of the RCC during the trial embankment established the following:

a. if compaction of the RCC was delayed beyond around 35 to 40 minutes, compressive strength of the RCC started to decrease slightly.[379]

b. the VeBe time of the mixes was difficult to determine and obtained values ranged from 23 seconds to more than one minute, averaging 42 seconds. Those results compared with a specified 'modified' VeBe time in the range of 20 to 30 seconds. It does not appear that any particular concerns were raised about the low workability of the mix. It is possible that the onsite test results were dismissed because most of the tests were '*done with an unleveled, unfixed, and broken vibrating table*'.[380]

c. RCC field densities were low. Only 40% of the measured densities reached the specified requirements. While the vast majority of densities in the top of the RCC lifts were compliant, the situation was worst at the bottom of the lifts. 33% of the densities there were below the specified limit for small compaction equipment, while 77% were below the threshold for areas compacted by heavy rollers.[381] A review of the spread of different compaction results by layer, along with test pits gave comfort that '*segregation was not generalised*'.[382]

4.230 The quality of all lift joints in the trial embankment was evaluated using the LJQI. All final scores were reported as being more than 1. '*One of the reasons is that high densities were obtained in almost all top layers'.*[383] That observation shows that the LJQI scores recorded at that time were skewed by observations of the top of RCC lifts. Any low densities (indicating segregation) at the bottom of the layer did not impact the LJQI score.

374 Exhibit 50, **SUN.114.003.0001**, .0038, .0044.
375 Exhibit 50, **SUN.114.003.0001**, .0044.
376 Exhibit 50, **SUN.114.003.0001**, .0044.
377 Exhibit 50, **SUN.114.003.0001**, .0044.
378 Exhibit 50, **SUN.114.003.0001**, .0047.
379 Exhibit 50, **SUN.114.003.0001**, .0048.
380 Exhibit 50, **SUN.114.003.0001**, .0049.
381 Exhibit 50, **SUN.114.003.0001**, .0050.
382 Exhibit 50, **SUN.114.003.0001**, .0052.
383 Exhibit 50, **SUN.114.003.0001**, .0060.

4.231 The LJQI overlooks the impact that segregation at the base of an RCC layer will have on the shear strength of the lift joints. That is important because, as Dr Schrader said, the friction angle would decrease with segregation of sufficient degree in the base of RCC layers.[384]

4.232 The notion that all LJQI scores exceeded 1 does not accord with Figure 23 of the summary report, which shows that the LJQI score of one of the base RCC layers was -2. However, a note to the figure says that '*RCC segregated surface was treated using bedding improving its quality*'.[385] That shows an early departure from the specified LJQI and is indicative of the approach that was taken to the LJQI later.

4.233 Although the trial embankment was designed to have only one cold joint, five type II cold joints and two cold type I cold joints developed during its construction. This is a result of lower production rates due to, among other things, pausing RCC production for workers to take breaks, breakdown of equipment, poor coordination of tasks, and not enough workers.[386]

4.234 Bonding between layers in areas covered by bedding mix was reported as '*excellent*'.[387] Those observations were made of the vertical face that was cut through the trial embankment depicted in the figures below.

*Figure 4.9 – Vertical face cut through at chainage 903 showing damped bond between two lift joints. (Exhibit 109, **SCE.019.0001**, .0009)*

384 **TRA.500.010.0001**, .0034 ln 15-37.
385 Exhibit 50, **SUN.114.003.0001**, .0061.
386 Exhibit 50, **SUN.114.003.0001**, .0067 to .0068.
387 Exhibit 50, **SUN.114.003.0001**, .0062.

*Figure 4.10 – Vertical face cut through at chainage 902 (Exhibit 109, **SCE.019.0001**, .0009)*

4.235 Shortly after the trial embankment was constructed, Dr Schrader provided the following evaluation, with which Mr Herweynen was '*content*':[388]

> *The downstream face of the trial section has been trimmed. It is the best trimmed face I have seen, especially for lean RCC.*
>
> *From a distance it has a very good and uniform appearance. Up close, localized defects can be seen, but these are less than typically seen in lean RCC, especially for a trial section, and of no major significance.*

4.236 Dr Schrader later described the approach taken to inspecting the lift joints in the trial embankment in these terms:[389]

> *Trenches have become the best way of inspecting and scrutinizing RCC test fills and the real RCC mass. They allow everyone to see the real in-situ situation and quality. By blowing off the surfaces with air, and then washing it, everything can be easily seen by everybody. … By washing the surface and allowing it to dry back any defects will be highlighted. They will tend to stay dark and damp longer than the surrounding areas that will dry back to a light grey color. A person can also pick at the lift joints to see if they have contamination and are tight or not. The photographs [above] are of the full section of the right abutment (the trial section) at about chainage 902. The dam was constructed past this chainage and then excavated back to expose the interior for all to see. Despite being the trial section and the first RCC where crews were on a 'learning curve' the RCC is basically one very solid mass. There are two lift joints where wetting showed tight, but less than perfect, lift joint quality. However, this is in an area where good bond is not required and it is not through the entire section. It may also be*

388 Exhibit 51, **SUN.010.002.0355**, .0355.
389 Exhibit 109, **SCE.019.0001**, .0008.

where, as part of the trial, something was done as a variable to see the outcome. At any rate, even these joints will clearly have good friction but probably reduced cohesion.

4.237 Project personnel relied on the observed condition of lift joint bonding in the trial embankment. For instance, when a corehole taken from the Dam wall in early 2006 indicated debonded lift joints, the engineer who had observed that condition, Mr Montalvo, took comfort from the trial embankment, which, he said, truly represented the state of RCC lift joints within the Dam.[390]

Lessons learned from the trial embankment

4.238 The trial embankment report included a list of the lessons that had been learned from the process:[391]

- *Effect of segregation on the quality of the RCC.*
- *The importance to control the speed of the roller to increase RCC field densities.*
- *The effect of rain on uncompacted fresh RCC and the right response in case this situation happens again.*
- *Right way to do the cleaning activities, avoiding undercutting the surface when air jetting was used and deciding when it is necessary or convenient to use it.*
- *Appropriate way for curing and control the loss of moisture during spreading operations using a mist of water.*
- *How to roll edges of RCC and cleaning them afterwards.*
- *How to spread the bedding mix against vertical U/S & D/S faces in order to improve quality of the facing.*
- *Coordination between RCC & bedding placing activities in order to minimise delays between their placements.*
- *Recognising when RCC has too much moisture. In this case it is advisable to improve the coordination with the Aran pug mills operator regarding the RCC moisture dosage required on site.*

390 Exhibit 95, **ALC.001.001.1683**, .1683.
391 Exhibit 50, **SUN.114.003.0001**, .0071 to .0072.

Construction

4.239 Under the Alliance Agreements, Macmahon was to apply its construction expertise and resources to achieve the required standard for the Works and rectify defects.[392] Under Schedule 7 to the Alliance Agreements, Macmahon was responsible for:[393]

> *The provision of resources to manage and supervise the delivery of the Works in accordance with the Alliance Agreement, in particular the provision of resources to plan and manage the construction of the Works.*

4.240 An organisation chart for the Alliance[394] from around April or May 2005[395] shows that construction personnel included:

a. Mr Hamilton – Project Manager

b. Mr Embery – Construction Manager

c. Colin Lindschau – Project Superintendent

d. Brian Langridge – Area Manager Materials

e. Rob Frazer – Services/QA Manager

f. Jason Wells – Engineer RCC Works.

4.241 The personnel in sub-paragraphs c. to f. above had a direct reporting line to Mr Embery, who in turn reported to Mr Hamilton.[396]

4.242 Other personnel not depicted on the organisation chart but whose roles were relevant to RCC placement included:

a. shift supervisors, one of whom was Brian Chivers[397]

b. Mr Lopez and Mr Montalvo who, on the recommendation of Dr Schrader, were employed as the RCC Engineers[398]

c. Mr Brampton and Paul Rickert, who were the RCC shift engineers (referred to in this report as '**RCC Inspectors**').[399]

4.243 The RCC Engineers reported to the QA Manager, Mr Frazer.[400]

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392 Exhibit 19, **ALL.144.002.0389,** .0411.
393 Exhibit 18, **SUN.009.002.0020**, .0089; Exhibit 19, **ALL.144.002.0389**, .0460.
394 Exhibit 115, **SUN.175.006.0009**.
395 **TRA.500.009.0001**, .0073 ln 8-16.
396 Exhibit 115, **SUN.175.006.0009**.
397 **TRA.500.009.0001**, .0075 ln 11-19.
398 Exhibit 244, **HER.001.0001**, .0040 [187].
399 **TRA.500.009.0001**, .0074 ln 20-21.
400 **TRA.500.009.0001**, .0075 ln 7-9.

4.244 When the Dam was built, there were few people in Australia with experience in constructing RCC Dams. Dr Schrader recommended that Mr Lopez and Mr Montalvo be employed as the RCC Engineers. Dr Schrader had worked with Mr Lopez previously. At Dr Schrader's request, Mr Lopez had been employed on an RCC Dam in Mexico. Mr Lopez had also worked as an RCC Engineer in Jordan on a dam that incorporated LCRCC with basalt aggregate, no flyash and a Carpi membrane.[401]

4.245 Mr Montalvo had worked on an RCC dam in Mexico[402] as the Laboratory Supervisor on that project.[403] Dr Schrader had consulted to the contractor, which explains how Mr Montalvo came to be recommended for a position at the Dam.[404] At the Dam, Mr Montalvo reported to Mr Lopez.[405]

4.246 Mr Brampton, one of the RCC Inspectors, was a graduate engineer when he started working on the Dam. He had no prior experience with dams or in construction.[406] Mr Brampton saw himself as an 'underling' on the project. No one reported to him.[407]

RCC placement method

4.247 Given the very lean nature of the RCC mix used at Paradise Dam, the construction methods required by Section 11 of the Specification were exacting.

RCC production

4.248 Aggregate used in the RCC mix was basalt from the diversion channel, supplemented by additional basalt upstream of the diversion channel and Goodnight Beds material from the dam foundation.[408]

4.249 A crushing plant was established on site and an aggregate stockpile built adjacent to the Dam footprint. Rather than using separate stockpiles of aggregate of different gradation, a single stockpile blended the graded aggregate.[409]

4.250 Over one million tonnes of aggregate were prepared and stockpiled ready for use in RCC production.[410] The stockpile was almost 30 m high during the early stages of RCC production.[411]

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401 Exhibit 88, **DNR.005.4886**, .5144.
402 Exhibit 99, **MOR.001.0001**, .0004.
403 **TRA.500.006.0001**, .0023 ln 42 to .0024 ln 8.
404 **TRA.500.006.0001**, .0024 ln 22-35.
405 **TRA.500.006.0001**, .0025 ln 28-29.
406 **TRA.510.015.0001**, .0001 ln 4-8.
407 **TRA.510.015.0001**, .0006 ln 20-22.
408 Exhibit 75, **PDI.037.0001**, .0003.
409 Exhibit 75, **PDI.037.0001**, .0003.
410 Exhibit 75, **PDI.037.0001**, .0003.
411 Exhibit 75, **PDI.037.0001**, .0004.

Photo 3. View of primary (jaw) crusher for aggregate production.

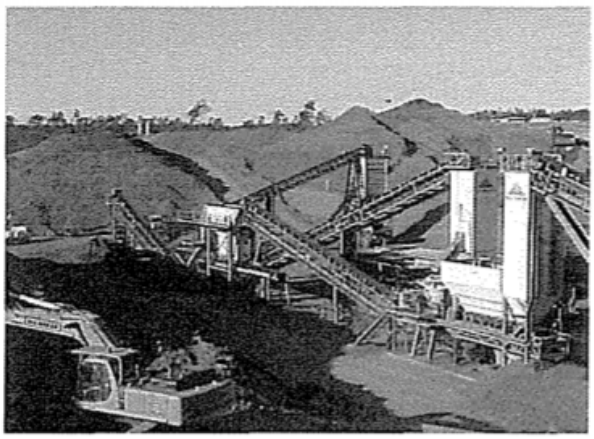


Photo 4. View of crushing plant for aggregate production.

Photo 5. Aggregate delivery by trucks from crushing plant to stockpile.

Photo 6. Panoramic view of the aggregate stockpile.

*Figure 4.11 – Figures of aggregate production and stockpiling. (Exhibit 50, **SUN.114.003.0001**, 0015)*

4.251 Two RCC batching plants were erected on site. Aggregate was fed to the plants from the stockpile with front-end loaders, which loaded their buckets from the face of the stockpile. A dozer mixed aggregate on the stockpile and supplied the face with blended aggregate.

Delivery of RCC

4.252 RCC was required to be deposited at the location at which it was to be spread. The primary delivery system for RCC from the pug mills to the wall of the Dam was by conveyor. The conveyor was to discharge RCC in '*windrows*'. The windrows were to be deposited adjacent to one another in a manner that did not cause '*unacceptable segregation*' and that moved the leading edge of the RCC forward. The Specification required that there be no 'fill-in' between windrows.[412]

4.253 The system consisted of a feeder conveyor from the pug mills that connected with the main tripper line conveyor which delivered RCC along the length of the Dam. A radial placing conveyor was used to deposit RCC in windrows at the leading edge of

412 Exhibit 21, **DNR.003.8385**, .8463.

the RCC. Once the Dam wall reached EL 61.8 m, the radial conveyor could not be used and instead a chute placed RCC on the Dam wall.

*Figure 4.12 – General arrangement of the construction site, including the main tripper conveyor and the radial placing conveyor. (Exhibit 75, **PDI.037.0001**, .0005)*

*Figure 4.13 – RCC being placed with the chute above EL 61.8 m. (Exhibit 75, **PDI.037.0001**, .0006)*

4.254 RCC was also delivered by dump truck in some locations. Truck delivery was required by the Specification to be accomplished with a dump-spread action while the truck was moving forward. In areas where the radial placing conveyor could not deliver RCC (known as '*shadow areas*'), RCC was placed with dump trucks that were loaded by the radial placing conveyor (as depicted below). Trucks were also used to place RCC:[413]

a. in the left abutment above EL 65.8 m

b. in the secondary spillway above EL 73.9 m

413 Exhibit 75, **PDI.037.0001**, .0006.

c. on the primary and secondary spillway aprons.

Figure 4.14 – Dump truck being loaded with RCC by the radial placing conveyor.
*(Exhibit 75, **PDI.037.0001**, .0005)*

Figure 4.15 – RCC being placed by dump truck on the secondary spillway apron.
*(Exhibit 75, **PDI.037.0001**, .0007)*

4.255 The third method of placing RCC was with a Creter crane. Trucks that were loaded directly from the pug mills unloaded into a hopper that fed a mobile conveyor to the Creter crane. By that system, RCC could be placed continuously in narrow areas, including over the diversion conduits.

Figure 4.16 – Creter crane being fed RCC by a conveyor system from the pug mills.
*(Exhibit 75, **PDI.037.0001**, .0006)*

Spreading of RCC

4.256 Within ten minutes of RCC being placed on the receiving lift, it was required to be spread into an even layer of nominal 310 mm thickness after compaction. Spreading in thinner layers to build up the lift thickness was generally not permitted. Spreading a thin layer onto previously spread and compacted RCC was absolutely prohibited.[414]

4.257 The spreading was done by a CAT D5-H dozer and two AC-100 Positracks, depicted in photographs below. Rubber tracks were fitted to the equipment to prevent movements causing damage to the lift surface. Other specified measures to protect the integrity of lift surfaces included allowing tracked spreading equipment to operate only on uncompacted material and not allowing equipment to '*crab, turn, or back onto freshly compacted RCC*'.[415]

Figure 4.17 – RCC being spread by the D5 dozer and compacted with a single drum vibratory roller.
*(Exhibit 75, **PDI.037.0001**, .0007)*

414 Exhibit 21, **DNR.003.8385**, .8464.
415 Exhibit 21, **DNR.003.8385**, .8464.

*Figure 4.18 – A Positrack spreading RCC in a confined area. (Exhibit 75, **PDI.037.0001**, .0007)*

4.258 As the placement of a lift progressed, exposed edges were to be kept '*live*' by progressively placing RCC out from them.[416] That approach reflects the general philosophy in RCC construction that the leading edge must always progress forward. For instance, RCC cannot be back-dragged with the blade of a bulldozer because that encourages separation of large aggregate particles. [417] RCC is spread in a forward motion into an even layer ready for compaction.

4.259 The Dam was to be raised at the same level within each placing area. As nearly as practical, the surfaces of only the receiving layer and the new layer were to be exposed at one time. More layers could be exposed in special circumstances.[418]

4.260 There were strict limits on the time from mixing to spreading and again to compaction. The time from the start of mixing to the completion of compaction depended on the temperature of the mix measured after spreading. For RCC mix not containing retarder or fly ash (which was the case at the Dam, at least at the start), when that measured temperature was:

a. above 25ºC, the time limit was 20 minutes

b. between 15ºC and 25ºC, the time limit was 30 minutes

c. less than 15ºC, the time limit was 40 minutes.

4.261 The Specification said that those times could be increased if tests and field performance showed that a relaxation was '*technically acceptable*' for the RCC mix and temperatures in question.[419]

416 Exhibit 21, **DNR.003.8385**, .8463.
417 Exhibit 100, **TAG.001.0001**, .0008 to .0009 [29].
418 Exhibit 21, **DNR.003.8385**, .8463.
419 Exhibit 21, **DNR.003.8385**, .8459-.8460.

Compaction of RCC

4.262 RCC compaction was mainly by large single drum vibratory rollers. Each layer was required to be compacted generally within 10 minutes of it having been spread.[420] This meant that the specified time from mixing to compaction was 20 minutes. Placing, spreading and compacting RCC called for a highly coordinated work crew and site operation.

4.263 The specified time to compaction was later extended to 35 minutes based on field observations and tests. The Specification was not revised in that regard. The fact of the changes was recorded by Dr Schrader in March 2005. His memorandum states:[421]

> *Without any site specific test data we initially stated in the specifications that the mix had to be compacted within 20 minutes of mixing at 30 C degrees. Based on field observations and tests, we later said that this could be extended to 35 minutes. The recent tests verify this limit as a good guide, with 45 minutes being the real limit. This is the maximum length of time from the start of mixing until the final pass of compaction. The maximum achievable density starts dropping immediately after mixing, and continually drops every minute The strength does not suffer much until about 40 minutes, at which time the loss of strength with every minute is significant An extra 10 minutes means about a 25% loss of strength.*

4.264 The large vibratory rollers could be used in open areas. Compaction of RCC in confined spaces and adjacent to the upstream face where the PVC membrane needed protection required smaller compaction equipment. The density requirements for these confined areas were less strict.[422]

*Figure 4.19 – Compaction with vibratory plate in a confined area (Exhibit 75, **PDI.037.0001**, .0007)*

420 Exhibit 21, **DNR.003.8385**, .8465.
421 Exhibit 204, **SUN.010.002.0286**.
422 Exhibit 21, **DNR.003.8385**, .8466.

4.265 Rolling was required to achieve a flat surface with minimum marks from the edge of the roller drum. The Specification stated that:[423]

> *Spreading and rolling shall be done so that a flat surface results with minimum roller marks from the edge of the drum, and so that the drum does not bridge over more than 10 percent of the surface beneath the roller drum after the last pass.*

4.266 This requirement does not align with the criteria against which 'surface flatness' was to be evaluated using the LJQI, namely:[424]

Surface flatness	0	Roller drum contacts at least 80 per cent of the surface (not more than 20 per cent bridging low spots). All RCC surface contacted by roller with at least one pass.
	-2	Roller bridges more than 20 percent while going over high points, but all RCC surfaces contacted by roller with at least one pass.
	-7	Roller leaves any area of RCC surface without compaction.

4.267 While the Specification for compaction required that the drum not bridge over more than 10% of the lift surface after the last pass, the RCC Inspectors were looking to see whether the drum bridged more or less than 20% and whether the entirety of the surface was contacted by the drum with at least one pass.

4.268 It is not clear why the requirements in section 11.9.1 of the Specification did not dovetail with the criteria for surface flatness in the LJQI.

Placement rates

4.269 The Specification stated an intent to construct the RCC mass in as nearly a continuous operation as was practicable. Before construction started, the Alliance had planned for the rate of rise within each placing area to be at least 0.6 m per day.[425] That rate was not achieved. The maximum placement rate was close to 350 m^3/hour when both pug mills were operating at the same time. The maximum daily placement rate was 5,966 m^3. The maximum monthly placement was achieved in March 2005 with 78,614 m^3 of RCC being laid.[426]

4.270 The Specification stated that the contractor would work at least seven days per week, 20 hours per day during placement of RCC.[427]

4.271 The original placement schedule had RCC in the critical section of the Dam being placed during the cooler parts of the year. It was intended that the Dam would be completed before ambient temperatures increased in the warmer and wetter months of the year. Not only would that schedule have aided in the quality of RCC placement, because with lower ambient temperatures the RCC remained workable

423 Exhibit 21, **DNR.003.8385**, .8465.
424 Exhibit 21, **DNR.003.8385**, .8468.
425 Exhibit 21, **DNR.003.8385**, .8462.
426 Exhibit 75, **PDI.037.0001**, .0008.
427 Exhibit 21, **DNR.003.8385**, .8462.

for longer and there was less chance of rain damaging lift surfaces, it was also important for managing thermal stresses.

4.272 The peak period of RCC production and placement was from December 2004 to April 2005, being the wetter and warmer months that the Alliance had originally hoped to avoid for RCC placement. This resulted in construction issues on site. RCC placement was less efficient because it was conducted during the wet season, which caused extra work and delays.[428] Related to this issue was that placing and rolling RCC during even short light rain can result in serious damage to the RCC lift surface. For that reason, the Specification said that '*RCC placing may have to be suspended during rains*'.[429]

Protecting the lift joints

4.273 One of the disadvantages of building with LCRCC is that, because it is prone to segregation, care in construction is required. LCRCC is not as workable nor as forgiving as HCRCC.[430] Consistent with that disadvantage is the degree to which the lifts needed protection during construction.

4.274 Lift surfaces were required to be kept continuously damp until the next layer was placed. The standards required by the Specification included that:[431]

> *At least one person 24 hours per day, seven days a week shall be on duty on the placement with the sole responsibility of operating the water system to maintain the entire surface moist but not over watered. He may be allowed to perform routine maintenance of nozzles and move hoses only if these activities do not prevent him from fully accomplishing his responsibility of keeping the entire exposed surface in a continuously damp condition.*

4.275 The lift joints were required to be uncontaminated when receiving the next RCC lift. The Specification required that no equipment be allowed to track mud or other contamination onto previously-placed RCC. Any contamination that did occur was required to be cleaned before placing the next lift.

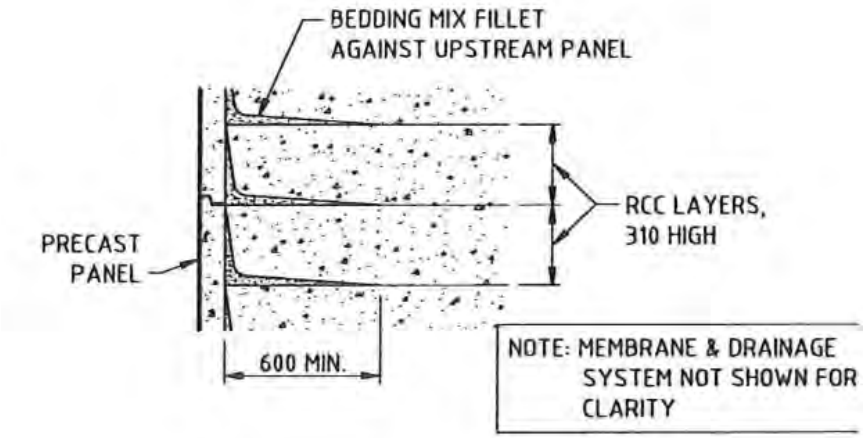
4.276 At least one portable vacuum hose connected to a vacuum truck was required to be available during all RCC placement to clean water, sand, debris, segregated aggregates, and loose stones from RCC surfaces.[432] All machinery operating on the lifts was required to be maintained in good operating condition to prevent leaking oil or grease from contaminating the lift surfaces.[433] Compacted RCC surfaces that were to receive bedding mix at the upstream face were required to be especially

428 **SUN.018.019.6859**, .6882.
429 Exhibit 21, **DNR.003.8385**, .8463.
430 **TRA.500.002.0001**, .0033 ln 16-19.
431 Exhibit 21, **DNR.003.8385**, .8473.
432 Exhibit 21, **DNR.003.8385**, .8467.
433 Exhibit 21, **DNR.003.8385**, .8464, .8465.

clean. The upstream half of the Dam was required to be '*essentially uncontaminated*' when the RCC was placed on it.[434]

Bedding mix

4.277 All lift joints were to receive 600 mm of bedding mix at the upstream face as shown below.[435]



4.278 The original bedding mix had a maximum aggregate size of 10 mm and a cement content of 300 kg/m^3. In September 2004, the cement content was reduced to 200 kg/m^3 with an equal amount of flyash replacing the other 100 kg/m^3 of cement. The slump of the bedding mix was generally 200 mm. The bedding mix included superplasticiser, an air entrainment agent, and retarder admixtures.[436]

4.279 Where RCC was spread onto bedding mix, the RCC mix was to be spread and compacted:

a. within two hours of the time the bedding mix was batched

and

b. before the time that bedding mix begins to set or dry from exposure, and within 40 minutes of when the bedding mix was first deposited.[437]

434 Exhibit 21, **DNR.003.8385**, .8467.
435 Excerpt from **DNR.006.0001**, .0017.
436 Exhibit 75, **PDI.037.0001**, .0008.
437 Exhibit 21, **DNR.003.8385**, .8464.

*Figure 4.20 – Bedding mix laid at the upstream face of a lift surface. (Exhibit 293, **HYT.600.003.0001**)*

Construction in monoliths

4.280 The Dam was constructed in monoliths generally 45 m wide from monolith A (in the left abutment) to monolith W (at the far end of the right abutment). The primary spillway comprised monoliths D to K as shown below:

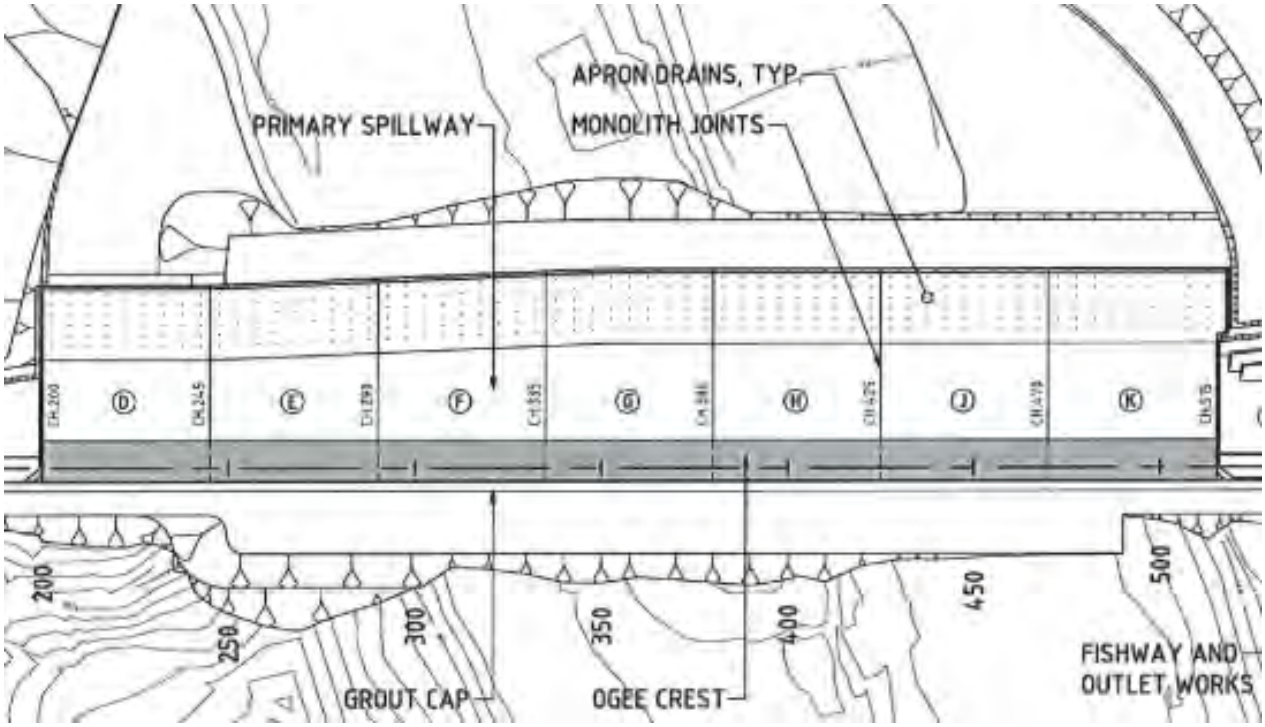


*Figure 4.21 – General arrangement of the primary spillway. (Excerpt from **DNR.006.0001**, .0014)*

4.281 The contraction joints were formed using reinforced plastic joint formers. The formers were L-shaped and were erected on the receiving lift surface ahead of RCC being placed. They were used all the way from the upstream to the downstream face in the top and bottom 4 m of the Dam wall, but only across 25% of the width of the Dam from each face in the interim lift surfaces of the Dam.[438]

*Figure 4.22 – Monolithic joint formers. (Exhibit 75, **PDI.037.0001**, .0008)*

Cold joints

4.282 The nature of hot and cold joints was explained by Mr Brigden in the following way:[439]

> *A hot joint, in terms of RCC, is a joint where the material that has been placed has not as yet reached an initial set. Cement has two stages of setting - you have initial set and you have hard set. We also control initial set with additives, and in current jobs, in the last two or three years, that I've been involved with we actually slow the concrete down for 36 hours-plus to enable you to get the next lift on top and get inter-particle bond by - the aggregate in the lift above is penetrating the one below, plenty of fresh paste to give a good bond. Then you end up with one cold joint, and a cold joint is a joint that has passed final set. It therefore then needs special preparation to put the next layer on top. It's then treated as conventional concrete, which is then wire-brush broomed, hit with air, cut with water blasting to expose the aggregate, covered with mortar and then the next lifts are placed on top.*

4.283 Cold joints were contemplated by the Specification. The length of time that could be tolerated between placing successive layers of RCC was a function of the surface temperature and time since rolling. If the time exceeded specified limits, the receiving lift surface was considered to be a cold joint.

4.284 The treatment of cold joints differed according to how much time had elapsed. The maturity of a lift joint was a function of the time that had elapsed since placement and

438 Exhibit 75, **PDI.037.0001**, .0008.
439 **TRA.500.002.0001**, .0008 ln 7-23.

the average temperature at the lift surface measured hourly after compaction. The two factors were multiplied together to give the number of '*degree hours*' (or ºC-Hr).

4.285 The Specification classified a cold joint as '*Type I*' when more than 500 degree hours had passed before placement of the successive layer of RCC, but not more than 36 hours had elapsed. A type I cold joint was required to be treated with a nominal 25 mm thickness of bedding mix spread over the upstream 25% portion of the lift surface.[440]

4.286 A '*Type II*' cold joint occurred when more than 36 hours had passed before placement of the successive layer. Treatment of a type II cold joint was the same as for a type I, save that the bedding mix was to be 20% wider than for a type I cold joint (i.e. bedding mix was to be spread over the upstream 30% of the lift surface).[441]

4.287 Of cold joints, the Specification also stated:[442]

> *As placement of a lift progresses, exposed edges shall be kept "live" by progressively placing out from them. Whenever a cold joint at any edge of any lift does occur, it shall be located at least 3 m from the location of other cold joints that may have previously occurred in the same direction. The joint edge shall be prepared as required for "cold joints" prior to resumption of RCC placement. In general, no cold joint shall be allowed along the edge of a lift in the upstream-downstream direction for more than one-half of the upstream/downstream dimension of the dam at that elevation.*

Quality assurance and quality control

4.288 The Specification contained '*guidelines*' for RCC inspection, quality control and quality assurance. That part of the Specification was written by Dr Schrader. The system was composed of two complementary efforts:[443]

a. inspection and testing activities from an engineering and inspection perspective with appropriate QA documentation, for which '*the Engineer*' was responsible

and

b. control of construction operations and materials by workforce and supervisory personnel ('*quality control*' or '*QC*'), responsibility for which lay with the '*construction personnel*'.

4.289 Teamwork and coordination between QC and QA were expressed to be '*essential*'.[444] To avoid the possibility that the Engineer might be unduly influenced by programming pressures, the Specification required the Engineer to report directly to

440 Exhibit 21, **DNR.003.8385**, .8469.
441 Exhibit 21, **DNR.003.8385**, .8469.
442 Exhibit 21, **DNR.003.8385**, .8463.
443 Exhibit 21, **DNR.003.8385**, .8500.
444 Exhibit 21, **DNR.003.8385**, .8500.

the Project Manager.[445] However, the Alliance did not adopt that reporting structure. Members of the QA team reported to the QA Manager, Mr Frazer, who reported directly to Mr Embery.[446] The RCC Engineers did not report to the Project Manager because they had an independent role.[447] Mr Hamilton recalled that any matters that the RCC Engineers had would have been raised with the construction personnel or Mr Embery.[448] Mr Embery's primary focus during construction was on the program (i.e. the construction schedule).[449]

4.290 The overlap between QA and QC responsibilities between, first, the engineering and design teams and, secondly, the construction team respectively were described in the following way:[450]

> *Construction personnel are responsible for controlling and supervising the work so that it proceeds in accordance with the plans, specifications, and reasonable requirements of the Engineer. Quality control by construction personnel does not relieve the Engineer of responsibility for safeguarding the interest of design, the owner, and regulatory agencies associated with the project. The Engineer is responsible for activities necessary to assure that construction has indeed complied with the plans and specifications, that the construction group is implementing an adequate quality control program, and that the project is proceeding in accordance with obligations to environmental and dam safety criteria. These activities include checks and tests of products contained in construction, review of construction personnel quality control activities, verification of processes used in the work, and inspection of the finished work to determine that construction requirements have been met.*

4.291 The Engineer with QA responsibilities was required to be employed full time and to be '*fully experienced with dams and RCC construction, including the type of RCC and equipment to be used at Burnett River Dam*'.[451] Inspection responsibilities were originally envisaged to be split between construction and QA personnel. The Specification provided:[452]

> *[C]onstruction and QA personnel should establish, in an advanced meeting and in writing, exactly who in each organization will be responsible for exactly what tests and activities that comprise the requirements of QC and QA. The foremen of the construction group generally are responsible for issues such as assuring that adequate equipment meeting the specification requirements is available and functional, assuring that the minimum number of passes of the roller are achieved, checking the lift thickness and grade, etc. The QA engineer will have*

445 Exhibit 21, **DNR.003.8385**, .8500.
446 **TRA.500.009.0001**, .0079 ln 37-46.
447 **TRA.500.015.0001**, .0024 ln 35-37.
448 **TRA.500.015.0001**, .0024 ln 40-43.
449 **TRA.500.009.0001**, .0072 ln 24-34.
450 Exhibit 21, **DNR.003.8385**, .8500 to .8501.
451 Exhibit 21, **DNR.003.8385**, .8501.
452 Exhibit 21, **DNR.003.8385**, .8501.

specific individuals assigned to sampling the RCC and making cylinders, checking the density with a nuclear gauge, monitoring the results of aggregate tests, keeping records, etc.

4.292 That division of responsibility was not reflected onsite. All hold and inspection points on QA documentation for RCC placement, for instance, were signed off by the RCC Inspectors, who were relatively junior QA personnel. Construction personnel did not sign off any of those points despite that being required by the Specification.

Quality control

4.293 Managerial responsibilities for QC were described in the Specification:[453]

> *The quality control organization should be headed the project manager or general superintendent. His primary QC responsibility should be coordination and reporting of quality control activities. Under him will be an organization for each shift which will primarily consist of the foremen responsible for activities such as mixing, placing, cleanup, curing, and protection.*
>
> *All persons responsible for quality control functions should be able to demonstrate their ability to correctly perform the duties required of them. The construction project manager or superintendent should designate which employee has responsibility and authority to order action whenever testing or inspection shows noncompliance with the specifications or approved submittal.*

4.294 'QC responsibility' for RCC was described as including:[454]

> *[C]onsistently producing, stockpiling, and delivering to the mixer aggregates of acceptable quality, including particle shape, gradation, and plasticity. QC is responsible to set-up the mixing and delivery system that thoroughly blends all ingredients, and delivers it to the placement with the correct cement content and at the designated moisture for compaction. Water added to RCC at the plant should be adjusted by QC as needed to provide a mix that, at the time of compaction, just barley starts weaving and that is not so dry that segregation occurs. On a continuous basis, the placing foreman is usually the best person to determine if the water content should be increased or decreased, but he should be guided by the QA chief. Concern over attaining the specified water is not as critical as it is with conventional concrete. The more important criteria is performance of the mix under the roller. However, QC construction personnel should keep track of any mix adjustments for record purposes. This is best done by the mixing plant operator.*
>
> *Other contractor responsibilities for timing, spreading, handling, compacting, protecting, and joint treatment are explained in the technical and quality control portions of the specifications.*

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453 Exhibit 21, **DNR.003.8385**, .8502.
454 Exhibit 21, **DNR.003.8385**, .8503.

Quality assurance

4.295 In terms of QA, the Specification provided substantial detail. It set out how QA was to be organised on site and describe the resources necessary to support the system:[455]

> *At Burnett River Dam, inspection and testing will be headed by a Quality Control Engineer. He will have laboratory and inspection personnel as required, and the assistance of an RCC consulting specialist. The QA Engineer will be responsible for all phases of quality assurance from aggregate production through curing and protection of the concrete. Under him, the organization should include a shift supervisor (chief inspector) and laboratory chief for each shift. Additional people will be used as necessary to adequately cover activities such as inspecting each location where a major concrete placement is being made, the mixer and batch plant at start-up and if problems develop, aggregate handling, cleanup, curing, protection, and finishing. The organization should include an experienced chief laboratory technician for acceptance testing of aggregates and fabrication of RCC cylinders. Nuclear density and moisture tests of the compacted material can be done by a team of two people, with one of them being a designated construction personnel QC person on the placement.*

4.296 The reports and forms to be used were set out:[456]

> ***S12.3.2 Reports and Forms***
>
> *The value of clear, concise, and complete records is sometimes overlooked during the construction of a project. In devising a system of reports, and preparing forms for these reports, it should be realized that these reports will constitute a part of the official record of the construction operations and may be the sole source of reliable information on the procedures, practices, and results obtained during construction.* ***The records will serve as a basis for other RCC dams and may be subject to much public review. They may also be the primary basis for solving disputes and for evaluation of any problems that may develop.***
>
> *Reports will supply information on, and preserve as a permanent record, facts concerning progress of the work, factors affecting progress, instructions given to construction personnel, samples secured, tests made, and any other data. Mixing plant reports prepared by the mixing plant operator or inspector will usually serve as a basis for payment of cement.* ***These reports should also be in such form and detail as to show quantities and classes of concrete placed in each location,*** *concrete wasted and the reason for such waste, the test report identification for each bin of cement (and possibly fly ash) used in concrete production, data on quantities of materials used in the various classes of concrete placed, and any other information necessary to make the record complete. A record should also be kept off the calculated amount of cement used per day or shift based on the volume of concrete and mix deign(s), compared to*

455 Exhibit 21, **DNR.003.8385**, .8503-.8504.

456 Exhibit 21, **DNR.003.8385**, .8504-.8505 (emphasis added).

the amount of cement used based on truck deliveries and 'dipping' or 'sounding' the silo at the start and end of each shift. Unusual occurrences in the plant should be noted by the inspector or plant operator. Reports made by placing inspectors and inspectors engaged in cleanup, curing, finishing, and protection should follow a well-devised standard form on which the essential facts are recorded for the period of inspection.

4.297 The Specification set out the responsibilities of the QA team:[457]

The QA Engineer and his staff are responsible for acceptance testing and inspection, and for monitoring the construction group's quality control operations. These functions can be done in combination with the construction group's quality control program. They include, but are not limited to, inspection of all operations for compliance with specifications, reviewing and approving field adjustments of laboratory-designed concrete mixtures, verifying the properties and quality of concrete used in various structures or parts of the dam, and exercising all other necessary control over engineering aspects of the construction operations.

The effectiveness of inspection, with appropriate emphasis in areas of unique concern on an RCC project, depends to a large degree on the thoroughness with which inspection personnel are familiar with these types of activities and have been instructed. *While it is expected that the inspectors will have knowledge of the basic requirements for the production of concrete of high quality, it is nevertheless necessary to instruct even the best of inspectors in the details of inspections as they apply to each specific project. In the case of RCC, the inspector's also should have knowledge and experience in the placing of embankment materials and gravel fills.* ***Training conferences and discussions prior to and during construction will be used as needed. Previous experience cannot entirely compensate for proper instruction and training of inspectors in the duties peculiar to a particular job.*** *This is true of conventional concrete and especially true with RCC, where experience is limited. The specifications for this project include variations from standard practice where more lenient practices are acceptable.* ***Personnel responsible for inspection at Burnett River Dam will have the necessary general background in concrete and embankment, and will have been through an RCC orientation with the RCC consultant.***

4.298 The range of tests and inspections on the RCC were set out in section 12.3.7:[458]

(1) General: Quality assurance testing for roller-compacted concrete will primarily consist of being sure that the materials used are as specified, the mix proportions are correct, and placing is as specified. If these are correct, the strength and hardened material properties will develop with time to give a safe and crack-free monolithic mass. ***The actual material properties and strengths from cylinders and cores will probably not be known until well after the***

457 Exhibit 21, **DNR.003.8385**, .8505 (emphasis added).
458 Exhibit 21, **DNR.003.8385**, .8516 (emphasis added).

dam is essentially complete. To be sure the dam is constructed as designed, active Quality Assurance as the work is being done is, therefore, especially important. *On the other hand, roller compacted concrete can be more tolerant than conventional concrete in some regards. The specifications were written to take advantage of this latitude in tolerances, and thereby obtain the most economical safe project.*

...

(7) Modified VeBe: The modified vebee test procedure for workability is not an appropriate field control test for the RCC at Burnett River dam. It will used for record purposes only, each time a set of cylinders or a mixer/placement performance detest is done. The tests should use the 22.7 surcharge weight. The target VeBe time is expected to be about 20 to 30 seconds (initial VeBe time, or VeBe time 1). This will be verified or adjusted by the QA Engineer and RCC consultant at the start of RCC placement.

…

(12) Mix Designs: Mix designs for RCC will be established by the QA Engineer and the RCC consultant. As nearly as possible, it is the intent to use one mix design throughout the project. However, after consistent RCC production has been established, the QA Engineer may authorize or direct adjustments. An exception to this is the quantity of mix water which will probably be adjusted frequently according to weather conditions so that a compactable mix is maintained at the placement. Unlike conventional concrete, experience with lean RCC mixes and test of the Burnett materials has shown that high changes in water content at the time of compaction on the order of plus or minus up to 0.1 % by weight do not significantly affect strength, but changes of more than 0.2% can be significant. As described in the specifications, the placing foreman should immediately request minor adjustments in water as necessary to maintain compactability and consistency of the RCC at the placement, but he should request confirmation of the QA Engineer if these changes exceed about 0.2% of the design moisture. From a density and joint integrity standpoint, it is preferable to operate at the wettest possible consistency which does not produce a sticky mix or allow the roller to sink.

…

(16) Cold Joints: The contract specifications are clear concerning what constitutes various degrees of cold joint that could occur and the treatment required for each one. The maturity limit before a bedding mix is needed is based on results of test slabs made using various surface conditions and various maturity times with RCC mixes for other projects. ***After taking into account the effects of normal load which will be applied by the mass of RCC above any given layer at any point on the layer, adequate shear strength will be achieved if the specification requirements are followed.*** *As with the foundation contact, occasional small zones of less than perfect conditions can be tolerated, but no condition should be allowed where a continuous flow path from*

the upstream to downstream face of the dam could develop. ***This means that the upstream part of each lift is most critical and the downstream part is less critical.*** *There may be some instances where the maturity criteria has passed, but it is in the best interests of the project to recognize the situation and let it pass without bedding. At other times, a situation may develop where the maturity limit has not been reached, but a bedding mix is warranted. The Specifications allow the Engineer to direct the use of bedding in this case. The main uncontrollable variables are the joint surface conditions, and the fineness and chemistry of the cement. It is possible to get shipments of cement which meet specification requirements but which have considerably different set-time properties than assumed when the specifications were prepared. Continual review and evaluation of the chemical and physical property analyses of cement (which should be in the hands of the Engineer before unloading materials at the jobsite) will provide the best guidance as to whether cold joints may require special attention. Other factors are environmental conditions, the temperature of cement when unloaded, location of the joint in the structure, the amount of fines in the mix, the particular RCC mix being placed, and the degree to which construction personnel has complied with the requirement to keep the joint surface damp.*

(17) RCC Spreading and Layer Thickness: The thickness of RCC layers is not especially critical. Tolerances in the specifications reflect this. This is one area where detailed inspection would be difficult, but fortunately it is not of great importance. From previous work, and for the rolling equipment specified, essentially the same quality should be achieved for 250, 300, 350, and 400 mm thick layers. A slight increase in density may be found with thinner layers down to about 200 mm, but production decreases and the number of joints increases. If the construction group can show adequate compaction and joint quality for thick lifts, this should be encouraged.

It is also important to recognize that compaction of RCC in excessively THIN layers less than about 200 mm result in poor final compaction. When an RCC layer tapers down in thickness as the foundation rises, it should be stopped before the thickness is less than about 200 mm, and the edge should be compacted.

(18) Placing Rate: As a guide, the maximum length of time allowed from mixing until depositing, from depositing until spreading, and from spreading until compaction is about 10 minutes for each operation. This is a good guide, but in reality it is the total time from the start of mixing until the completion of compaction that is important. The specifications provide time limits based on the mix to be used and the temperature. In warm weather, the time allowed is less than in cool weather. ... In cool weather it can sometimes be increased.

(19) Vibratory Rollers: Contract specifications are very clear as to the type of equipment to be used, when compaction should be accomplished, the equipment capability, and the number of passes required. There is no question that the

equipment specified will accomplish the job. Lesser equipment will not be satisfactory.

…

One of the most important things to keep track of is the length of elapsed time between spreading and rolling. During the spreading operation, the dozer tracks will give an initial compaction. The cement will be hydrating at that time and will begin to chemically develop strength and bond. The longer the wait between spreading and rolling, the more damage will be done to the mix and more difficulty will be experienced in compaction. This becomes more critical as the weather becomes warmer. The time also is dependent upon the actual chemistry and fineness of the cement being used.

Quality Management Plan

4.299 To identify the structure and practices that would '*provide control and assurance of the quality of all work to be carried out*' on the Dam, a Quality Management Plan was prepared for the Alliance.[459] That Plan required a review of the Specification and the construction drawings to understand and identify their key requirements including certifications of materials, construction methods, inspections by designers, testing requirements, and statutory witnessing or approval. The information from that document review was used to prepare inspection and test plans (**ITPs**). The Project Manager was responsible for ensuring that the document review was conducted.[460]

4.300 The ITPs were required to:[461]

a. identify the progressive checks that were required throughout construction and the nature of those checks, be they visual inspection, performance testing, as-built surveys, or '*authority inspection*'

b. describe what part of the work was to be inspected and what criteria it was to be checked against

c. specify the frequency of checking and the person responsible

d. describe what record of each check would be generated (e.g. signatures required, test reports).

4.301 The objective of using ITPs was to '*ensure that ALL individuals involved in the QA process, from Leading Hands to the Project Manager, develop a common understanding about each component of the ITP, Checklist and the approval process*'.[462] The Construction Manager was required to sign ITPs to '*show*

459 Exhibit 116, **SUN.162.002.0019**, .0025.
460 Exhibit 116, **SUN.162.002.0019**, .0045.
461 Exhibit 116, **SUN.162.002.0019**, .0046.
462 Exhibit 116, **SUN.162.002.0019**, .0050.

approval'.[463] The Construction Manager had other responsibilities under the Quality Management Plan, including to:[464]

a. ensure technical procedures and instructions were adhered to

b. control *'for receiving, in-process and final inspection & testing'*

c. identify and report on non-conformances

d. maintain the Non-Conformance Register

e. identify, report on and implement corrective action.

4.302 Asked about those responsibilities, Mr Embery said that his role was limited to ensuring that people who would do that work were on the project.[465] When it was suggested that his responsibilities went beyond that, Mr Embery said:[466]

> *[A]ll you need to do is to provide the people and the resources to do what they have to do. It physically doesn't involve the construction manager, in this case, going out there and checking every truck that comes on to site for materials, and so on. That's not the purpose of this. It's to ensure that the resources are there to do the work.*

4.303 Mr Embery was not involved in quality documentation. That work was done by others who reported ultimately to him.[467]

4.304 Foremen, site engineers and subcontractors were responsible for carrying out inspections, verifications and approvals in accordance with the ITPs and were required to sign off as confirmation.[468] Where an ITP nominated a hold point, work was not allowed to progress beyond that hold point '*until all nominated inspections and tests are carried out satisfactorily by the relevant person or authority*'.[469]

4.305 The Project Manager had overall responsibility for these requirements, in addition to the following further QC requirements relating to ITPs:[470]

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463 Exhibit 116, **SUN.162.002.0019**, .0047.
464 Exhibit 116, **SUN.162.002.0019**, .0033.
465 **TRA.500.009.0001**, .0079 ln 10-11.
466 **TRA.500.009.0001**, .0079 ln 29-35.
467 **TRA.500.009.0001**, .0080 ln 13-22.
468 Exhibit 116, **SUN.162.002.0019**, .0048.
469 Exhibit 116, **SUN.162.002.0019**, .0048.
470 Exhibit 116, **SUN.162.002.0019**, .0048.

Records	*Review test results and attach accepted Test Results to [Checklists] as required.* *Ensure records are maintained in accordance with this Management Plan and the Contract requirements*
Completion	*Inspect work handed over by the subcontractor and prepare punch lists of defects and omissions* *Re-inspect punch list items after rectification/action as required*
Non complying Work	*Process in accordance with Section 5.5*
Review	*Regularly review the implementation of the QC Plan (ITP) and whether the specified quality is being achieved* *Take action as early as possible to ensure compliance by the Subcontractor*

4.306 Section 5.5 of the Quality Management Plan concerned non-complying work. It provided little detail, referring only to the types of forms that would be used (including NCRs) and otherwise to the '*Procedure for Management of Nonconformances*' that would be documented in the '*Procedures Manual*'.[471]

RCC QA documentation

4.307 In relation to RCC placement, the following ITPs were prepared and variously used during construction:

a. '*Placement of Roller Compacted Concrete*' with Alliance document reference BDA-QA-CHK-0060 (**RCC Placement ITP**)[472]

b. the LJQI Scorecard[473]

c. '*Cold Joint Treatment Checklist*' with Alliance document reference BDA-QA-CHK-0061.[474]

471 Exhibit 116, **SUN.162.002.0019**, .0051. When the Commission notified SunWater that a copy of the Procedures Manual was required to be produced, SunWater was unable to locate any document styled as a Procedures Manual.

472 See, for example, Exhibit 276, **SUN.113.005.0202**.

473 See, for example, Exhibit 117, **DNR.020.014.4624**, .4625; Exhibit 275, **SUN.113.005.0202**, .0203.

474 See, for example, Exhibit 117, **DNR.020.014.4624**, .4626.

4.308 Other ITPs were completed in relation to a 'lot' of the RCC placed. For instance, a checklist for '*Carpi Membrane Joint Welding and Repair*', with reference number BDA-QA-CHK-0062 (**Membrane Repair ITP**), at times accompanied RCC Placement ITPs in the documents that were produced to the Commission.[475]

4.309 Revision 9 of the RCC Placement ITP had a total of 24 items for sign off, four of which were hold points.[476] The sign off items related to, among other things, previous lots having been completed in a way that conformed to the Specification, whether cold joint treatment was required, the placement, spreading and compaction of the RCC, curing, precast facing panels, and shaping and clean-up of the downstream face.

4.310 Revision 10 of the RCC Placement ITP had 21 sign off points, including four hold points.[477] The last sign off point in both revisions '*guaranteed'* the work. Despite the Specification indicating that construction personnel ought to check and sign off construction related items, every item (including each hold point) was checked and initialled by an RCC Inspector. Only four items were countersigned by a more senior RCC Engineer.[478] Three of those four points were hold points and the fourth was the sign-off to guarantee the work. The hold point that was not countersigned was hold point 1 seemingly related to workplace health and safety: '*Work method and [Job Safety and Environmental Analysis] reviewed and conveyed to all concerned*'.

4.311 During Mr Embery's evidence, the manner in which hold points on the RCC Placement ITPs were observed was explained in the following exchange:[479]

> *Q. This document starts off with hold points: hold points 1 and 2, hold point 2 is repeated, and then hold point 3. This is items 1, 2, 3 and 4 on the form. Can you explain what a hold point is?*
>
> *A. You're working on a section of work, and you reach a hold point and then you have to clear that before you then go on to the next phase of the work, so it's just a QA-style process.*
>
> *Q. Who's responsible for passing the hold point?*
>
> *A. The engineer, the supervisor.*
>
> *Q. So Ben Brampton here; is that correct?*
>
> *A. Yes.*
>
> *Q. Does work stop until Ben Brampton signs off the hold point; is that correct?*

475 Exhibit 275, **SUN.113.005.0202**, .0210.
476 Exhibit 117, **DNR.020.014.4624**, .4624.
477 Exhibit 276, **SUN.113.005.0202**.
478 See, for example, Exhibit 276, **SUN.113.005.0202**.
479 **TRA.500.009.0001**, .0109 ln 30 to .0110 ln 3.

A. *In theory, in theory. In practice, it tends not to be. It tends to be continuous operation, so you're not - if something is 200 metres long, you're not going to have 200 stop points. You just continue.*

4.312 The RCC Placement ITP was accompanied by the LJQI Scorecard, which required eight factors to be evaluated, with the total scores summed at the bottom of the form. The form reflected the LJQI in the Specification with three exceptions. First, the Specification said that -6 was the score for the worst assessment of surface tightness and condition; while the LJQI Scorecard allocated -4 points.[480] Secondly, -6 was the specified score for lifts with oil and fuel spills, which was changed to -8 on the LJQI Scorecard.[481] Thirdly, the Specification said that -5 was the score for the worst category of joint maturity while the LJQI Scorecard allocated -6 points.

4.313 An LJQI Scorecard was completed for each '*placement area*' by an RCC Inspector. The Scorecard made no provision for countersigning by an RCC Engineer.[482]

4.314 The Cold Joint Treatment Checklist had eight points at which a signature was required. One of those required the cold joint type to be classified by: (1) recording the hours that had elapsed since the receiving layer had been placed; (2) recording the average '*surface temperature*' since then; and (3) calculating the degree hours. The form said that bedding mix should be placed across:

a. '*10% of dam width for a continuous layer u/s to d/s*' for a type I cold joint

and

b. '*15% of dam width for a continuous layer u/s to d/s*' for a type II cold joint.[483]

4.315 Most lift joints were cold joints.[484] Less than 50 Cold Joint Treatment Checklists were identified in documents produced to the Commission. That compares with the many hundreds of RCC Placement ITPs generated. It appears that the Cold Joint Treatment Checklist was not used from about 22 November 2004.[485]

4.316 RCC lifts were divided into placement areas for the purposes of the ITPs. The QA forms completed for each lot were kept together as a bundle. Where relevant, they had other documents attached to them, including delivery dockets for bedding mix that was delivered and (presumably) applied to the lot. There are also survey records of the coordinates of the RCC lot. It is not clear how lifts were divided into lots or work areas. Some lots accord with monolith widths in the Dam wall being as short as 35 m[486] and 42 m,[487] while others were as long as 280 m.[488]

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480 Exhibit 117, **DNR.020.014.4624**, .4625.
481 Exhibit 117, **DNR.020.014.4624**, .4625.
482 Exhibit 276, **SUN.113.005.0202**, .0203.
483 Exhibit 117, **DNR.020.014.4624**, .4626.
484 **TRA.500.006.0001**, .0037 ln 40-41.
485 **SUN.021.006.0417**, .0419.
486 Exhibit 276, **SUN.113.005.0202**.

4.317 Where a lift was 280 m long, a single LJQI assessment for the whole lift could not be a meaningful assessment. The area of the lift is very large and it took a long time to place the lift. Mr Neumaier recognised this problem when discussing a later dam project he worked on that adopted the sloped layer method of RCC construction. That approach was used in the alternative to '*placing RCC on a very large area across the whole footprint of the dam and then running the risk that the assessment made in one corner of the layer might be different to the one half an hour later*'.[489]

4.318 The RCC Inspectors were required to sign off on up to 42 sign off points for a lot of RCC placed (not including the points on the Membrane Repair ITP).

4.319 The primary spillway was 315 m wide, while the Dam wall from the extremities of the abutments was approximately 920 m long.[490] The RCC layers were nominally 310 mm thick and the primary spillway was approximately 37.6 m high. In the primary spillway alone, that means that around 120 layers of RCC were placed equating to over 39 km of RCC placed horizontally in lifts of varying widths. If RCC lots were around 40 m long, there were nearly 1000 lots in the primary spillway alone. If lots are assumed to have been 280 m long, there were around 140 lots in the primary spillway. On either assumption, the RCC Inspectors were required to check and initial thousands of inspection and hold points in the primary spillway, let alone for the remainder of the Dam wall.

4.320 It was suggested to Mr Embery by Counsel Assisting that the QA system was complicated, involving hundreds of inspection points for one person to sign off on each RCC lift. Mr Embery recognised that it was typical of modern construction projects for there to be too much paperwork for the wrong personnel.[491]

Non-conformance reports

4.321 There were 251 NCRs raised during the project. Ultimately, each of the 251 non-conformances was 'closed out' by various means.

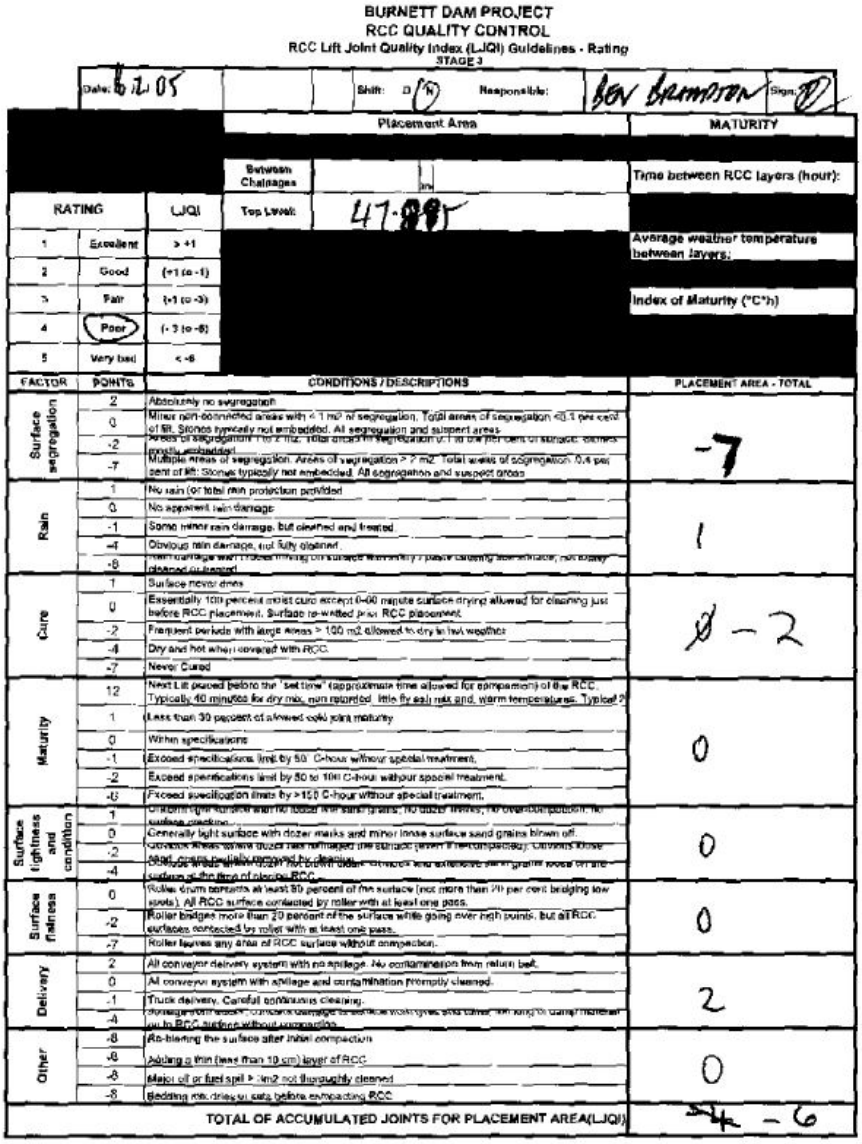
4.322 Mr Embery recalled that an NCR would have been raised if an LJQI score was low.[492] However, no NCRs appear to have been raised for that reason in spite of the LJQI inspections being the primary means of identifying quality issues with lift surfaces.

4.323 Increasing the LJQI scores after bedding mix was applied may have prevented the quality issues that low LJQI scores are meant to flag from being elevated to more senior personnel. The RCC Inspectors completed the LJQI Scorecards. The RCC Engineers signed them off, although often not at the time of RCC placement. All of

487 Exhibit 117, **DNR.020.014.4624**.
488 Exhibit 311, **SUN.021.006.2078**.
489 Exhibit 302, **TRA.510.021.0001**, .0011 ln 12-20.
490 **GHD.002.0001**, .0020
491 **TRA.500.009.0001**, .0117 ln 5-9.
492 Exhibit 114, **TRA.510.018.0001**, .0009 ln 19-32.

those personnel reported to Mr Frazer,[493] who was the QA Manager and responsible for raising NCRs.[494]

4.324 The Specification prohibited an RCC lift with an LJQI score of less than -5 from remaining in place.[495] However, there appear to be occasions where RCC with an LJQI score less than -5 was left *in situ*.[496] By way of example, an area of RCC in the primary spillway spanning 310 metres at EL 47.895 m was given an LJQI score of -6:[497]



BURNETT DAM PROJECT
RCC QUALITY CONTROL
RCC Lift Joint Quality Index (LJQI) Guidelines - Rating
STAGE 3

Date: 6 12 05 | Shift: D (N) | Responsible: BEN BRAMPTON | Sign:

[redacted] | Placement Area | MATURITY

[redacted] | Between Chainages | | Time between RCC layers (hour): | [redacted]

| RATING | | LJQI | Top Level: | 47.895 | [redacted]

| | | | | | Average weather temperature between layers: | [redacted]

| | | | | | Index of Maturity (°C*h) | [redacted]

1	Excellent	> +1
2	Good	(+1 to -1)
3	Fair	(-1 to -3)
4	Poor	(-3 to -6)
5	Very bad	< -6

FACTOR	POINTS	CONDITIONS / DESCRIPTIONS	PLACEMENT AREA - TOTAL
Surface segregation	2	Absolutely no segregation	-7
	0	Minor non-connected areas with < 1 m2 of segregation. Total areas of segregation <0.1 per cent of lift. Stones typically not embedded. All segregation and suspect areas	
	-2	Areas of segregation > 1 m2. Total areas of segregation >1 to 2 per cent of surface. Stones mostly embedded	
	-7	Multiple areas of segregation. Areas of segregation > 2 m2. Total areas of segregation 0.4 per cent of lift. Stones typically not embedded. All segregation and suspect areas	
Rain	1	No rain (or total rain protection provided	1
	0	No apparent rain damage	
	-1	Some minor rain damage, but cleaned and treated	
	-4	Obvious rain damage, not fully cleaned	
	-8	Rain damage with loose mixing on surface. Warranty / paste causing delamination, not fully cleaned or treated	
Cure	1	Surface never dries	Ø -2
	0	Essentially 100 percent moist cure except 0-60 minute surface drying allowed for cleaning just before RCC placement. Surface re-wetted prior RCC placement	
	-2	Frequent periods with large areas > 100 m2 allowed to dry in hot weather	
	-4	Dry and hot when covered with RCC	
	-7	Never Cured	
Maturity	12	Next Lift placed before the "set time" (approximate time allowed for compaction) of the RCC. Typically 40 minutes for dry mix, non retarded. little fly ash mix and, warm temperatures. Typical 2	0
	1	Less than 30 percent of allowed cold joint maturity	
	0	Within specifications	
	-1	Exceed specifications limit by 50 C-hour without special treatment.	
	-2	Exceed specifications limit by 50 to 100 C-hour without special treatment.	
	-6	Exceed specification limits by >150 C-hour without special treatment.	
Surface tightness and condition	1	Uniform light surface and no loose fine sand grains, no dozer marks, no over compaction, no surface cracking	0
	0	Generally light surface with dozer marks and minor loose surface sand grains blown off.	
	-2	Obvious areas where dozer has damaged the surface (green if not compacted). Obvious loose sand, grains reliably removed by cleaning	
	-4	Obvious areas where dozer has burnt oil on surface and extensive sand grains loose on the surface of the first of placed RCC	
Surface flatness	0	Roller drum contacts at least 80 percent of the surface (not more than 20 per cent bridging low spots). All RCC surface contacted by roller with at least one pass.	0
	-2	Roller bridges more than 20 percent of the surface while going over high points, but all RCC surfaces contacted by roller with at least one pass.	
	-7	Roller leaves any area of RCC surface without compaction.	
Delivery	2	All conveyor delivery system with no spillage. No contamination from return belt.	2
	0	All conveyor system with spillage and contamination promptly cleaned.	
	-1	Truck delivery. Careful continuous cleaning.	
	-4	Spillage with loader. Contact damage to surface with tyres and tracks. Attempting to dump material on to RCC surface without compacting	
Other	-8	Re-blading the surface after initial compaction	0
	-8	Adding a thin (less than 10 cm) layer of RCC	
	-8	Major oil or fuel spill > 1m2 not thoroughly cleaned	
	-8	Bedding mix dries up, cuts, before compacting RCC	
TOTAL OF ACCUMULATED JOINTS FOR PLACEMENT AREA(LJQI)			~~-4~~ -6

Figure 4.23 – LJQI Scorecard for area of RCC placed in the primary spillway

493 **TRA.500.009.0001**, 0079 ln 37-46; .0075, ln 7-9.
494 **TRA.500.006.0001**, .0029 ln 31-41.
495 Exhibit 21, **DNR.003.8385**, .8467.
496 Exhibit 310, **SUN.021.006.1405**; Exhibit 311, **SUN.021.006.2078**; **SUN.021.005.8259**.
497 Exhibit 310, **SUN.021.006.1405**, 1406, .1413.

4.325 This low score was largely attributable to segregation. Despite the score of -6 recorded on the LJQI Scorecard, no NCR was raised. Mr Lopez said that an NCR should have been raised if that truly was the score; however, he said that he had re-evaluated the lift and given it a score of -4.[498] That is the opposite of the change that was made on the document.

4.326 The LJQI Scorecard does not record that additional points were added for bedding mix having been applied. However, an electronic database of LJQI scores records that 4 points were added to the lift.[499] It appears that the electronic database was used to generate the summary data and graphs of the LJQI that appeared in the RCC QC Reports. It is concerning that the electronic database does not reflect the paperwork completed at the time of RCC placement.

4.327 The same problem arises in respect of a 280 m long section of RCC in the primary spillway at EL 55.955 m.[500] The LJQI Scorecard shows a total score of -6, which does not include an additional point for bedding mix that was added to the '*surface segregation*' category. The database does not align with the form. Rather than adding 1 point for bedding mix (which would have been consistent with the form), 4 points were added to bring the total score up from -6 to -2.[501]

Bi-monthly RCC Quality Control Reports

4.328 Every two months, an RCC QC Report was prepared by Mr Lopez and Mr Montalvo.[502] Those reports typically explained the sections of the Dam wall in which RCC had been placed during the preceding period. The reports summarised the important milestones that had been reached, how much RCC had been produced and placed, and the results of QA testing. Chronological photographic reports were also included.

4.329 A section of each RCC QC Report addressed the LJQI. It was said, for example in section 7 of the last of those reports to be prepared:[503]

> *The lift joint quality index (LJQI) is a very important quality aspect that has been considered throughout the construction of Burnett Dam. It provides guidelines related to the evaluation for acceptance of lift joints and gives a criterion to follow during the dam construction related to:*
>
> - *Required width of bedding mix to be used (related to the maturity of the lift)*
> - *Cleaning & treatment requirements*

498 Exhibit 324, **LOJ.003.0001**, .0011.
499 **HYT.100.002.0037**.
500 Exhibit 311, **SUN.021.006.2078**, .2079.
501 **HYT.100.002.0037**.
502 **TRA.500.006.0001**, .0027 ln 23-30.
503 Exhibit 38, **SUN.110.003.0001**, .0084 to .0085.

- *Overall quality assessment of the bond between two RCC lifts that define safety factors against stability of the dam[.]*

The principles outlined in the Technical Specifications (S11.10.1) have been followed to determine the LJQI for each individual lift, and very often for different areas related to one lift according to the placement as it happened[.]

In the evaluation of the LJQI, parameters like surface segregation, rain, cure, maturity, surface tightness, surface flatness, delivery & other factors were taken into account. Each parameter was evaluated and added up according to the criterion indicated in Technical Specification in order to get the LJQI.

Each LJQI's evaluation was made before adding bedding mix on the RCC surface affected by segregation or rain, were done. Once the bedding mix was placed on the affected RCC surface to correct these conditions, a new evaluation of real LJQI was made. The points added in this new evaluation of LJQI are indicated in Table 23:

Factor	***Points***	***Conditions/descriptions***	***Points added***
Surface Segregation	*0 or – 1*	*Minor non-connected areas with <1m^2 of segregation. Total areas of segregation <0.1% of lift surface. Stones partially embedded in mortar.*	*2*
	-2	*Areas of segregation 1 to 2m^2. Total areas of segregation 0.1% to 0.4% of surface. Stones mostly embedded.*	*4*
Rain	*-1*	*Some minor rain damage, but cleaned or treated*	*2*
	-4	*Obvious rain damage, not fully cleaned*	*4*

Table 23. RCC treatment with bedding mix – Points added to determine actual LJQI

[O]n the occasions where LJQI was lower due to rain damage or indeed any of the other deleterious factors like segregation on the top of RCC surface, bedding mix was used as corrective in order to improve the bond between two consecutive RCC layers. A detailed comparison of the LJQI with and without bedding mix is provided below.

Nearing completion of the Main Spillway, the width of the bedding mix was increased to the point that it was placed covering the whole of the surface in the last layers of the referred area, providing excellent bond between the layers. Points added in this case were 4.

4.330 The last RCC QC Report contains numerous graphs of LJQI evaluations conducted in different parts of the Dam.[504] Many of the graphs are difficult to read. The text is small and unclear; the markings on them overlap and merge, and it is not evident what each figure seeks to represent nor is it obvious how readers at the time could have been usefully informed by them. This is especially so for the LJQI graphs which are densely populated and small in size. This problem was raised with Mr Hamilton who agreed that a lot of information was presented in the graphs and said that an intimate involvement in the project was needed to understand that information.[505]

Questions about the effectiveness of QA and QC for RCC placement

4.331 The graphs in RCC QC Reports visually represent the numerous LJQI evaluations that were carried out by the RCC Inspectors. While the amount of data collected on the LJQI Scorecards, and the graphical representations of them, might appear impressive at first, when the system is scrutinised more closely, the meaningfulness of the LJQI evaluations is questionable. The same may be said for the RCC Placement ITPs, including because of the sheer volume of paperwork and extent of inspections and check points for which the RCC Inspectors were responsible.

4.332 All inspection points on the RCC Placement ITPS, LJQI Scorecards, and Cold Joint Treatment Checklists were initialled by the RCC Inspectors. Mr Brampton was a graduate engineer with no experience with dams or in construction.[506] Mr Rickert was thought, by Mr Embery, to have had around five years' experience when he worked on the Dam.[507]

4.333 Although the Specification required that there be separation between personnel responsible for QA and those responsible for managing the program, the reporting lines of the RCC Inspectors ultimately led to Mr Embery, whose 'life' on the project was program (i.e. the construction timetable).[508] The RCC Inspectors were subject to pressure from those who were responsible for meeting production deadlines. This was recognised by Mr Neumaier. After the Dam, Mr Neumaier was involved in the design and construction of Susu Dam in Malaysia.[509] That was also an LCRCC dam with a membrane. The LJQI system was used on that project. However, the method of construction for Susu Dam was different. The sloped layer method was used so that only a very short length of the receiving RCC layer was exposed for assessment before the next layer was placed.[510] Mr Neumaier explained that, since the Dam had been built:[511]

> *… we have refined the method of RCC placement because of that subjective method of assessing the quality of lift joints. It was recognised, I guess, as*

504 Exhibit 38, **SUN.110.003.0001**, .0086 to .0099.
505 Exhibit 309, **TRA.510.022.0001**, .0028 ln 19-24.
506 **TRA.510.015.0001**, .0001 ln 4-8.
507 **TRA.500.009.0001**, .0074 ln 27-31.
508 Exhibit 114, **TRA.510.018.0001**, .0028 ln 6.
509 Exhibit 302, **TRA.510.021.0001**, .0025 ln 2-9.
510 Exhibit 302, **TRA.510.021.0001**, .0011 ln 22-32.
511 Exhibit 302, **TRA.510.021.0001**, .0011 ln 22-32 (emphasis added).

> *something that is a little bit vulnerable to interpretation. And of course you can imagine, you have the construction engineer on site and you have the next load of RCC, a truckload, coming on to the dam,* ***and the pressure on him to say yes or no and rejecting it and saying, 'No, you have to place bedding mix' is something very difficult for somebody who's standing on site and you have the contractor breathing down your neck.***

4.334 Some insight into the approach of the construction personnel to QA issues may be gleaned from Mr Embery's evidence. In discussing memoranda prepared by the RCC Engineers that raised QA issues,[512] he characterised at least some of the memoranda '*hysterical*'.[513] According to Mr Embery, the personalities of the RCC Engineers were '*such that they overreacted to certain things*'.[514]

4.335 Those memoranda are not hysterical. They are measured and analytical.

4.336 Mr Embery wrote a memorandum to the RCC Engineers and Mr Lindschau on 11 November 2004, which conveyed frustration.[515] He criticised the RCC Engineers for changing the requirements for cleaning and making '*late requests for cleaning*' and said that such an approach was affecting the motivation of the crew. Mr Embery took issue with the RCC Engineers communicating with construction personnel in these terms:[516]

> *Any concerns Jose/Roberto and others have with regard to the RCC operation should be directed to the Shift Supervisors. Instruction should only be issued by QA/QC or other persons where some person/s is in danger of being injured.*
>
> *If the Supervisor does not respond to the concerns you raise, then they should be passed to Col, Jason or myself.*

4.337 While the evidence reveals irritation with the RCC Engineers, it did not show that the concerns that the RCC Engineers raised were disregarded. For instance, in the memorandum of 11 November 2004, Mr Embery said that additional gear was being procured for cleaning using air and water, despite having understood that this was the least preferred option for dry surfaces. While the remark suggests annoyance (and, perhaps, disagreement with what was being done), it does indicate that the recommendation of the RCC Engineers in respect of cleaning was in fact implemented.

4.338 The Project Manager's attitude is also relevant, and Mr Hamilton expressed his commitment to his QA responsibilities. Mr Hamilton explained that:[517]

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512 Exhibit 31, **SUN.009.002.0147**; Exhibit 32, **SUN.009.002.0203**.
513 **TRA.500.009.0001**, .0101 ln 45.
514 **TRA.500.009.0001**, .0104 ln 4-6.
515 Exhibit 31, **SUN.009.002.0147**, .0197.
516 Exhibit 31, **SUN.009.002.0147**, .0197.
517 Exhibit 309, **TRA.510.022.0001**, .0033 ln 9-24.

> *One thing we tried to do was develop a culture of excellence on that job. We had declarations all around all the crib rooms. I used to get around and talk to these guys once every 10 days. There were 500-plus men on that site. One of the things I wanted to do was talk to the guys on the face once every 10 days, get to their pre-start meetings, get to their crib meetings, those sorts of things, so you can get a feel for, you know, did these people really care about quality, did they really care about producing an excellent product there?*
>
> *I would say at every level of that operation, there was an intent to do a great job, from the guys that were operating the posi-tracks, to the shovel, to the guys that were in the batch plant, and that culture permeated, I believe, through everybody that was there.*

4.339 In an effort to reinforce the importance of quality on the project, the Alliance's Quality Policy Statement[518] was articulated to the workforce and displayed in *'every single crib room'*.[519]

4.340 While provision was made on the RCC Placement ITPs for countersigning by the RCC Engineers, in practice the forms were not signed off contemporaneously. Some RCC Placement ITPs were not signed by Mr Montalvo until nearly a year after the RCC had been placed. Mr Montalvo countersigned many RCC Placement ITPs on 5 August 2005,[520] including one ITP for work done on 29 August 2004 at the base of the primary spillway.[521]

4.341 SunWater conducted a due diligence workshop on site on 11 and 12 August 2005,[522] a short time after Mr Montalvo had signed off many ITPs (many of them filled in by Mr Brampton months beforehand). SunWater's observations from that workshop included that the quality management system had been implemented successfully. SunWater reported that it been provided with '*[e]vidence of exceptional records pertaining to the construction of the works (Inspection and Test Plans, Non Conformance Report etc)*'.[523]

4.342 Mr Montalvo explained that the delay in signing an RCC Placement ITP was because he must have lost it. There are so many that that seems an unlikely explanation. But if it were true, it only shows poor practice.[524] It was suggested to Mr Montalvo that a '*number of these documents*' had been signed by him in August 2005 and that his late signature showed '*a lack of attention*' to something that was his responsibility to

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518 Exhibit 116, **SUN.162.002.0019**, .0066.

519 Exhibit 309, **TRA.510.022.0001**, .0046 ln 31-34.

520 Examples are provided by **SUN.021.005.8654**, **SUN.021.005.8650**, **SUN.021.005.8643**, **SUN.021.005.8639**, **SUN.021.005.8632**, **SUN.021.005.8624**, **SUN.021.006.0027**, **SUN.021.005.8987**, **SUN.021.005.8948**, **SUN.021.006.0098**, **SUN.021.006.0241**, **SUN.021.006.0515**, **SUN.021.006.0551**.

521 See for example, **SUN.021.005.8659**.

522 Exhibit 53, **SUN.020.003.6637**, .6637.

523 Exhibit 53, **SUN.020.003.6637**, .6637.

524 **TRA.500.006.0001**, .0036 ln 31-43.

be diligent about. Mr Montalvo did not agree with the suggestion; however, he could not explain the delays in having signed the forms.[525]

4.343 The documentation that the QA system generated looked impressive. SunWater personnel regarded it in that way in the context of the due diligence exercise in August 2005. The graphs in the RCC QC Reports appear to illustrate the positive outcomes of that system in operation. However, a somewhat different picture emerges when QA measures for RCC placement are more closely scrutinised, for the following reasons:

a. Despite the starting point in the Specification being that construction foremen should check that construction methods were implemented, while QA personnel would check on in situ sampling and laboratory testing,[526] all checks were left to the RCC Inspectors.

b. The RCC Inspectors were junior members of staff, one of whom had no prior dam experience.

c. There were around 40 inspection points for each lot of RCC placed. Each one was required to be checked and initialled by the RCC Inspectors. Across the project, there were thousands of hold and check points for RCC placement primary responsibility for which lay with two junior QA personnel.

d. Hold points on QA forms were not respected. Work was not stopped until the requisite test or inspection was conducted. Instead, RCC placement continued.

e. The subjective nature of assessments that the RCC Inspectors were tasked with making left the QA process open to interpretation. That was problematic in the light of pressures that the RCC Inspectors were likely under from construction personnel seeking to meet target placement rates and in view of the continuous nature of RCC production and placement methods.

f. While the QA forms for RCC placement were designed to be countersigned by a delegate of the Alliance, the forms were seldom endorsed by the RCC Engineers at the time of RCC placement in many instances. Countersignatures of the RCC Engineers were not made for substantial periods after the relevant RCC had been placed and well after an opportunity to remedy any issues had been lost.

g. The LJQI scores recorded in the electronic database do not always align with the scores on the handwritten forms completed by the RCC Inspectors at the time. The database appears to have been used to provide summary information for the RCC QC Reports.

525 **TRA.500.006.0001**, .0037 ln 10-20.
526 Exhibit 21, **DNR.003.8385**, .8501.

h. Rather than being quarantined from scheduling pressures as the Specification had required, the QA team reported to the Construction Manager, whose focus during the project was on the construction program rather than on quality issues.[527] Despite having responsibilities for quality management under the Quality Management Plan, the Construction Manager was not involved in quality documentation and left that work to personnel who reported to him.

i. Some quality issues with RCC lifts were evident throughout construction.

527 **TRA.500.009.0001**, .0083 ln 38-41.

Chapter 5 – Sliding stability of the Dam

Introduction

5.1 The focus of this Chapter is explaining the structural and stability issues relating to Roller Compacted Concrete (**RCC**) that have been identified since 2013 and the possible explanations for those. It was GHD's recent work that gave rise to doubts about whether the RCC lift joints have sufficient sliding resistance. A technical review panel (**TRP**) reviewed GHD's stability assessment, which adopted residual shear strength values from testing on RCC cores in 2015 and 2019.

5.2 In hypothesising about why those values were lower than in the detail design, Glenn Tarbox considered that problems were experienced in the construction of Paradise Dam (**the Dam**). Section 3.2 of TRP Report No. 2 discusses those problems. No party called evidence to contradict Mr Tarbox's view, although Hydro Tasmania says the preponderance of evidence from those directly involved in the design and construction of the Dam was that any issues with the lift joints were identified and addressed.[1]

5.3 The problems which Mr Tarbox identified were drawn by him from various memoranda provided to TRP members by SunWater. Those memoranda were written by Jose Lopez and Roberto (sometimes known as Robert) Montalvo.[2] They show the issues raised with senior construction personnel during the course of construction. Those memoranda raise quality control issues with RCC placement.

5.4 This Chapter commences with a discussion of quality issues that are relevant to the Dam's sliding stability. The construction memoranda are then considered, along with the extent to which the problems they identify were remedied. Procedures to certify that the Dam had achieved its design intent are canvased, before GHD's findings and criticisms of them are discussed. Finally the Chapter deals with the whether the Dam is currently stable and the root causes of the identified issues.

Were there problems with construction that went unremedied?

Relevance to sliding stability

5.5 In a 2017 publication of the US Bureau of Reclamation (**USBR**), the following conditions are listed as necessary for achieving good bond between RCC lifts:[3]

> 1. *Providing sufficient paste and mortar volume and workability of the RCC mixture.*
>
> 2. *Controlling segregation during placing.*

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1 **HYT.008.0001**, 0042 [132].

2 Exhibit 31, **SUN.009.002.0147**; Exhibit 32, **SUN.009.002.0203**.

3 USBR, *Design and Construction Considerations for Hydraulic Structures Roller-Compacted Concrete*, 2nd ed (2017), 29 [4.3] <https://www.usbr.gov/tsc/techreferences/mands/mands-pdfs/RCCManualFinal09-2017-508.pdf>.

3. *Providing adequate compaction with the vibrating roller.*

4. *Providing good surface cleanup of the lift.*

5. *Placing a bonding layer of mortar or concrete between lifts of RCC.*

6. *Placing RCC at a high rate, reducing the exposure time between lifts.*

7. *Maintaining good construction practices for mixing, placing, compacting, and curing RCC.*

Paste and workability

5.6 The importance of sufficient paste and workability of the RCC mixture is consistent with the evidence of Mr Tarbox.[4] The USBR publication explains:[5]

> *Having adequate paste and mortar provides the 'glue' needed to bond layers together. Insufficient paste leads to segregation, rock pockets, and an inability to properly compact the full thickness of the RCC lift. Voids present at the bottom of a lift of RCC, caused by either segregation or lack of compaction, reduce the cohesion of RCC to essentially zero. This was a problem in some early RCC dams, leading to excessive seepage and lack of bond.*

Segregation, compaction and density

5.7 The properties of a fresh RCC mix affect the ability to compact the full lift. The most important property of fresh RCC is that the mix has minimum segregation. If large aggregate segregates, that can lead to '*poor bond between subsequent lifts of RCC, increased volume of voids between aggregates, and may result in excessive seepage between lifts*'.[6] Segregation in a fresh mix is typically caused by the moisture content being too low, combined with poor placement techniques. Segregation and density are interrelated. RCC that has been poorly compacted and is segregated will return poorer density readings than RCC where good compaction has eliminated segregation.

5.8 The density of the RCC and any voids in a lift influence a dam's performance. Density is governed by the density of the fresh RCC mix and the degree to which air voids are removed from it by compaction. If a lift of RCC is not fully compacted, voids along lift joints may result in excessive seepage and poor bonding.[7] Timothy Dolen

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4 Exhibit 100, **TAG.001.0001**, .0006 [23] to .0011 [36].

5 USBR, *Design and Construction Considerations for Hydraulic Structures Roller-Compacted Concrete*, 2nd ed (2017), 29 [4.3] <https://www.usbr.gov/tsc/techreferences/mands/mands-pdfs/RCCManualFinal09-2017-508.pdf>.

6 USBR, *Design and Construction Considerations for Hydraulic Structures Roller-Compacted Concrete*, 2nd ed (2017), 20 [4.1.2] <https://www.usbr.gov/tsc/techreferences/mands/mands-pdfs/RCCManualFinal09-2017-508.pdf>.

7 USBR, *Design and Construction Considerations for Hydraulic Structures Roller-Compacted Concrete*, 2nd ed (2017), 20 [4.1.4] <https://www.usbr.gov/tsc/techreferences/mands/mands-pdfs/RCCManualFinal09-2017-508.pdf>.

referred to this when he pointed to the deficiency of the Lift Joint Quality Index (**LJQI**) in not considering the base of the upper layer of RCC (discussed above).[8]

5.9 On site quality control testing for density has two purposes: a) it confirms the design assumptions for unit weight of the structure used in stability calculations; and b) measurements of density indirectly assess the compaction of the lift and at the joint interface. If RCC is not properly compacted in the lower portion of a lift, the result may be low (or no) bond strength with the underlying RCC layer.[9]

5.10 According to Mr Dolen, compaction is the construction problem with the biggest impact on the friction angle between RCC lifts.[10] While giving evidence concurrently with Dr Ernest Schrader, Dr Paul Rizzo, James Willey and Stephen Tatro, Mr Dolen referred to the following chart:[11]

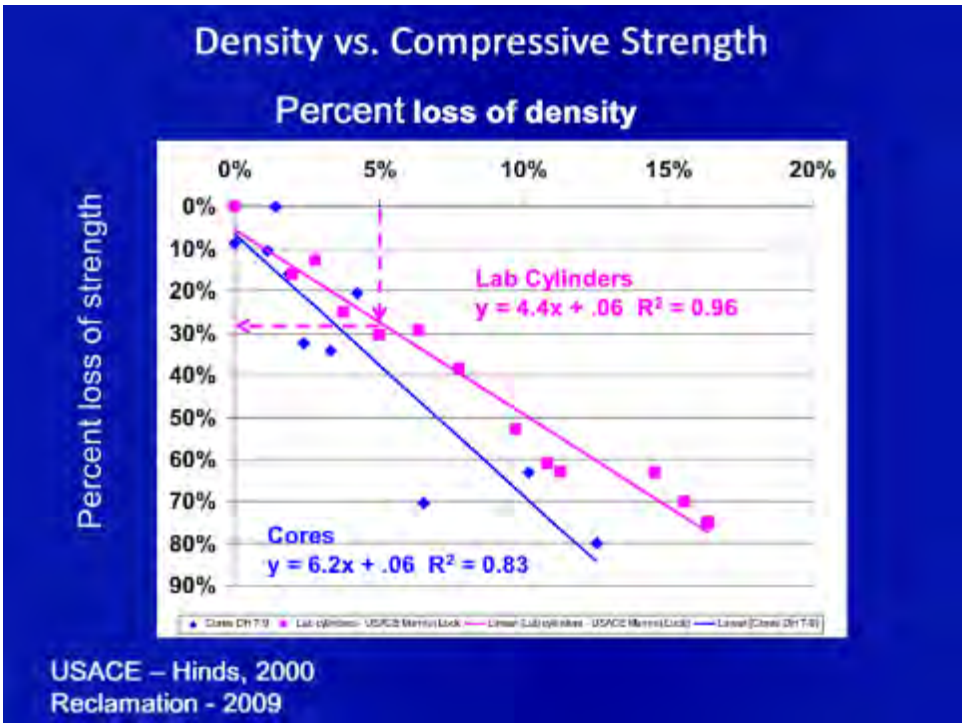

*Figure 5.1 – Effect of entrapped air from lower density (i.e. lower compaction) on compressive strength of RCC cylinders and cores. (Exhibit 69, **ICO.002.0001**, .0142 Figure 7.12)*

5.11 The following exchange occurred between Mr Dolen and Mr Horton QC, Senior Counsel Assisting the Commission:[12]

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8 **TRA.500.009.0001**, .0046 ln 18-22.

9 USBR, *Design and Construction Considerations for Hydraulic Structures Roller-Compacted Concrete*, 2nd ed (2017), 53 [5.12.3] <https://www.usbr.gov/tsc/techreferences/mands/mands-pdfs/RCCManualFinal09-2017-508.pdf>.

10 **TRA.500.009.0001**, .0021 ln 3-8.

11 **TRA.500.009.0001**, .0021 ln 15-37.

12 **TRA.500.009.0001**, .0021 ln 15-37.

MR DOLEN: There is a graph shown in the ICOLD report of 2020 that I put together with Mr Hinds - this goes back several years ago - that shows the effect of compaction on strength. For every per cent loss of compaction, every per cent of density you could have achieved with full compaction, you are going to lose between 5 and 6 per cent of your compressive strength. So a 1 per cent loss is about a 5 or 6 per cent loss of strength. If you have a 5 per cent loss of density, you are looking at a 25 to 30 per cent loss of strength that could have been achieved if you had full compaction.

MR HORTON: Just explain … how, then, does that analysis bear particularly on friction, loss of strength, but is it frictional strength?

MR DOLEN: You can imply that. I think you are losing almost all direct tensile strength if you have a 25 per cent loss of compressive strength, you are losing most of your direct tensile strength, and you would be losing, I think, significant percentages of shear strength and degradation of the friction angle.

5.12 And later:[13]

MR DOLEN: Obviously I'm the one who is concerned about the density and the effect on strength and the effect on friction. The graph that I talked about cites that it is friction angles around 45 degrees for well-compacted materials, and then it drops down with small voids that were - these are tested samples. This is the tested samples. It then goes down to - I don't have that number in front of me, the shear value, the friction value, when there is a higher percentage of voids.

MR HORTON: Could we show you that, because we can identify it at the same time. …. Are they the tables to which you were making reference?

MR DOLEN: Yes, those are [see Figures 5.1, 5.2 and 5.3]. On the one on the left, "Density vs. Compressive Strength", the lab cylinders were done in our laboratory with Mr Hinds. He took very great care to get the density uniform through those values. If it was 20 per cent or 15 per cent loss of density, that was throughout the entire specimen from top to bottom. That's very important. So those are good numbers.

The cores you see there were taken from a dam in Oregon with fairly low cementitious content, and it was extremely difficult to get those numbers, because it's difficult to get a specimen representative of that. The loss of density you see here is mostly at the joint. There is a little bit different from uniform throughout, so the strength results are affected by small areas of low density at the joint, especially the low ones you see there.

5.13 In relation to the chart that follows, Mr Dolen said:[14]

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13 **TRA.500.009.0001**, .0026 ln 30 to .0027 ln 15.
14 **TRA.500.009.0001**, .0027 ln 29 to .0028 ln 8.

As it reflects to friction angles and you take a look to the right here on sliding resistance, it shows values of 35, 43 and 53 from a rough surface, a surface with voids and a surface with rock pockets. The ones with rock pockets are larger voids from lack of compaction.

…

That showed a change in the phi angle, the friction angle only, related to the presence of voids. I will also state that these curves point down beyond where the data points are and shouldn't be considered in this.

MR HORTON: So a compaction problem gives rise to voids, you are saying –

MR DOLEN: Voids give rise to decrease in friction properties.

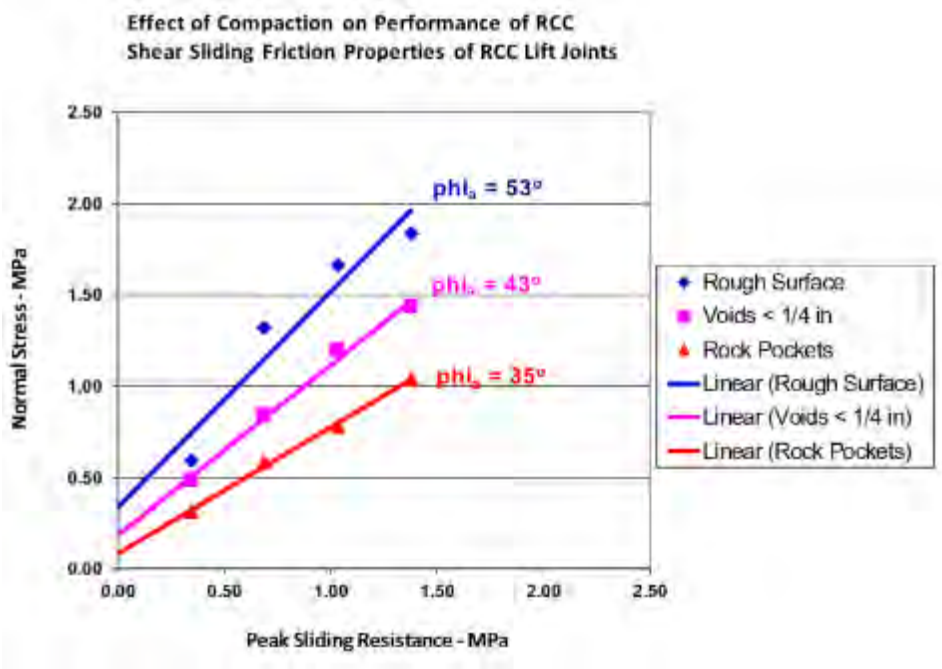

*Figure 5.2 – Graph showing the effect of compaction on residual sliding properties of RCC layer joints. (Exhibit 69, **ICO.002.0001**, .0143 Figure 7.14)*

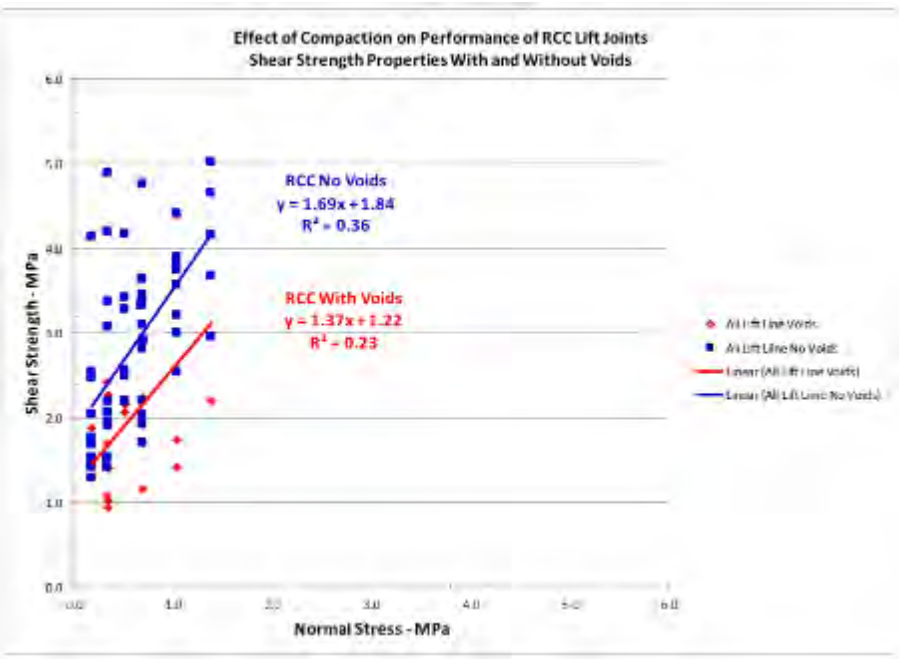

*Figure 5.3 – Chart of shear strength properties from several USA projects with full compaction and with identified voids. (Exhibit 69, **ICO.002.0001**, .0142 Figure 7.13)*

Cleaning and surface treatment

5.14 Mr Tarbox explained the importance of cleaning and curing RCC lifts:[15]

> *Cleaning of the lifts*
>
> *23. The foundation upon which the contractor starts to build an RCC dam and all successive lifts of RCC must be properly prepared. The surface must be absolutely clean. It must be cleaned of all loose impediments, all loose rock, all soil, dust, mud, etc. It must be washed using high pressure water, blow dried and/or vacuumed to remove free standing water. When ready to place material on the surface, the surface must be in a condition called 'saturated super-dry'. This essentially means that there is no freestanding water, but the surface remains damp. The rock or a previous lift should not cause moisture to be drawn out of the layer being placed by sapping water out of the new RCC. Sapping the RCC of its moisture can lower the water to cement ratio and lower the strength of the concrete. Thus, it is very important that the preparatory steps be done consistently, continuously, and correctly.*
>
> *Curing of the lifts*
>
> *24. Once a lift is placed, compacted and the surface is sealed - it is important that it be cured, not with a curing compound (because that is a bond-breaker) but with water to keep it wet. Evaporative cooling of the cure water helps to exhaust heat out of the placed RCC. Concrete has a*

15 Exhibit 100, **TAG.001.0001**, .0006 [23] to .0011 [36].

property of RCC known as the 'adiabatic temperature rise' where the exothermal reaction of the chemicals in cement create heat. It is desirable to enhance the reduction in that temperature and enhance the evacuation of that heat. Water curing is an effective means of reducing the heat gain in fresh RCC. Water curing must be continuous. If the surface is allowed to dry out, the surface begins to form a layer which, if left too long, will actually not be conducive to bond with the next layer, whether referencing bedding mix or the RCC. Water curing must be applied uniformly, consistently, universally in all locations and throughout the entire construction period.

25. Curing not only helps to take away some of the heat of hydration, but it helps to mitigate the surface from drying out and cracking. Fresh concrete will shrink as it cures. As it dries out, it shrinks and may crack. It is proper practice always to protect against cracking in concrete.

…

Restoring the surface

36. Once a lift is spread and compacted and it is being water cured, it should never be allowed to dry out. It is cured continuously until the next layer is deposited. If it dries out, the surface must be prepared with high pressure water blasting or the use of a bedding mix. The surface cannot be contaminated and must be kept very clean. Nothing should be spilled on it. There should not be tracks, dust, dirt, mud or any other foreign substance such as fuel or oils that come in from off the site onto the lift. If that occurs, cleaning of the lift is essential before any fresh material is dumped and compacted: it must be restored to pristine condition including removal of unacceptable RCC.

Rate of placement

5.15 The time interval between placing lifts can affect the bond achieved between lifts.[16] The aim is to achieve a rate of placement that allows the next lift to be placed on a joint that has not set. That maximises bonding between lifts by '*knitting the two layers together and allowing recompaction of the lower lift of RCC*'.[17]

16 USBR, *Design and Construction Considerations for Hydraulic Structures Roller-Compacted Concrete*, 2nd ed (2017), 30 [4.4] <https://www.usbr.gov/tsc/techreferences/mands/mands-pdfs/RCCManualFinal09-2017-508.pdf>.

17 USBR, *Design and Construction Considerations for Hydraulic Structures Roller-Compacted Concrete*, 2nd ed (2017), 30 [4.3] <https://www.usbr.gov/tsc/techreferences/mands/mands-pdfs/RCCManualFinal09-2017-508.pdf>.

Recognition of constructability issues during the tender stage

5.16 During the tender phase, the Hydro Tasmania consortium identified constructability issues and risks associated with RCC placement. In April 2003, the following preliminary management strategies were suggested to address identified issues and risks:[18]

RCC PLACEMENT Constructability Issues or Construction Risks	Preliminary Management Strategy
Ogee Shaped Crest Constructability Issue • This is difficult to construct with RCC on the downstream face above RL 55.	• Constructing top of spillway in 300mm steps, and form ogee shape with a second stage shotcrete process.
RCC Mix Performance • Segregation. • Compaction difficulties. • Inter-layer cohesion. • Strength requirements.	• Add mixtures. • Extra jointing procedures. • Modify compaction process. • Introduce cooling method.
Ambient Temperature Constructability Issues • More costly cooling method. • May have to cease placement until temperature falls. • Impact on set time of layers.	• Procedures in place to examine cooling and pouring times Retardant to delay setting. • Place exclusively in winter. • Avoid placing between 2pm and 6pm.
Constructability Issues Related to Formwork to Vertical Upstream and Stepped Downstream GE RCC surfaces • Coordination and production with RCC placement and sequential lifting of forms.	• Modify operations. • Use scope placement method where appropriate.

RCC PLACEMENT (Cont'd) Constructability Issues or Construction Risks	Preliminary Management Strategy
Meeting Specifications for Layer Interface Joints • Difficulty with achieving adhesion and extra work due to time delays or cold joint effects.	• Vary outputs to facilitate optimum intervals between lifts.
Wet Weather Risk • Need to stop work on RCC placement if rain becomes more than drizzle. • Unseasonal high flows.	• Suspend specific parts of work. Continue with restart procedures for mix delivery and joints. • Flexibility in construction methods. • Evacuation procedures in place.
Breakdown of RCC Equipment • Delays in RCC placement and overall program.	• Backup equipment in place.

5.17 The possibility of using the sloped layer method was also considered for constructing the spillway. It was not ultimately employed.

Memoranda by the RCC Engineers

Background

5.18 This section discusses the construction memoranda that Mr Tarbox considered in concluding that construction problems were the cause of the poor condition of the Dam. Those memoranda were written by Mr Lopez and Mr Montalvo (together, **the**

[18] Exhibit 251, **HYT.510.004.0001**, .0081 to .0082.

RCC Engineers). At least after the involvement of Walter Construction Group Limited (**Walter**) ceased, they were employed by Macmahon Contractors Pty Ltd (**Macmahon**).[19] They were the only on site personnel with RCC experience: Mr Lopez had greater experience than did Mr Montalvo. Mark Hamilton said that the Burnett Dam Alliance (**the Alliance**) drew on the experience of Dr Schrader, Mr Lopez and Mr Montalvo, who were involved in the project '*specifically to make sure that we did train our crew and get it right*'.[20]

5.19 Mr Lopez and Mr Montalvo carried out the tasks allocated in Section 11.21 of the Specification to the '*RCC Quality Control Engineer*', '*Quality Control Engineer*', '*RCC Quality Engineer*' and the '*Engineer*'.[21] The Specification directed that one of them was to remain on site full time.[22] This occurred.[23]

5.20 The RCC Engineers produced memoranda in the course of the Dam's construction. Mr Lopez said that about 80 of them were written.[24] He understood that his role included identifying technical issues with construction, which he raised in the memoranda.[25] Mr Lopez said that initially the people working on the project had a lack of knowledge about RCC so he had to explain it to them.[26]

5.21 The RCC Engineers were not part of the construction personnel line management and did not have any power to direct workers on site, or to direct the construction method. As discussed elsewhere in this report, Bruce Embery's memorandum of 11 November 2004[27] to Mr Lopez, Mr Montalvo and Colin Lindschau shows that divisions emerged between Quality Assurance (**QA**) and construction personnel. In that memorandum, Mr Embery directs that instructions were not to be given by the RCC Engineers directly to workers except in absolute emergencies. Instead, the RCC Engineers were to communicate with more senior construction personnel.

5.22 The work of the RCC Engineers was fundamental. QA measures began with the observations that the RCC Engineers made in the memoranda. Those memoranda typically identified issues and the nature of particular problems, included photographs, and explained how problems could either be remediated or avoided in the future.

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19 **TRA.500.006.0001**, .0025 ln 31-33; Robert Montalvo, LinkedIn Profile, LinkedIn <https://www.linkedin.com/in/robert-montalvo-43362512/>; Jose Eduardo López Moreno, LinkedIn Profile, LinkedIn <https://www.linkedin.com/in/jose-eduardo-lópez-moreno-b85723112/>.

20 Exhibit 309, **TRA.510.022.0001**, .0011 ln 36-32.

21 See, for example, these terms used interchangeably in exhibit 21, **DNR.003.8385**, .8480 to .8481.

22 Exhibit 21, **DNR.003.8385**, .8480.

23 Exhibit 309, **TRA.510.022.0001**, .0025 ln 4-5; **TRA.500.006.0001**, .0026 ln 38 to .0027 ln 4.

24 **TRA.500.011.0001**, .0005 ln 8-12.

25 **TRA.500.011.0001**, .0005, ln 5-12.

26 **TRA.500.011.0001**, .0004, ln 36-42.

27 Exhibit 31, **SUN.009.002.0147**, .0197. This memorandum is discussed later in this Chapter under the heading '*11 November 2004 Memorandum: 'RCC Production - Discussion of 10 November 2004"*.

5.23 Dr Schrader described the work of the RCC Engineers in the following way:[28]

> *I insisted that there be someone there full time, any hour, any minute that RCC was being placed. In fact, they covered it so well, any time any work was being done on the project, one of those two men were there and they were responsible for the inspection and the quality inspection and quality report. They were responsible for this particular report, and every day, at the end of the day, they had a summary of what was done.*
>
> *Their reports are extraordinarily thorough about whether there was damage that occurred, for example, with rain that happened or trucks damaged the surface. They show the damage. They identify where it was. They then talk about what they did to correct the situation, had photographs of it being cleaned, if it was damaged, and photographs of bedding mix that was put on it to glue it back together. It was very well documented throughout the whole job.*

Chronology

15 June 2004 Memorandum: 'Trial Placement construction report – days 4th to 9th of construction'

5.24 On 15 June 2004, Mr Lopez wrote a memorandum with the subject '*Trial Placement construction report – days 4th to 9th of construction*'.[29] It was addressed to Matt Landers, the '*Area Manager RCC*' at the time,[30] and copied to (among others) Mr Hamilton, Mr Herweynen, Timothy Griggs, Mr Embery and Mr Montalvo as well as to Dr Schrader. Written early in construction, it was focussed on placement of RCC in the trial embankment.

5.25 Mr Lopez made observations about RCC placement which had been communicated to '*Engineers and Personnel involved with RCC dam construction*',[31] including:[32]

> *2. RCC delivery by trucks.*
>
> *Only three (3) trucks were used for RCC delivering from Aran 250 pug mill to Auger max. Lack of continuous supply of RCC caused additional exposition time of RCC edges and delay in compaction (up to 15 min), but within 40 min required by Technical Specifications.*
>
> *…*
>
> *3. RCC delivery by Crater Crane.*
>
> *In order to improve RCC productivity, and to help the dozer spread and control RCC segregation, the elephant trunk should dump the RCC*

28 Exhibit 127, **TRA.510.006.0001**, .0014, ln 5-21.
29 Exhibit 31, **SUN.009.002.0147**, .0147.
30 Exhibit 116, **SUN.162.002.0019**, .0030.
31 Exhibit 31, **SUN.009.002.0147**, .0148.
32 Exhibit 31, **SUN.009.002.0147**, .0148-.0149 (emphasis added).

continuously and in windrows no more than 400 mm high, covering all of the placement area and avoiding creating piles that cause segregation. It is important to control this procedure in order to avoid segregation that caused low densities specially in the bottom of some layers.

…

4. RCC Spreading.

A Positrack continues the RCC spreading activities. The Positrack Operator has increased his skills, increasing the RCC productivity. I hope his skill increase even more in order to improve productivity. He does not have too much time to do it.

…

6. RCC Curing.

RCC surface curing was done in a good way, applying fog of water continuously and in an opportune manner. However, it is important to prevent the RCC layer surface from being too damp before spreading RCC on it. An excess of moisture decrease shear strength along the surface and affects the stability of the dam.

Lack of curing affects the strength of the RCC surface, increasing debris and cleaning requirements. The RCC surface shall be in SSD condition.

7. RCC Cleaning activities.

RCC surface cleaning had been done in a good way using the Vacuum. Cleaning activities were done opportunely after RCC compaction when it is easy to remove debris and segregated material along the contact with dam foundation. Brushes instead of the vacuum were used in some cases in order to keep the RCC surface from being damaged by the wheels of the vacuum truck.

…

10. RCC layer thickness.

Before compaction RCC thickness range between 380 and 400 mm. After compaction thickness ranges to 280 to 310mm.

The RCC layer placed last 11 June increased its thickness after compaction. In some levels it reached up to 380 mm, affecting RCC field densities, specially in the bottom of the layer placed on 11 June 04.

…

12. Quality control.

…

> ***Low densities were detected*** *by the double probe nuclear gauge densimeter last 16 June 2,004 on level 73.6 RCC arrived with low moisture to the placement area. Segregation was increased also because during placement activities the Crater crane dumped the RCC with the elephant trunk from heights that ranges from trunk 700 -1 000 mm high over the previous RCC surface. Instructions were given to the person controlling the elephant trunk movements, who is also in charge of communicating to the Creter crane operator. The field engineer and the placement Foreman need to improve the control of this activity, it is very important that they do so. It is always better to prevent than to correct.*
>
> *Two test pits were dug in this layer in order to do a visual inspection of the placed RCC, immediately after this layer was compacted (Top level 73.6), thus affecting the visual inspection we did.* ***Segregation was observed in both test pits, but in an isolate way. RCC was not removed due to this situation****. The bond between layers was excellent in the areas where no segregation was observed.*

19 June 2004 Memorandum: 'Aggregate stockpile handling'

5.26 On 19 June 2004, Mr Lopez and Mr Montalvo wrote a memorandum with the subject '*Aggregate stockpile handling*'.[33] It was addressed to John Hunt, who worked for Wagners Quarries Pty Ltd (**Wagners**) as the '*Materials Manager*' at the time,[34] and copied to (among others) Mr Herweynen, Mr Griggs, Mr Embery and Dr Schrader. It focused on how to manage the aggregate stockpile to avoid segregation. A connection was drawn between:[35]

a. low moisture content of the aggregate increasing segregation

b. segregation of the aggregates and decreasing densities on the trial placement area.

19 June 2004 Memorandum: 'Low densities Vs. high densities, lessons learned'

5.27 Mr Lopez wrote a memorandum on 19 June 2004 with the subject '*Low densities Vs. high densities, lessons learned*'.[36] It was addressed to Mr Landers and copied to (among others) Mr Hamilton, Mr Herweynen, Mr Griggs, Mr Embery and Mr Montalvo as well as Dr Schrader. Mr Lopez compared RCC production and placement on 16 and 18 June 2004 and the improved segregation control on the latter date. He described the factors necessary to control segregation of RCC:[37]

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33 Exhibit 31, **SUN.009.002.0147**, .0153.
34 Exhibit 116, **SUN.162.002.0019**, .0030.
35 Exhibit 31, **SUN.009.002.0147**, .0153.
36 Exhibit 31, **SUN.009.002.0147**, .0155.
37 Exhibit 31, **SUN.009.002.0147**, .0155.

As it is possible to see, to get higher RCC field densities in Burnett RCC dam project and in general in any RCC gravity dam is determined by diverse factors that are necessary control during RCC placement works:

1. *RCC moisture*
2. *Placement procedure by Creter crane and its elephant trunk.*
3. *Speed of the roller (Maximum 2.5 km/h or 6.9 m/10 sec for practical control purposes on site).*
4. *Opportune support from the lab in order to correct RCC moisture during RCC placement works.*

Other factor that is necessary to take into account to keep RCC field densities from decreasing, are:

1. *Control of RCC layer thickness (Max 300 mm after compaction).*
2. *Control of compaction within time limits after RCC production at Aran 250 pug mill.*
3. *Improve handling of stockpile aggregate during pug mill feeding*

3 July 2004 Memorandum: 'Aggregate production - Action required'

5.28 Mr Lopez's memorandum on 3 July 2004 had the subject '*Aggregate production - Action required*'[38] and recipients of it included Mr Hamilton, Mr Herweynen, Mr Embery and Mr Montalvo. Aggregate fines content continued to be higher than specified. The problem needed to be solved, it was said, or it would affect '*the efficiency of the cement (strength per kg of cement added) and can increase lift surface cleaning requirements during dam construction*'.[39]

6 July 2004 Memorandum: 'Contamination of dam foundation'

5.29 In a memorandum dated 6 July 2004 with the subject '*Contamination of dam foundation*',[40] Mr Lopez identified concrete (which he was unable to distinguish as either conventional concrete waste or bedding mix) that had been poured on the dam foundation without any control. He recommended that the concrete be removed.[41] Other than general statements that problems identified were or would have been fixed,[42] the Commission has not seen evidence showing that this problem was rectified.

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38 Exhibit 31, **SUN.009.002.0147**, .0157.
39 Exhibit 31, **SUN.009.002.0147**, .0157.
40 Exhibit 31, **SUN.009.002.0147**, .0159.
41 Exhibit 31, **SUN.009.002.0147**, .0159.
42 **TRA.500.011.0001**, .0005 ln 47 to .0006 ln 2; **TRA.500.009.0001**, .0103 ln 19-22; Exhibit 309, **TRA.510.022.0002**, .0019 ln 33 to .0021 ln 4.

14 July 2004 Memorandum: 'Aggregate production'

5.30 On 14 July 2004, Mr Lopez wrote a memorandum titled '*Aggregate production*'[43] which was addressed to Mr Hunt and Brian Langridge, the '*Area Manager - Materials*'[44] at the time. Other recipients included Mr Hamilton, Mr Embery, Mr Herweynen and Mr Griggs. Mr Lopez said that a change in the '*stockpile aggregate gradation'* during crushing had been detected in the period 7 to 9 July 2004, which led to the aggregate having an excess of coarse material. He recommended the aggregate be stockpiled but mixed with aggregate that was within specified grading requirements.[45]

16 July 2004 Memorandum: 'RCC Low densities on 15 July 2004'

5.31 The memorandum of 16 July 2004, written by Mr Lopez, was titled '*RCC Low densities on 15 July 2004*'[46] and was addressed to Mr Embery with copies to others.

5.32 Mr Lopez noted that low densities were obtained in a '*small area*' of the RCC lift at EL 51.45 m adjacent to chainage 635 m. RCC placement was undertaken in two stages. The first stage met compaction requirements, but low densities were detected at the bottom of the lift in the second stage in an area of 5 m x 5 m. When test pits were dug, segregation was '*obvious*' but not continuous.[47] In some of the test pits, there was still good bond between the layers. Field densities were measured at 11 locations and density at the bottom of the lift was described as 'low' at five of those locations.[48]

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43 Exhibit 31, **SUN.009.002.0147**, .0161.
44 Exhibit 115, **SUN.175.006.0009**.
45 Exhibit 31, **SUN.009.002.0147**, .0161.
46 Exhibit 31, **SUN.009.002.0147**, .0163.
47 Exhibit 31, **SUN.009.002.0147**, .0164.
48 Exhibit 31, **SUN.009.002.0147**, .0163.

Photo 1. RCC placement stages on the 2nd RCC layer placed on 15 July 04 next to Chainage 635 (EL 51.4)

*Figure 5.4 – Second stage of RCC placed on 15 July 2004 in which low densities were detected. (Exhibit 31, **SUN.009.002.0147**, .0163)*

5.33 The Specification[49] (as clarified by Dr Schrader[50]) said that in areas where small compaction equipment was used, the average of density readings from the top, middle and bottom of a layer was required to exceed 93% of theoretical air free density (**TAFD**). The minimum permitted reading at any location was 89% TAFD. In areas where large rollers were used for compaction, the requirements were a minimum average of 96% TAFD, while the minimum single reading was 92%. The Specification stated that '*no individual reading of less than [the minimum] will be allowed in any part of any lift*'.[51] The Specification did not confer a discretion on the RCC Engineers to leave in place areas of RCC with density lower than the specified minimum. The requirement was strict; no RCC below the minimum specified density was to remain *in situ*.

5.34 The densities that were described as low by Mr Lopez in his memorandum on 16 July 2004 were less than 89%. The averages were within specified limits.[52] Rather than remove the areas where low densities were detected, Mr Lopez reviewed the stress demands in that location (i.e. the secondary spillway):[53]

> *Segregation decreases the RCC compressive and tensile strengths and it also decreases its strain capacity. Taking into account these concerns, a review of the stress demands was done according to stability analysis requirements for the secondary spillway - main section (CH 515 - CH 770) and also taking into*

49 Exhibit 21, **DNR.003.8385**, .8465 to .8466.
50 Exhibit 286, **HYT.002.0001**.
51 Exhibit 21, **DNR.003.8385**, .8465.
52 Exhibit 31, **SUN.009.002.0147**, .0164.
53 Exhibit 31, **SUN.009.002.0147**, .0165.

account the effect of the low density (93.9%) on predicted compressive and tensile strengths (85% of expected compressive -15 MPa, and. tensile strength - 1.3 MPa. Safety factors are still very high for the different design demands in that specific area of RCC placement.

5.35 Mr Lopez said '*it was concurred that it is "tolerable", but not really "good", to leave the material in place*'.[54]

5.36 Non-Conformance Report (**NCR**) 57 was raised in respect of the low density RCC that was the subject of Mr Lopez's memorandum. The recorded rectification of the non-conformance was that no action was necessary as a result of Mr Lopez's evaluation of stress demands in that location. The NCR was closed out on the basis of the evaluation.[55]

5.37 This was contrary to the Specification. While consideration of localised stress demands was permitted when evaluating whether to leave an area of RCC in place if the minimum LJQI score was not met, that same evaluation was not one permitted in respect of poorly compacted RCC. Mr Lopez's justification for leaving poorly compacted material in situ is concerning in light of Mr Dolen's evidence. Mr Dolen's main concern in connection with the Dam's stability was:[56]

[W]hether or not we have full compaction of all of the lifts. Based on my observations, there is reasonable doubt as to whether the lifts have been fully compacted, and there is sufficient to make me sufficiently concerned about a lift or successive lifts that may not have sufficient capability for these events, these loadings.

5.38 Mr Dolen was asked what the possible root cause of the Dam's instability might be, assuming such instability. Having identified that poor compaction as a possible explanation,[57] Mr Dolen said:[58]

I think this mix had a lower workability when compared to other mixes or types of concretes, which contributed to the inability to achieve that compaction.

COMMISSIONER BYRNE: Is that related to the low cementitious content or the materials properties to which Dr Schrader is referring?

MR DOLEN: I think it's the mix design that had a Vee-Bee consistency time that I would not have used if I wanted to achieve full compaction of all the lift joints. Where I use this, I've been using that for - since the mid-1980s, I have not had a dam using those kinds of consistency tests and workability levels that have had any issues with porosity on the lift joints.

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54 Exhibit 31, **SUN.009.002.0147**, .0165.
55 **DNR.006.3434**, .3517.
56 **TRA.500.008.0001**, .0084 ln 20-26.
57 **TRA.500.009.0001**, .0021 ln 3-8.
58 **TRA.500.009.0001**, .0064 ln 9-23.

5.39 Mr Herweynen was asked about the summaries of field density readings in the *'RCC Quality Control Reports'* (**RCC QC Reports**). For instance, readings from the primary spillway summarised in the last RCC QC Report, showed 5.7% of the data were below the 92% TAFD compaction requirement. In explaining why that might have occurred, Mr Herweynen referred to the minimum of 89% allowed for individual readings.[59]

5.40 The memorandum that Mr Lopez wrote on 16 July 2004 casts doubt over the correctness of that explanation. In this location at least, RCC at the bottom of a layer was left in place even though density readings were less than the minimum 89% TAFD that was permitted.

5.41 Mr Lopez listed four causes of segregation:[60]

a. A new unskilled Creter crane operator and a new worker managing the placement of RCC by that crane.

b. A ten to 15 minute break in placement between stages one and two during which RCC on the Auger Max hopper dried.

c. Lack of segregation control of RCC going into the Auger max.

d. Improper placement of RCC by creating '*excessively high piles and increasing the distance between windrows or overlapping them*'. Once spreading of RCC began, the segregation was covered but not fixed.

5.42 Eleven actions to avoid a repeat situation were set out addressing the training of new workers, proper RCC placement techniques, avoiding interruptions to RCC placement activities, and improving moisture content.[61]

5.43 Mr Embery gave evidence that this memorandum was part of getting the right people on the project. Mr Embery remembered '*very clearly*' that people involved in some of the poor quality work identified in the memorandum were not employed on the project much longer. There were significant personnel changes as a result of this early work.[62]

17 July 2004 Memorandum: 'RCC low densities – RCC LOW DENSITIES – METHODOLOGY SUGGESTED TO AVOID LOW DENSITIES'

5.44 In a memorandum dated 17 July 2004, Mr Lopez and Mr Montalvo sought to summarise previous memoranda in order to avoid low densities in RCC.[63] The document also included advice from Dr Schrader on how to control RCC

59 **TRA.500.014.0001**, .0011 ln 6-30.
60 Exhibit 31, **SUN.009.002.0147**, .0165.
61 Exhibit 31, **SUN.009.002.0147**, .0165.
62 **TRA.500.009.0001**, .0098 ln 38 to .0099 ln 4.
63 Exhibit 31, **SUN.009.002.0147**, .0167.

segregation. The memorandum was addressed to all RCC staff, with copies to Mr Hamilton, Mr Embery, Mr Herweynen and Mr Griggs.[64]

5.45 The RCC Engineers highlighted the following contributing factors to low densities:[65]

1. *RCC segregation*
2. *Low compaction energy applied to the RCC*
3. *Increasing thickness of RCC layer*
4. *Wrong RCC placement methods*
5. *Wrong handling of Raw feed aggregates*
6. *Wrong delivery procedure for RCC transport.*

5.46 The memorandum set out measures to control segregation applicable to aggregate production and stockpiling, RCC production, and RCC placement. Measures included training new workers, avoided creating piles higher than 400 mm when placing RCC, avoiding interruptions in RCC placement, improving moisture content, and controlling compaction including with a focus on the speed of the roller and compacting within specified time limits.[66]

17 July 2004 Memorandum: 'Report on Night Shift'

5.47 On 17 July 2004, Mr Montalvo wrote a memorandum to Mr Embery with the subject '*Report on Night Shift*'.[67] It was copied to (among others) Mr Brampton, Mr Lopez and Mr Hamilton. Mr Montalvo noted that RCC had been placed successfully during the night shift starting on 15 July 2004. However, an inspection the following night revealed issues that caused low densities at the bottom of the '*1st layer*' of RCC:[68]

3. *RCC for the 1st layer was too wet.*
4. *One roller was going too fast.*
5. *The raw feed stockpile is not uniform.*
6. *Vibrators in the AugerMax were not used.*

5.48 The densities were brought within specified limits by re-compaction, although that was done outside the specified 40 minute time limit. However, that process was considered appropriate because the worst case conditions to which the 40 minute limit is tailored were not present.[69] It was noted that '*we should try to stick to the*

64 Exhibit 31, **SUN.009.002.0147**, .0167.
65 Exhibit 31, **SUN.009.002.0147**, .0167.
66 Exhibit 31, **SUN.009.002.0147**, .0167.
67 Exhibit 31, **SUN.009.002.0147**, .0169.
68 Exhibit 31, **SUN.009.002.0147**, .0169.
69 Exhibit 31, **SUN.009.002.0147**, .0169.

specified limit (up to 45 minutes) so that the strength development of the RCC is assured'.[70]

5.49 Mr Montalvo observed that '*[g]enerally speaking, the placement was done properly*'. The second layer of RCC had '*the optimum amount of moisture, the placement was more controlled, the speed of the rollers was adequate, and as a result there were good densities with less number of passes (10)'.*[71]

5.50 Mr Montalvo summarised '*Good things*' he had observed:[72]

- *The whole crew is keen, focused, and determined to do a good job.*
- *Same adjectives apply for the Engineer.*
- *The placement with the elephant trunk was very good; just a few details need to be corrected. ·But the skill the operator has developed is very good.*
- *The crew foreman knows his trade, but still listens to advice and tries to apply it.*

5.51 There were several corrective actions listed including:[73]

> *We have to take notice of the variation in the grading we get from the raw feed. Attached is a graph of the grading made to the aggregate sample taken in the morning. As can be seen, almost half of it is out of spec.*

5.52 And:[74]

> *A method of lift removal in case we get a layer which doesn't comply with specifications should be determined. When the decision to remove a layer is taken, this should be done immediately in order for it to be easier. The longer you wait the hard it gets. It is not good to just hope everything will be fine until we get out of the 'hole at Ch635', we need to have a layer-removal method.*

28 August 2004 Memorandum: 'Dental concrete D7'

5.53 A memorandum of 28 August 2004 titled '*Dental concrete D7'*[75] was written by both RCC Engineers. It raised issues with dental concrete that had been poured on the night shift of 27 August 2004 that was '*too moist with slump too high'*. Too much water to cement increased the risk of cracking due to shrinkage. The water dosage

70 Exhibit 31, **SUN.009.002.0147**, .0169.
71 Exhibit 31, **SUN.009.002.0147**, .0169.
72 Exhibit 31, **SUN.009.002.0147**, .0169.
73 Exhibit 31, **SUN.009.002.0147**, .0169.
74 Exhibit 31, **SUN.009.002.0147**, .0170.
75 Exhibit 31, **SUN.009.002.0147**, .0173.

was corrected, but it was determined that this dental concrete mix would not be used again.[76]

12 September 2004 Memorandum: 'RCC Low densities – NCR No 82'

5.54 Mr Montalvo and Mr Lopez wrote a memorandum of 12 September 2004 titled '*RCC Low densities - NCR No 82'*.[77] The recipients were Mr Herweynen and Mr Embery, with Mr Hamilton copied. The memorandum was drafted on 10 August 2004 but its issue was delayed by '*other important and urgent matters.'*[78] The subject matter was the low densities in RCC placed in Block P on 5 August 2004.[79]

5.55 During the night shift on that date, aggregate with an increased amount of fine sand was used. Low density readings were obtained in half of the first layer placed. Test pits revealed segregation and that RCC was not '*tightly locked at the bottom, and consequently the bond between layers was not as good as in previous occasions, but it still look acceptable*'. A test pit at the lowest density reading identified a '*big pocket of segregation*', which decreased when the test pit was expanded. Causes of that segregation included the aggregate not being well mixed on the stockpile, the aggregate feed decreasing when the new aggregate was used, and increased sand in the aggregate.[80]

5.56 The density readings at the bottom of the RCC layer in question included readings of 84.5%, 88.8% and 76.9%. As discussed above, the Specification is clear that no individual reading of less than the minimum may be allowed in any part of any lift. The RCC Engineers undertook a review of the stress demands taking into account the effect of the low density on predicted compressive and tensile strengths. They concluded that safety factors were 's*till very high for the different design demands in that specific area of RCC placement*'.[81] It was said that the '*non-conforming material be left in place as it is more than adequate for this region of the dam*.'[82] This evaluation is the same as that which Mr Lopez had done in relation to segregated RCC on 16 July 2004. Compliance with the Specification should have seen the material removed. It was left in place even though the material was not '*ideal at all*'.[83]

5.57 The evaluations carried out by the RCC Engineers regarding segregated RCC placed on 16 July 2004 and 5 August 2004 focus on compressive and tensile strength demands. There is no mention in the relevant memoranda that *shear* strength demands had been analysed. This is a concern given that porosity at the

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76 Exhibit 31, **SUN.009.002.0147**, .0173.
77 Exhibit 31, **SUN.009.002.0147**, .0175.
78 Exhibit 31, **SUN.009.002.0147**, .0175.
79 The memorandum is concerned with the last week of placement in Block P between 31 July and 6 August 2004. Therefore, the reference to the last night shift placement on 'Thursday 5 July 2004' is an erroneous reference to Thursday 5 ***August*** 2004.
80 Exhibit 31, **SUN.009.002.0147**, .0175.
81 Exhibit 31, **SUN.009.002.0147**, .0176.
82 Exhibit 31, **SUN.009.002.0147**, .0176.
83 Exhibit 31, **SUN.009.002.0147**, .0176.

bottom of an RCC layer has negative implications for the shearing resistance of the lift.

13 September 2004 Memorandum: 'RCC works on Primary Spillway'

5.58 On 13 September 2004, Mr Lopez and Mr Montalvo wrote a memorandum with the title '*RCC works on Primary Spillway*'.[84] It was addressed to Mr Embery and copied to others. It dealt with issues that included compaction.[85] The RCC Engineers had detected that there were '*times where the small roller was compacting areas of a layer that had RCC more than 1 h old*'.[86] To vibrate outside specified time limits, the RCC Engineers said, could '*only cause harm, affecting the stability of the dam by creating fractures in the RCC that are not going to be noticed*'.[87]

5.59 The memorandum also provided a guideline for the application of bedding mix. The requirements stated were consistent with Dr Schrader's 'relaxed' requirement for bedding mix on cold joints:[88]

Guideline for Bedding Mix Widths - *Primary Spillway Section*

Ch 200 to Ch 515 -Layers 1 to 11

RCC Lift Condition	Width of Bedding	Location (Distance from monolith)[1]	Location (Distance from U/S)
Less than 36 h old and less than 500°C/h*	600 mm	Start the strip within 1 m from the vertical surface	Against the face, fillet + appropriate width
More than 36 h but less than 500°C/h*	10% width of the dam at that elevation		
More than 36 h and more than 500°C/h*	15% width of the dam at that elevation		

29 September 2004 Memorandum: 'Damage on Carpi Membrane'

5.60 The memorandum dated 29 September 2004 was written by Mr Lopez and Mr Montalvo, and was addressed to Mr Embery, and copied to others.[89] With the subject '*Damage on Carpi Membrane',* the memorandum concerned damage to the Carpi membrane caused by the vibrating plate. A solution of welding a vertical metal sheet to the side of the vibrating plate was suggested.[90]

84 Exhibit 31, **SUN.009.002.0147**, .0177.
85 Exhibit 31, **SUN.009.002.0147**, .0177.
86 Exhibit 31, **SUN.009.002.0147**, .0177.
87 Exhibit 31, **SUN.009.002.0147**, .0177.
88 Exhibit 31, **SUN.009.002.0147**, .0178.
89 Exhibit 31, **SUN.009.002.0147**, .0179.
90 Exhibit 31, **SUN.009.002.0147**, .0179.

24 October 2004 Memorandum: 'Surface Cleaning & Bedding Issues + RCC D/S Steps + Curing + Roller'

5.61 Mr Lopez and Mr Montalvo wrote a memorandum on 24 October 2004, titled '*Surface Cleaning & Bedding Issues + RCC D/S Steps + Curing + Roller'.*[91] It was addressed to Mr Embery and Mr Lindschau, and copied to others. The first part of the memorandum served as a reminder of good practices ahead of RCC being placed in an area where the underlying RCC surface had been exposed for a long time. The memorandum also raised three quality control issues relevant to the shear strength of the Dam.

5.62 First, as to lift joint preparation:[92]

> *There was an issue raised yesterday when the placement foreman said he had cleaned the placement area in the morning so he thought it was OK to place in the afternoon. One of the problems caused by not enough curing on layers of lean RCC mixes is the fact that debris just builds up, particularly if you clean it in the morning and then let it dry while having vehicles and people circulating on top of it (even just letting it dry). The most important thing deriving from this is the fact that the area needs to be OK just before placement; it's a matter of planning and placing. This is expected to become less of a problem as we begin having full layers and continuous placement, where we can have people cleaning 25 m ahead of the placement.*
>
> *It is important not to soak the area within 25 m of RCC placement; this should be in [Saturated Surface Dry] condition in order to have increased cohesion between layers. The "coin" test to ascertain if the RCC surface has the right moisture should be used. This should not be confused as to mean that it is preferred to have a dry area before placement.*

5.63 Secondly, as to the application of bedding mix:[93]

> *It is completely wrong to place bedding mix on top of loose material, there will be no bonding whatsoever and we'll have a weak spot and seepage areas where this occurs. The fact that this has been happening (not frequently, though) in different crews, is also worrying. Usually the crews are very good and keen to do the right thing, but this is an act of apathy and carelessness that shouldn't even need anyone to ask for correction, it should be corrected automatically. If whilst placing the bedding mix, a spot of loose material that somehow was not seen by the sweeper or vacuum is reached, they should skip the area, keep placing elsewhere, and go back when the loose material is removed. This remotion of unexpected loose material is in a "as-best-as possible" basis, unless it is near the upstream face where it should be perfectly cleaned.*

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91 Exhibit 31, **SUN.009.002.0147**, .0181.
92 Exhibit 31, **SUN.009.002.0147**, .0181.
93 Exhibit 31, **SUN.009.002.0147**, .0181.

5.64 Thirdly as to segregation:[94]

> *[W]e need to improve our segregation control on the D/S steps. Pockets of segregation can be found along the steps. This can be a problem, enhanced by the weakness caused by the lack of curing; we'll have a weak interface between RCC and conventional concrete.*

5.65 This comment relates to the steps in the primary spillway which are covered with conventional concrete.

5.66 The memorandum goes on to discuss cleaning that needed to be done for that day's placement, the curing of the downstream RCC steps, the insufficient number of rollers available, and the seepage zone causing issues for that night's placement.[95]

5.67 Attached to the memorandum were photographs that illustrated the problems that the RCC Engineers had raised. Two of those photographs are reproduced below. Their poor quality reflects their nature as produced to the Commission.[96]

Picture 1 -23 Oct 04- Bedding placement on RCC edge contaminated with debris

Picture 5 – 23 Oct 04- Cars on RCC surface causing contaminating and increasing cleaning activities. It is not allowed to circulate cars on RCC surface

Figure 5.5 – Photographs from QA memorandum dated 24 October 2004

28 October 2004 Memorandum: 'RCC issues & Cleaning activities Vs RCC production'

5.68 Mr Lopez and Mr Montalvo wrote a memorandum dated 28 October 2004, titled '*RCC issues & Cleaning activities Vs RCC production*' to Mr Embery and Mr Lindschau.[97] The memorandum attached a document that discussed measures to increase RCC production and improve quality and that sought to revisit '*some important issues that should not be overlooked*'.[98]

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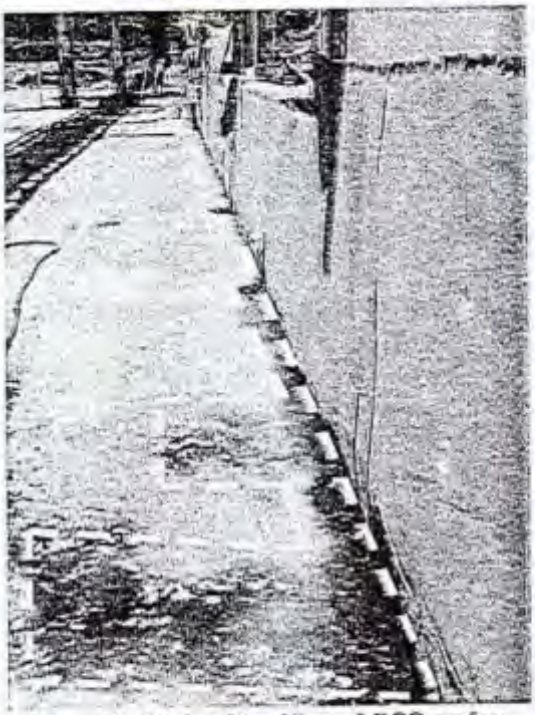
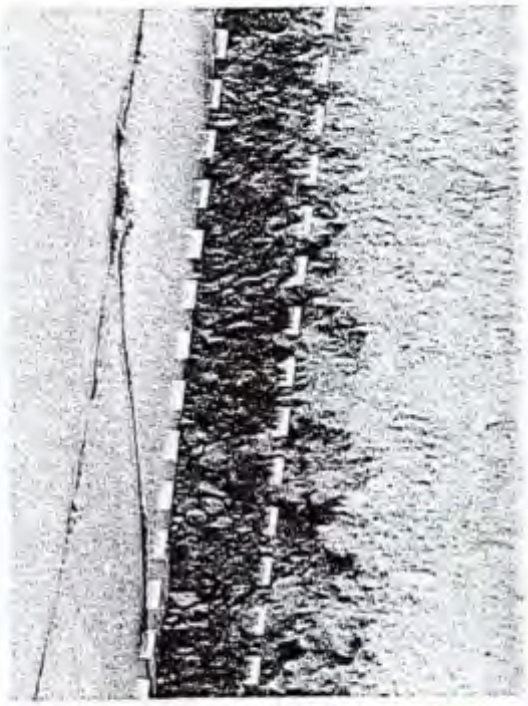
94 Exhibit 31, **SUN.009.002.0147**, .0181.
95 Exhibit 31, **SUN.009.002.0147**, .0181.
96 Exhibit 31, **SUN.009.002.0147**, .0182.
97 Exhibit 31, **SUN.009.002.0147**, .0185 (It was also copied to others).
98 Exhibit 31, **SUN.009.002.0147**, .0185.

5.69 Nine issues were raised, one of which related to cleaning of lift joints. The memorandum observed that RCC production and quality were being affected by '*[d]elays on RCC surface cleaning activities, not doing them properly, all this worsened by lack of coordination between both work shifts (Day & night shift) in this important activity*'. To address the problem, '*[a] good cleaning management needs to be established and applied*'.[99] Contaminated and poorly cleaned surfaces were depicted in several photographs included in the memorandum, some examples of which follow:[100]



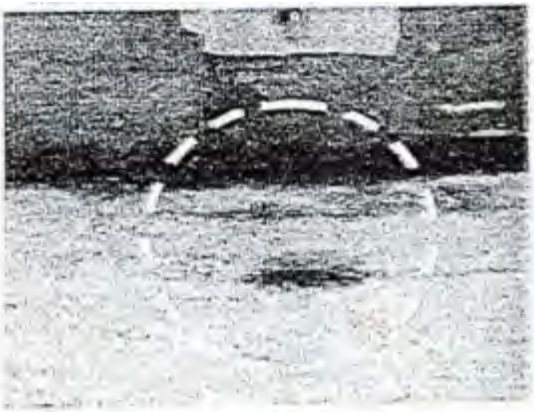
Picture 7. 25 Oct 04 – View of RCC surface next to U/S panel contaminated with sand, & mud creating delays to start RCC placement.



Picture 8. 25 Oct 04 - Detailed view of debris & mud next to U/S face panels.

99 Exhibit 31, **SUN.009.002.0147**, .0186.

100 Exhibit 31, **SUN.009.002.0147**, .0190, .0191, .0193, .0194.



Picture 17. 26 Oct 04 - Not acceptable U/S surface to place RCC. This is the most critical part of the dam regarding cleaning activities

Picture 31. 27 Oct 04 10:30 AM. Accumulation of debris after RCC surface cleaning by air jetting method. Lack of curing increases them.

Picture 33. 27 Oct 04 10 AM. Car inside the dam, contaminating the RCC surface. This is not acceptable.

Picture 42. 27 Oct 04 – Damage of RCC surface by wheels of the placing conveyor. It is advisable to clean & remove this contamination before RCC placing.

Figure 5.6 – Photographs of cleaning issues from QA memorandum dated 28 October 2004

5.70 The memorandum discussed the incorrect application of bedding mix:[101]

> *The purpose of the bedding mix is to improve the bonding between layers, and to seal the layers to water seepage. By not covering all the area that requires bedding, or by not placing the required thickness, seepage risks are left within the dam:*

[101] Exhibit 31, **SUN.009.002.0147**, .0187.

- *MSA aggregate is 10 mm. Then, the minimum thickness of bedding mix should be 10 mm.*
- *The thickness of the bedding mix should be uniform and regular over all the RCC surface that requires it (See pictures).*
- *Attention is needed also in the bedding mix fillet along the U/S face: some areas have been left without bedding and some with insufficient bedding. This requires immediate correction.*

5.71 These points were illustrated in photographs, including the following:[102]

Figure 5.7 – Photographs of issues with application of bedding mix from QA memorandum dated 28 October 2004

102 Exhibit 31, **SUN.009.002.0147**, .0190, .0191, .0193.

5.72 Of curing lift surfaces, the memorandum said:[103]

> *Curing is a concern. It has not been done as it should be. The RCC surface is dry during long periods of time. This is caused by the fact that now we have bigger areas to cure than the previous two (trial section and block P), and as a cause the number of workers needed to have good curing using this method is insufficient.*
>
> *Another curing method is urgently needed, a more efficient one due to the fact that because of future weather conditions, curing will become more and more difficult (higher temperatures). A round table should be devoted to this issue and actions taken immediately.*
>
> *The lack of curing affects both strength and cleaning activities. By keeping the RCC moist immediately after compaction, the surface particles acquire better bonding, and … the youngest the RCC the more it needs water*

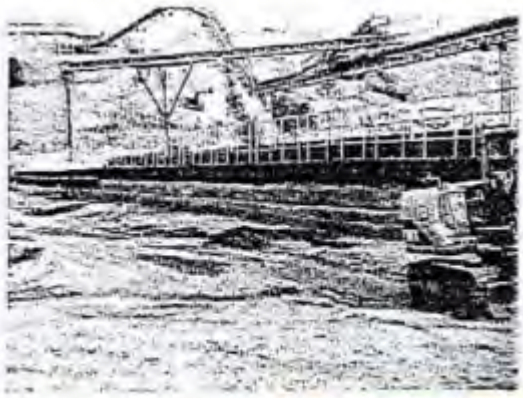
5.73 The following photographs from the memorandum illustrate some of these issues:[104]

Picture 30. 27 Oct 04 – Dry RCC surface due to the lack of curing.

Picture 38. – 27 Oct 04 – Bad curing at left side of the dam (Ch 300 to 260) approx)

103 Exhibit 31, **SUN.009.002.0147**, .0187.
104 Exhibit 31, **SUN.009.002.0147**, .0193, .0194.

Picture 39. 27 Oct 04 – Bad curing area on critical area next to U/S face



Picture 40. 27 Oct 04 – Bad curing at D/S step spillway section

Figure 5.8 – Photographs of curing issues from QA memorandum dated 28 October 2004

5.74 The memorandum also discussed the impact of placing RCC in higher temperatures than had been assumed in Dr Schrader's thermal analysis. The temperatures were up to seven degrees Celsius higher than anticipated. The strain capacity of the RCC was said to be exceeded but there was still *'an appropriate safety factor for the dam*'.[105] Four inexpensive measures were suggested, although there was no mention of pausing placement activities (as had been contemplated at the tender stage).[106]

5.75 While many of the photographs included in the document depicted problems, the following picture was complimentary of work on site:[107]

Picture 37. 27 Oct 04 NS. Good aggregate feed, good surface quality. Compare the surface of RCC placed 27-Oct to the one above placed 26-Oct

Figure 5.9 – Photograph of good lift surface quality from QA memorandum dated 28 October 2004

105 Exhibit 31, **SUN.009.002.0147**, .0188.
106 Exhibit 31, **SUN.009.002.0147**, .0189.
107 Exhibit 31, **SUN.009.002.0147**, .0194.

9 November 2004 Memorandum: 'Bedding mix B5 – Retarder & air entrainment admixture dosage'

5.76 On 9 November 2004, Mr Lopez and Mr Montalvo wrote a memorandum to Mr Hunt and Brian Langridge, titled '*Bedding mix B5 – Retarder & air entrainment admixture dosage*'.[108] The memorandum noted that the slump of bedding mix was below specified requirements[109]. It recommended modifications to the retarder admixture for bedding mix B5 to allow for variations in temperatures whether placing RCC during a day or night shift.[110]

11 November 2004 Memorandum: 'RCC Production - Discussion of 10 November 2004'

5.77 Mr Embery wrote a memorandum, dated 11 November 2004, to Mr Lopez, Mr Montalvo and Mr Lindschau. It was titled '*RCC Production - Discussion of 10 November 2004'*.[111]

5.78 Mr Embery recalled writing that memorandum because '*We were dealing with a divergence of issues and how to manage them, yes. So it's really to say, 'This is how it works'.*[112]

5.79 In the memorandum, Mr Embery discussed steps that would be taken for further RCC production seemingly in light of comments from the RCC Engineers. In several ways, the memorandum suggested that instructions from the RCC Engineers had not been consistent to that point. This is evident in comments within sections headed:

a. 'Motivation':[113]

> *Of particular concern to the motivation presently is … [c]leaning several times due to changing needs and late requests for cleaning;*

b. 'Equipment and Tools for Cleaning':[114]

> *The use of air/water is noted although we had been previously advised that this was the least preferred option for dry surfaces. Additional gear is being procured / made.*

c. 'Confused signals / instructions':[115]

> *Any concerns Jose/Roberto and others have with regard to the RCC operation should be directed to the Shift Supervisors. Instruction should only be issued by QA/QC or other persons where some person/s is in danger of being injured.*

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108 Exhibit 31, **SUN.009.002.0147**, .0195.
109 Exhibit 31, **SUN.009.002.0147**, .0195.
110 Exhibit 31, **SUN.009.002.0147**, .0195.
111 Exhibit 31, **SUN.009.002.0147**, .0197.
112 Exhibit 114, **TRA.510.018.0001**, .0034 ln 15-17.
113 Exhibit 31, **SUN.009.002.0147**, .0197.
114 Exhibit 31, **SUN.009.002.0147**, .0197.
115 Exhibit 31, **SUN.009.002.0147**, .0197.

If the Supervisor does not respond to the concerns you raise, then they should be passed to Col, Jason or myself.

d. 'Surface Cleaning':[116]

The standards of cleaning of the surface must be consistent. Having to re-clean areas again which previously been acceptable needs to be addressed.

23 November 2004 Memorandum: 'RCC placement activities'

5.80 The RCC Engineers wrote a memorandum dated 23 November 2004, with the subject '*RCC placement activities*'.[117] It was addressed to Mr Embery and Mr Lindschau. The memorandum commenced by saying that comments on RCC placement were made 'w*ith the purpose of increasing RCC productivity and correct certain procedures that still have room for improvement, increasing in this way the RCC quality*'.[118] Many of the points raised in the memorandum were of a general educational nature and not specific to problems that had been identified on site. However, there was discussion of shortcomings, including the 'irregular' placement of bedding mix that was often too thick. The exposition time of the bedding mix needed to be controlled and kept within the specified 45 minute limit. The authors explained that '*placing long strips of bedding mix that are reached late by the placement must be avoided, especially in the critical zones where the spreading and placement of RCC takes more time. We need to improve the application of this point in the field*'.[119]

5.81 In relation to lift joint preparation, the memorandum stated:[120]

During RCC compaction it has been noted that rough irregular areas lacking fines, worsened by not ideal of curing and an excess of RCC moisture in the mix, generate debris increasing the cleaning requirements. This fact becomes more critical in the areas where the finishing of the surface is done late with the small roller and the vibrating plates.

5.82 In relation to placing RCC one layer at a time, the memorandum stated:[121]

It is better to complete a layer before beginning to place the next one, avoiding placing the RCC in these places simultaneously with the next layer. In this way, exceeding the compaction time in the following layer will be avoided.

This undesirable situation has required removal of RCC due to increased layer thickness, late compaction, affecting the densities of the RCC.

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116 Exhibit 31, **SUN.009.002.0147**, .0197.
117 Exhibit 31, **SUN.009.002.0147**, .0199.
118 Exhibit 31, **SUN.009.002.0147**, .0199.
119 Exhibit 31, **SUN.009.002.0147**, .0199.
120 Exhibit 31, **SUN.009.002.0147**, .0200.
121 Exhibit 31, **SUN.009.002.0147**, .0200.

5.83 The memorandum concluded with the observation that '*The process of delivering a good finished product is well on track, however, we need to pay attention to the little details that can improve our performance*'.[122]

14 January 2005 Memorandum: 'RCC Works'

5.84 Mr Lopez and Mr Montalvo wrote to Mr Embery and Mr Lindschau on 14 January 2005, in a memorandum with the subject '*RCC Works*'.[123] The memorandum served as a reminder of quality issues for the crew returning after the Christmas holiday period. Certain procedures were said to require correcting and there was still room for improvement with RCC quality.[124]

5.85 Observations about poor practices included in relation to curing:[125]

> *In some occasions, lack of good curing of the RCC has increased the cleaning requirements of the surface, delaying the start of RCC placement.*
>
> *This situation has been more critical on the slot, after doing an excellent cleanup of the surface using pressured air during the night shift, leaving the surface ready for RCC placement on the next night shift, the lack of adequate curing during day shift delayed placement of RCC.*
>
> *Likewise, there are relatively long periods in both shifts and in between where the surface remains dry. It is necessary and advisable to improve the control of this activity by the supervisors.*

5.86 Although cleaning was said to be generally satisfactory, issues were identified:[126]

> *Some specific topics regarding cleaning can still be improved:*
>
> *a. RCC edges: It is recommended to clean RCC edges (backswing & others) using always water and the suck-truck. It is not enough to only use the suck truck. Along the RCC edges mud is accumulated with the circulation of vehicles, due to the effect of rain and curing water. This makes them a risk and the logical path for any seepage in the body of the dam, we need to seal them.*
>
> *b. In general, the cleaning of the RCC surface is being done satisfactorily. Cleaning requirements are increased by the movement of equipment over the fresh RCC and by irregularities in the surface, caused especially when the RCC is too wet. It is important to have these comments in mind to avoid increasing the cleaning activities.*

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122 Exhibit 31, **SUN.009.002.0147**, .0201.
123 Exhibit 32, **SUN.009.002.0203**, .0203.
124 Exhibit 32, **SUN.009.002.0203**, .0203.
125 Exhibit 32, **SUN.009.002.0203**, .0203.
126 Exhibit 32, **SUN.009.002.0203**, .0205.

c. *The vacuum truck in the areas to be cleaned must be used without spraying water onto the surface. This procedure creates a slurry and/or mud over the surface that affects the bond between layers. It is important to clarify this to the truck operators in both shifts. This was noted by Dr. Schrader during his last visit, but continues to be done.*

d. *Use of hand brushes for final cleaning activities on RCC surface is not advisable, and they should not be used on top of a wet surface. After using the hand brushes, the RCC surface looks good, but a micro-slurry that is only possible to see with a magnifying glass is created, affecting the bond between RCC surface and bedding mix and the bond between RCC and RCC of the following layer. After using the hand brushes on a dry surface, this needs to be blown (near the U/S face).*

5.87 The memorandum included positive comments about work on site, including in relating to compaction:[127]

> *The loss of moisture control has been improving since the start of the roster, and most of the time the crew is attentive and trying to prevent it.*

19 January 2005 Memorandum: 'RCC surface affected by the rain – Comments'

5.88 Mr Lopez and Mr Montalvo wrote a memorandum titled '*RCC surface affected by the rain – Comments*', which appears to be erroneously dated 19 January 2004.[128] The content reveals that the relevant date was 19 January 2005. The memorandum was addressed to Mr Embery and Mr Lindschau. The memorandum stated:[129]

> *[T]hat the crews are doing a good job, and it's in the spirit of getting better that these comments are forwarded. Their purpose is not to attack or disqualify, but rather to compliment the good work being done and suggest some improvements. Both RCC crews have experienced this situation by now, and both reacted in similar ways.*

5.89 A prior rainfall event was relied upon to illustrate quality problems of rain on uncompacted RCC. The recommended procedure was to compact the RCC and roll down the edge, without the rollers moving back onto previously compacted RCC. That had occurred during the rainfall event in question. When compacting the roll down edge, the rollers '*went back onto the RCC surface for as much as 12 metres causing unnecessary damage to the RCC surface and increasing the cleaning requirements*'.[130]

5.90 The RCC Engineers explained the procedure to be implemented where there was a risk of rain and when it actually rained.

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127 Exhibit 32, **SUN.009.002.0203**, .0204.
128 Exhibit 32, **SUN.009.002.0203**, .0207.
129 Exhibit 32, **SUN.009.002.0203**, .0207.
130 Exhibit 32, **SUN.009.002.0203**, .0207.

3 February 2005 Memorandum: 'Comments prepared for RCC meeting 2nd February 2005'

5.91 The memorandum dated 3 February 2005, with the subject '*Comments prepared for RCC meeting 2nd February 2,005'* was written by Mr Lopez and Mr Montalvo.[131] It was addressed to Mr Embery and Mr Lindschau and copied to Mr Hamilton.[132] It seemed to provide further explanation of comments that had been prepared (presumably by the RCC Engineers) for a meeting with Mr Embery the day before.

5.92 The memorandum discussed downtime at the start of night shifts, which appear to have been the subject of prior discussions. The delays were identified by the RCC Engineers to identify and address the causes.[133] Mr Lopez said that his role extended beyond quality issues to assist in managing schedule and cost:[134]

> *We hold a different view regarding QA/QC. QA/QC for me is related not only to RCC placement but also to control of schedule and cost of the project. This is what I (Jose) have done in previous projects and including this project, preparing and bringing up to date the dam construction program. Before coming to Australia, Ernest Schrader and Steve Johnson knew about the job I developed in other projects and its importance in order to prevent these kinds of issues from happening. A detailed record of delays was kept in Mujib Dam Project as can be seen in the copy of the final report I provided to this project. However, if you consider we shouldn't do this, we will not do it.*

5.93 This memorandum suggests a level of tension (and possibly division) between construction and QA personnel. Such tension is suggested again in the concluding remarks of the memorandum:[135]

> *Finally, we understand that our function here is to be proactive using our experience and work so the finished product is the best possible. Our comments are made with the best intentions, not aiming to offend or attack anyone in particular, just thinking in the best interest of the project.*

5.94 The memorandum refers to photographs that were presumably discussed on 2 February 2005. While those images are described in the memorandum, they were not included in the document produced to the Commission. The discussion about certain pictures reveals the following issues with quality of RCC placement.

5.95 RCC placed at the downstream area against the insertion form for the training wall had not been sufficiently compacted. Insufficient compaction of RCC at the downstream face of the primary spillway led to RCC breaking away when the form for the left training was removed. The RCC Engineers raised the concern to '*prevent possible problems that may occur due to lower quality in this critical area of the*

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131 Exhibit 32, **SUN.009.002.0203**, .0209.
132 Exhibit 32, **SUN.009.002.0203**, .0209.
133 Exhibit 32, **SUN.009.002.0203**, .0209.
134 Exhibit 32, **SUN.009.002.0203**, .0209.
135 Exhibit 32, **SUN.009.002.0203**, .0210.

dam'.[136] The wacker packer should be used when placing RCC in the downstream area and in places against rock where irregularities prevented the vibrating plate from doing a better job. Dr Schrader had suggested that approach and the memorandum said that '*[i]n order to convince everyone about this, we can try this procedure on one complete layer and assess its results*'. That everyone needed convincing suggests that the QA personnel had to make some effort to have construction personnel adopt their recommendations.

5.96 Tension between those personnel was noted:[137]

> *There's no conflict whatsoever in what has always been advised. The requirement has consistently been the same from the start and reinforced in the RCC presentation for Supervisors and crew offered in early November 2,004 (copies of this presentation were provided to each supervisor and people involved in RCC placement and it was included in the October-November Quality Report).*

7 March 2005 Memorandum: 'RCC Dosage & Moisture Control during production'

5.97 Mr Lopez and Mr Montalvo wrote a memorandum with the subject '*RCC Dosage & Moisture Control during production'*, dated 7 March 2005.[138] It was addressed to Mr Langridge and Rob Frazer. The issue addressed in the memorandum was excessively high moisture content in the RCC mix:[139]

> *It has been noted that sometimes RCC moisture has been excessive and variable during both shifts. As mentioned above, having excessive moisture affects the finish of the layer, because the material is picked up by the roller and it leaves flakes of un-bonded material on the top. There's an urgent need to correct this as it affects the Lift Joint Quality, which is essential to have better bond between layers.*
>
> *The method used to improve the Lift Joint Quality Index before placing the next layer, has been the application of bedding mix in the affected areas. This is an acceptable method, but it increases the cost of the project.*
>
> *Requesting the RCC with excessively high moisture (more than 5.5%) is not the answer for the loss of moisture during placement and spreading activities, in doing this, we are also affecting the strength at the long term and the variability of the RCC we are placing. The answer is to control the placement activities so the rollers are not left behind and the surface is kept moist before and after it is compacted (using a light mist).*

5.98 The memorandum went on to explain how to optimise moisture content.

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136 Exhibit 32, **SUN.009.002.0203**, .0209.
137 Exhibit 32, **SUN.009.002.0203**, .0209.
138 Exhibit 32, **SUN.009.002.0203**, .0211.
139 Exhibit 32, **SUN.009.002.0203**, .0211.

11 May 2005 Memorandum: 'Low RCC densities at Main Spillway on RL 66.185'

5.99 A memorandum written by Mr Lopez and Mr Montalvo dated 11 May 2005 was titled '*Low RCC densities at Main Spillway on RL 66.185'*.[140] Only Mr Lopez is named as the sender on the first page, but the names of both of the RCC Engineers appear at the end of the document. The memorandum was addressed to Mr Embery and Mr Lindschau, and it was copied to others including Dr Schrader.

5.100 The memorandum concerned low densities that had been '*obtained in a very important area given that it is the last layer of RCC in the Main Spillway where the slide and tension requirements are critical*'.[141] A summary of density values taken from the area showed that minimum readings were all at or below 92% and that three of four were below the minimum allowable 89% TAFD. However, from other content in the memorandum, it was clear that large compaction equipment had been used to roll the RCC. The applicable minimum (individual) reading was therefore 92%.

5.101 The causes of the low densities were summarised in the following way:[142]

- *Compaction Delay (from 1 to 1.5 hours)*
- *Use of new compaction equipment that did not comply with specifications (CC 422C HF)*
- *Segregation due to generation of high piles on hard RCC surface (see picture 1), lack of moisture, (area on the downstream where the chute delivers the RCC – see pictures 2 & 3)*
- *Lack of compaction in a localized area (near jackpost 12 – see picture 4)*
- *Low Densities - Related to previous issues*
- *Lack of moisture control on RCC surface at times*

5.102 Some of the listed causes are fundamental aspects of RCC placement. It is of concern that problems with, for instance, delayed compaction were occurring in May 2005, many months after RCC placement began, and near the top of the primary spillway.

5.103 In response to the low densities detected, a small area of RCC was to be removed because it had not been compacted properly. However, the following areas of low-density RCC were left in place (save for removing segregated areas on the upstream and downstream faces):[143]

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140 Exhibit 32, **SUN.009.002.0203**, .0215.
141 Exhibit 32, **SUN.009.002.0203**, .0215.
142 Exhibit 32, **SUN.009.002.0203**, .0215.
143 Exhibit 32, **SUN.009.002.0203**, .0215.

a. RCC on the downstream side of the area from chainage 200 m to chainage 226 m

b. RCC placed from chainage 226 m to 255 m, which had low densities from upstream to downstream. It was expected that segregation and loose material identified in test pits would be removed when cleaning the surface to prepare it for pouring the conventional concrete Ogee Crest.

5.104 The decision to leave low density RCC in place was justified in two ways: firstly, by an analysis to consider the effect of low densities on compressive and direct tensile strength.[144] This was not the sort of evaluation that the Specification permitted in relation to non-complying density readings. The RCC Engineers did not assess the impact of the low density on shear strength demands.

5.105 The second basis for leaving non-complying RCC in place was the visual inspection of test pits. The results of that inspection indicated that there was:[145]

a. excellent bond between the RCC lifts was observed because the underlying layer had been covered with bedding mix

b. segregation in a localised area only

c. general segregation and loose material on the downstream face.

5.106 While concluding that removal of all affected areas was not required, the RCC Engineers recommended that the designer decide if the factor of safety should be increased by placing anchors in the affected area.[146]

5.107 The following recommendations for future RCC placement were made. Some of these are fundamental to the RCC placement methodology, which is concerning because of the late stage that the project had reached:[147]

1. *Fill the test pits with dental concrete (dental concrete can also be used in the trench dug in order to rescue the drainage pipe on the U/S at Chainage 360 approx.*

2. *Segregation control should also be done correction the placement method as has been suggested in previous occasions and was also suggested in this case (placing on top of fresh RCC - stopping production if required - using the posi-track to spread the RCC and leave a bed for placement with the chute)*

3. *No RCC should be compacted after 1 hour - if need be roll down an edge rather than trying to keep it live to the extreme*

144 Exhibit 32, **SUN.009.002.0203**, .0216.
145 Exhibit 32, **SUN.009.002.0203**, .0217.
146 Exhibit 32, **SUN.009.002.0203**, .0217.
147 Exhibit 32, **SUN.009.002.0203**, .0217.

4. *Rollers complying with specification should be used*

5. *Moisture of RCC should be increased to account for loss of moisture until compaction and to control segregation*

6. *CC 422C HF in combination with vibrating plate is acceptable in areas where the big roller can not compact (only around jackposts).*

5.108 In the picture report attached to the memorandum, the following photographs depicted segregation and a localised area not compacted:[148]

Figure 5.10 – Photographs included in QA memorandum dated 11 May 2005

13 May 2005 Memorandum: 'Low RCC densities at Main Spillway on RL 66.185 – Additional comments'

5.109 On 13 May 2005, Mr Lopez and Mr Montalvo wrote a memorandum with the title '*Low RCC densities at Main Spillway on RL 66.185 – Additional comments'.*[149] It was addressed to Mr Embery and Mr Lindschau and copied to others. This document

148 Exhibit 32, **SUN.009.002.0203**, .0217, .0218.
149 Exhibit 32, **SUN.009.002.0203**, .0221.

followed up the memorandum of 11 May 2005, discussed above.[150] After the test pits had been cleaned and inspected, the following observations were made:[151]

- *Segregation is noticeable at the bottom of the layer in some downstream test pits close to jackpost number 12 (Ch 286) and between jackposts 13 and 14 (Ch 226 to Ch 255)*
- *Segregation affects an area that ranges from 2.5 m (see picture 1) to 1.2 m (see picture 2) into the layer from the downstream side of the final layer. The upstream side of the layer looks good.*
- *Rain water from last night's showers is seeping through the layer, as reported by the sweeper operator, indicating the presence of segregation.*

5.110 Based on the visual inspections, the RCC Engineers had decided that all of the areas affected by segregation needed to be removed by an excavator. Segregation also appeared to have occurred on the downstream side of the RCC layer below. Photographs in the memorandum depict the issues with segregation:[152]

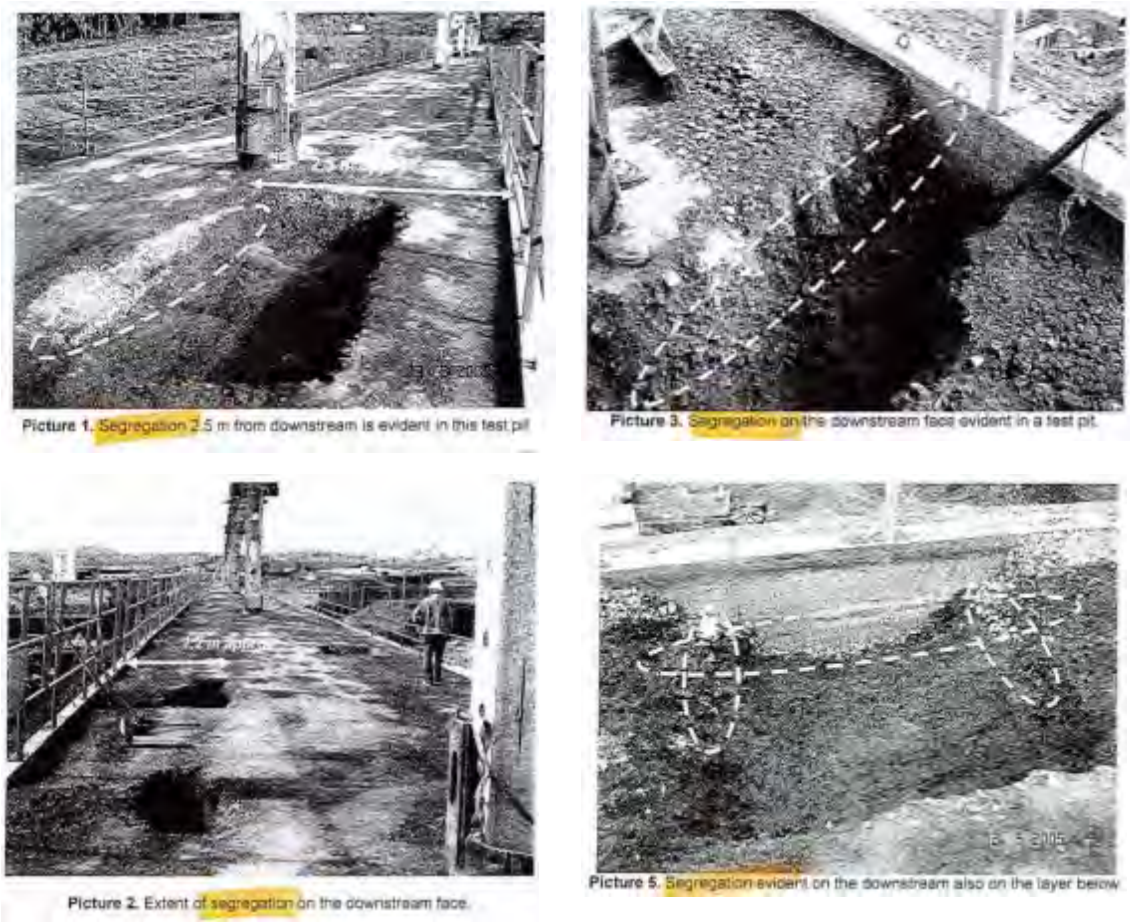

Figure 5.11 – Photographs of segregation from QA memorandum dated 13 May 2005

150 See memorandum dated 11 May 2005.
151 Exhibit 32, **SUN.009.002.0203**, .0221.
152 Exhibit 32, **SUN.009.002.0203**, .0222, .0223.

13 June 2005 Memorandum: 'Main Spillway Apron- Suggested Placement Sequence''

5.111 The memorandum dated 13 June 2005, with the subject '*Main Spillway Apron- Suggested Placement Sequence'* was written by Mr Lopez and Mr Montalvo.[153] It was addressed to Mr Embery and Mr Lindschau, and copied to others. The purpose of the memorandum was to suggest improvements but, in so doing, it pointed to quality issues affecting RCC placement. For instance, the memorandum said that '*most of the RCC gets compacted before 40 minutes (within specification) but some minor areas are compacted until up to 1.8 hours'*.[154]

5.112 The following photographs from the memorandum depict issues in compacting RCC too close to edges, placing thin layers of RCC on top of previously compacted RCC surfaces (which was 'absolutely prohibited' by the Specification[155]), and loose material being left along RCC edges and affecting the bond between old and new RCC.

Figure 5.12 – Photographs of issues placing RCC in the primary spillway apron from QA memorandum dated 13 June 2005[156]

153 Exhibit 32, **SUN.009.002.0203**, .0225.
154 Exhibit 32, **SUN.009.002.0203**, .0225.
155 Exhibit 21, **DNR.003.8385**, .8464.
156 Exhibit 32, **SUN.009.002.0203**, .0227 to .0229.

5.113 NCR 196 was raised in relation to insufficient compaction of the top layer of RCC on the primary spillway apron between chainages 470 m and 480 m. The non-conformance occurred on 28 May 2005 when the vibrating plate had malfunctioned.[157] The problem was rectified by removing three areas of RCC (which were detailed in the NCR by Mr Montalvo) and making sure that the RCC edges were cleaned before RCC was placed against them.[158]

13 July 2005 Memorandum: 'RCC sampling frequency'

5.114 Mr Lopez wrote a memorandum to Mr Frazer, on 13 July 2005, with the subject '*RCC sampling frequency'*.[159] The memorandum concerned the frequency of RCC sampling and did not raise any presently relevant quality issues.

4 August 2005 Memorandum: 'Re-Start of RCC Works – Placement at Left Abutment'

5.115 Mr Montalvo authored a memorandum to Mr Lindschau on 4 August 2005, with the title '*Re-Start of RCC Works – Placement at Left Abutment*'.[160] Mr Montalvo stated that '*most things [were] going well*' and that the memorandum was intended to assist in continuing improvements.[161]

5.116 Of the issues raised for continuous improvement, the following are indicative of quality issues that Mr Montalvo had observed:[162]

> 1. *Traffic onto the RCC requires control - as we all are aware, more traffic equals a bigger cleaning effort (See picture 2, unathorised vehicle on top of RCC.*
>
> …
>
> 3. *A few ramps were placed in order to allow for access and to erect panels (Ch 125 approx). As it is known, the edges need to be chipped back when they are too thin, and segregation needs to be removed (See picture 6).*
>
> 4. *When an edge is rolled down or overlapped to another edge, it needs to be cut back to sound RCC. Picture 7 shows an area with cracks on the RCC. When this happens, it is advisable to remove this section.*

157 **SUN.117.004.0187**, .0187.
158 **SUN.117.004.0187**, .0189.
159 Exhibit 32, **SUN.009.002.0203**, .0231.
160 Exhibit 32, **SUN.009.002.0203**, .0233.
161 Exhibit 32, **SUN.009.002.0203**, .0233.
162 Exhibit 32, **SUN.009.002.0203**, .0233.

5.117 Pictures 2, 6 and 7 mentioned above are included below:


Picture 2 (4-August-05DS). DO NOT allow unauthorised vehicles on the RCC


Picture 6 (4-August-05DS). Ramps for access and to erect panels – Chip thin edges and remove segregation on the edges

Picture 7 (4-August-05DS). Cracks on RCC edges – the edge needs to be cut back to sound RCC.

Figure 5.13 – Photographs from QA memorandum dated 4 August 2005.
*(Exhibit 32, **SUN.009.002.0203**, .0234 to .0235)*

3 September 2005: Low Densities on 2 September 2,005 dayshift

5.118 Mr Montalvo issued a memorandum dated 3 September 2005 to Mr Lindschau, Mr Brampton and Mr Rickert (among others).[163] The memorandum was titled '*Low Densities on 2 September 2,005 dayshift'* and copied to recipients including Mr Embery. Placement of RCC on the non-overflow section was said to be '*mostly going very well*'. Two layers had been placed on 2 September 2005, the first of which returned a low density reading on the upstream side. That is an area where there was some segregation from trucks dumping the RCC. The densities were still within specified limits because:[164]

a. lower thresholds applied to density readings because small compaction equipment were used in the area; and

b. in crediting bedding mix with aiding with the density problem, '*bedding mix was placed within more than 1m of width, overlapping the "problem" area*'.

163 Exhibit 32, **SUN.009.002.0203**, .0237.
164 Exhibit 32, **SUN.009.002.0203**, .0237.

5.119 It was also noted that:[165]

> *The segregation could have been minimised by an attachment at the pugmill, from where segregation originates by the way we are loading the moxies. As we have only 9 more layers to go, it is certainly something that is not cost-efficient, so we are forced to correct it at the placement after dumping the RCC.*
>
> *This can be done either by the posi-track, spending a little more time in the spreading by having the moxies dump the RCC further back on top of fresh RCC, or by shovelling the segregated material up before the posi-track pushes it.*

Explanations of the construction memoranda

Timing considerations

5.120 Various witnesses offered context to these memoranda. Mr Herweynen explained that the memoranda were intended to educate the inexperienced workforce:[166]

> *I would suggest that at the time, and to put it in perspective, as I said at the beginning, there was no-one in Australia that we could find, in terms of Golders, to find these quality control engineers. In the same way from a construction point of view, going out to market to try to find people that have placed RCC before - would you find them if there hasn't been an RCC [dam] constructed? Probably not.*
>
> *The net result is that you had a lot of people that had never done this before, so it was one of the requirements of both Jose and Robert, and also Ernie when he came to site in that early phase, to be very critical of things that we don't want and things that we do want and to actually put that in writing, eg, these memos, and take them through training processes of saying, 'That's no good. This is how you should do it'.*

5.121 He said that the memoranda were representative, but only of their time and in light of their purpose being to draw attention to deficiencies before the main RCC laying commenced:[167]

> *I think the key part of their function was to help train these people that were going to be on the lift surface both in terms of cleaning and placement, techniques and processes, to ensure that we get improved quality for the main project.*

5.122 Mr Herweynen emphasised that many of the construction memoranda had been written during the early stages of RCC placement. He referred to the following graph contained in RCC QC Report No 9:[168]

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165 Exhibit 32, **SUN.009.002.0203**, .0237.
166 Exhibit 247, **TRA.510.007.0001**, .0054 ln 35 to .0055 ln 4.
167 Exhibit 247, **TRA.510.007.0001**, .0055 ln 22-26.
168 Exhibit 38, **SUN.110.003.0001**, .0030.

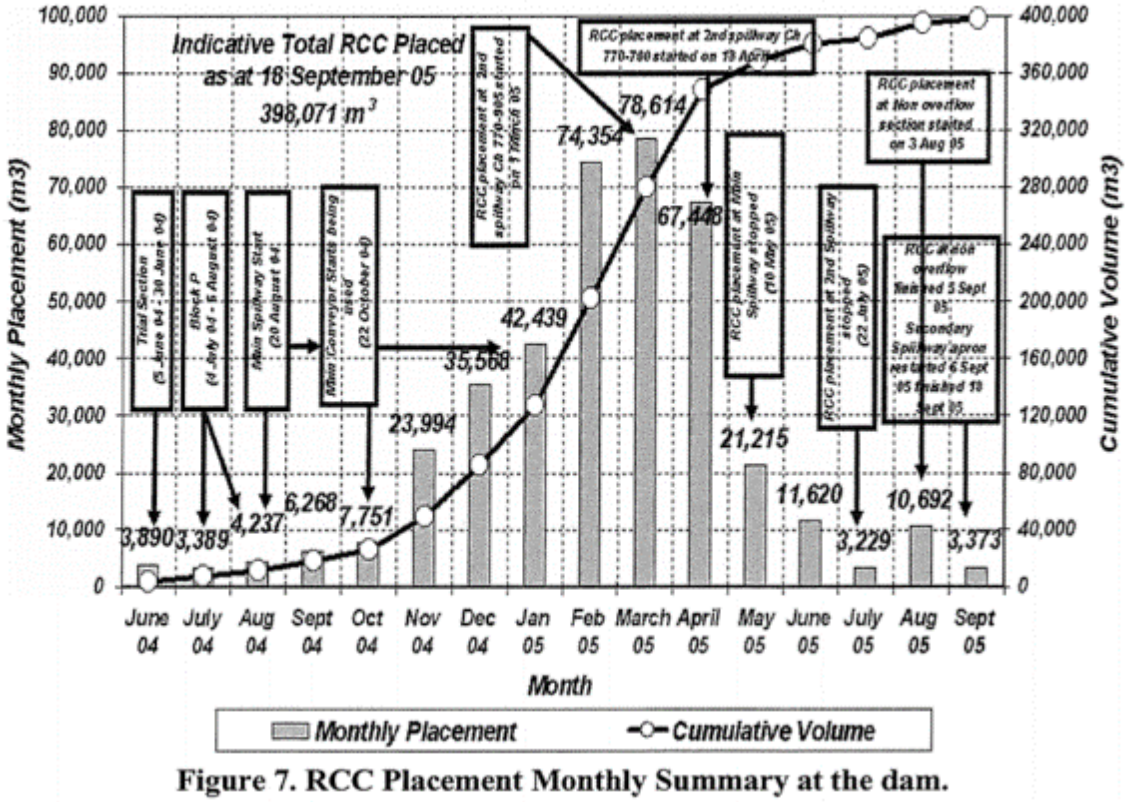

Figure 7. RCC Placement Monthly Summary at the dam.

Figure 5.14 – Summary of RCC placement by month from the final RCC QC Report

5.123 Mr Herweynen went on to say:[169]

> *Most of these memos were written from 1 June to 30 October. If you look there, the placement of RCC in that time was going at a very, very slow rate. The reason for that is the main primary spillway conveyor, the high production hadn't started. And it was done by mobile conveyors, and some of that wasn't ready for RCC, so there were even delays within the trial embankment to the start of the RCC being placed.*
>
> *So you can see that there are approximately - I would suggest maybe 30,000 cubic metres of RCC had been placed out of a total of 400,000 cubic metres of RCC, all in the upper-right abutment, so on the secondary spillway, none of it in the higher part of the dam in the primary spillway.*
>
> *This period of time was seen as a time that we could use to get our workforce skilled at placing RCC. I think that's an important element.*

5.124 Mr Herweynen's point is that the preponderance of quality control problems were encountered during the early stages of RCC placement. He accepted that issues might have arisen during later stages of construction, '*because of course things happen on a project*'. Where that was the case, his evidence was that '*you would*

169 Exhibit 247, **TRA.510.007.0001,** .0056 ln 8-24

hope that they pick it up and you would hope that there is adequate documentation to show the action that was taken associated with that'.[170] The adequacy of documentation showing remedial action is discussed in more detail below.

5.125 As to Mr Herweynen's point about the timing of construction issues and the placement of RCC:

a. three of the memoranda[171] identified quality problems associated with RCC placement in the trial embankment between 5 and 30 June 2004

b. three[172] identified quality control problems associated with RCC placement in Block P in the right abutment between 4 July and 5 August 2004

c. one[173] identifies problems during RCC placement in the main spillway in the period from 20 August 2004 until 22 October 2004 before the conveyor was operational

d. eleven raised quality control issues associated with RCC construction methodologies (including bedding mix) in the main spillway by the conveyor from 22 October 2004 to 10 May 2005. (One memorandum during that period related to the Carpi membrane and so is not included in this total number.) Of those 11:

   i. two were written in October 2004[174]

   ii. three were written in November 2004[175]

   iii. two were written in January 2005[176]

   iv. one was written in February 2005[177]

   v. one was written in March 2005[178]

   vi. two were written in May 2005, although those two memoranda concerned the same segregation and density issues[179]

e. none of the memoranda concerned RCC placement in the secondary spillway from 1 March 2005 to 22 July 2005

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170 Exhibit 247, **TRA.510.007.0001**, .0057 ln 30-34.
171 Exhibit 31, **SUN.009.002.0147**, .0147, .0153, .0155.
172 Exhibit 31, **SUN.009.002.0147**, .0163, .0169, .0175.
173 Exhibit 31, **SUN.009.002.0147**, .0177.
174 Exhibit 31, **SUN.009.002.0147**, .0181, .0185.
175 Exhibit 31, **SUN.009.002.0147**, .0195, .0197, .0199.
176 Exhibit 32, **SUN.009.002.0203**, .0203, .0207.
177 Exhibit 32, **SUN.009.002.0203**, .0209.
178 Exhibit 32, **SUN.009.002.0203**, .0211.
179 Exhibit 32, **SUN.009.002.0203**, .0215, .0221.

f. one memorandum raised quality problems with RCC placed in the primary spillway in May and June 2005[180]

g. two memoranda raised problems with RCC placed in the non-overflow section on the left abutment from 3 August to 5 September 2005.[181]

5.126 The timing of the memoranda and the RCC quality issues they identify do not accord with Mr Herweynen's view. Only four of the 29 memoranda relate to the trial embankment. The purpose of building it was to train the workforce in RCC placement methods. Quality problems identified in that area are of less significance for present purposes, given that the trial embankment is subjected to '*very low levels of stress and [did] not require the same quality as the main spillway*'.[182]

5.127 QC issues continued to be raised regularly. Mr Herweynen said that memoranda written before 30 October 2004 were composed before high RCC production started. However, this overlooks that before this time, construction had occurred in important areas of the Dam, including the foundation layers of RCC in the primary spillway.

5.128 Rates of placement in the base of the primary spillway were low for a number of reasons, including because RCC had to be placed in '*patches*' as there was not a large enough area in which to continuously place it.[183] Problems with the foundation layers of RCC in the primary spillway had potential to impact on the sliding stability of the Dam. These problems included delayed compaction of RCC, which might have created fractures in the RCC that could go unnoticed.[184] The base of the primary spillway is a critical zone for the Dam's sliding stability. The quality of RCC lift joints there was important.[185]

5.129 In short, over a third of those memoranda concerned quality problems in (or associated with) RCC placed in the primary spillway. Problems were identified regularly during construction of that part of the Dam.

Not reflective of what was occurring on site

5.130 Mr Embery said that the memoranda '*produced a view of the work which doesn't necessarily match with what was happening on site*'.[186] Taken in evidence to some of the memoranda, he said that they were selective:[187]

> *Well, at home I've got reports that are completely contrary to all the ones that you've just been through. You've been through all the ones that appear to have some negative impact on how the project was being run. I have others that are*

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180 Exhibit 32, **SUN.009.002.0203**, .0225.
181 Exhibit 32, **SUN.009.002.0203**, .0233, .0237.
182 Exhibit 21, **DNR.003.8385**, .8479.
183 Exhibit 31, **SUN.009.002.0147**, .0177.
184 Exhibit 31, **SUN.009.002.0147**, .0177.
185 Exhibit 24, **GHD.002.0001**, .0156.
186 **TRA.500.009.0001**, .0104 ln 30-31.
187 **TRA.500.009.0001**, .0107 ln 30-35.

equally glowing. You can't just extract - it has to be done on a global basis. That's my point.

5.131 Mr Embery said that he could produce documents that might cast a different light on the issues in the memoranda.[188] The following day, he wrote to the Commission and said that '*after a search of my personal records and archives, I have not been able to locate the material. Please submit my apologies to the Inquiry for not being able to produce the documents which were disposed of approx. 3 years ago*'.[189]

5.132 Mr Embery's employer during the project was Macmahon.[190] It was also invited to produce any documents that might cast a different light on the construction memoranda. Macmahon produced no further documents.[191]

Problems revealed by the construction memoranda

5.133 A summary of the issues identified in the construction memoranda follows:

Memorandum date	Quality issue identified	Date issue identified	RCC placement location[192]	Corresponding NCR
15 June 2004[193]	Segregation Low densities Bedding mix application	6 to 17 June 2004	Trial embankment	NCRs 50,[194] 51[195] and 52[196]
19 June 2004[197]	Segregation Aggregate moisture Aggregate handling	17 June 2004[198]	Trial embankment	NCR 52[199]
19 June 2004[200]	Segregation Low densities Low moisture	16 and 18 June 2004	Trial embankment	NCRs 52[201] and 53[202]
3 July 2004[203]	Aggregate gradation	6 June to 2 July 2004	N/A - aggregate stockpile	
6 July 2004[204]	Dental concrete	6 July 2004	Block P	

188 **TRA.500.009.0001**, .0108 ln 22-32.
189 Exhibit 322, **MCM.017.0001.**
190 **TRA.500.009.0001**, .0072 ln 13-14.
191 Exhibit 321, **MCM.018.0001**.
192 Based on Exhibit 38, **SUN.110.003.0001**, .0030.
193 Exhibit 31, **SUN.009.002.0147**, .0147.
194 **DNR.006.3533**, .3556.
195 **DNR.006.3533**, .3554.
196 **DNR.006.3533**, .3552.
197 Exhibit 31, **SUN.009.002.0147**, .0153.
198 See correction to the date made in exhibit 31, **SUN.009.002.0147**, .0155.
199 **DNR.006.3533**, .3552.
200 Exhibit 31, **SUN.009.002.0147**, .0155.
201 **DNR.006.3533**, .3552.
202 **DNR.006.3533**, .3550.
203 Exhibit 31, **SUN.009.002.0147**, .0157.
204 Exhibit 31, **SUN.009.002.0147**, .0159.

Memorandum date	Quality issue identified	Date issue identified	RCC placement location[192]	Corresponding NCR
			foundation	
14 July 2004[205]	Aggregate gradation	7 to 9 July 2004	N/A - aggregate stockpile	NCR 55[206]
16 July 2004[207]	Segregation Low densities	15 July 2004	Block P foundation	NCR 57[208]
17 July 2004[209]	N/A – general methodology	N/A	N/A	
17 July 2004[210]	Low densities Segregation Aggregate gradation	16 July 2004	Block P	
28 August 2004[211]	Dental concrete slump	27 July 2004	Dam foundation[212]	
12 September 2004[213]	Low densities Segregation Aggregate gradation	5 August 2004	Block P	NCR 82[214]
13 September 2004[215]	RCC layer thickness Dental concrete slump Compaction	20 August 2004 to 13 September 2004	Primary Spillway before conveyor	
29 September 2004[216]	Damage to Carpi membrane	29 September 2004	Primary Spillway	NCR 106[217]
24 October 2004[218]	Segregation Cleaning Curing Application of bedding mix	23 October 2004	Primary Spillway	
28 October 2004[219]	Cleaning Curing Application of bedding mix	22 to 28 October 2004	Primary Spillway	
9 November	Slump of bedding mix	9 November	Primary Spillway	

205 Exhibit 31, **SUN.009.002.0147**, .0161.
206 **SUN.102.002.0040**.
207 Exhibit 31, **SUN.009.002.0147**, .0163.
208 **DNR.006.3434**, .3517.
209 Exhibit 31, **SUN.009.002.0147**, .0167.
210 Exhibit 31, **SUN.009.002.0147**, .0169.
211 Exhibit 31, **SUN.009.002.0147**, .0173.
212 Exhibit 38, **SUN.110.003.0001**, .0007.
213 Exhibit 31, **SUN.009.002.0147**, .0175.
214 **SUN.102.001.0402**.
215 Exhibit 31, **SUN.009.002.0147**, .0177.
216 Exhibit 31, **SUN.009.002.0147**, .0179.
217 **DNR.006.4207**, .4277.
218 Exhibit 31, **SUN.009.002.0147**, .0181.
219 Exhibit 31, **SUN.009.002.0147**, .0185.

Memorandum date	Quality issue identified	Date issue identified	RCC placement location[192]	Corresponding NCR
2004[220]		2004		
11 November 2004[221]	Cleaning	10 November 2004	Primary Spillway	
23 November 2004[222]	Segregation Moisture control Placement of bedding mix Curing	Unspecified	Primary Spillway	
14 January 2005[223]	Cleaning Curing	Unspecified	Primary Spillway	NCR 150[224]
19 January 2005[225]	Cleaning	Unspecified	Primary Spillway	
3 February 2005[226]	Insufficient compaction Compaction delays	Unspecified	Primary Spillway	
7 March 2005[227]	Moisture content affecting LJQI	Unspecified	Primary Spillway /Secondary Spillway	
11 May 2005[228]	Low densities Segregation Insufficient compaction Moisture control.	10 May 2005	Primary spillway crest	NCRs 192[229] and 193[230]
13 May 2005[231]	Segregation	10 May 2005	Primary Spillway	NCRs 192[232] and 193[233]
13 June 2005[234]	Compaction delay and procedure Incorrect placement of RCC	28 May 2005 29 May 2005 02 June 2005 08 June 2005 09 June 2005 12 June 2005	Primary Spillway Apron	NCR 196[235] raised for issue on 28 May 2005
13 July 2005[236]	N/A – explanation of RCC sampling frequency	N/A	Secondary Spillway	

220 Exhibit 31, **SUN.009.002.0147**, .0195.
221 Exhibit 31, **SUN.009.002.0147**, .0197.
222 Exhibit 31, **SUN.009.002.0147**, .0199.
223 Exhibit 32, **SUN.009.002.0203**, .0203.
224 **DNR.006.3916**, .3917.
225 Exhibit 32, **SUN.009.002.0203**, .0207.
226 Exhibit 32, **SUN.009.002.0203**, .0209.
227 Exhibit 32, **SUN.009.002.0203**, .0211.
228 Exhibit 32, **SUN.009.002.0203**, .0215.
229 **SUN.117.004.0233**.
230 **SUN.117.004.0220**.
231 Exhibit 32, **SUN.009.002.0203**, .0221.
232 **SUN.117.004.0233**.
233 **SUN.117.004.0220**.
234 Exhibit 32, **SUN.009.002.0203**, .0225.
235 **SUN.117.004.0187**.
236 Exhibit 32, **SUN.009.002.0203**, .0231.

Memorandum date	Quality issue identified	Date issue identified	RCC placement location[192]	Corresponding NCR
4 August 2005[237]	Cleaning Segregation	4 August 2005	Left Abutment	
3 September 2005[238]	Low density Segregation	2 September 2005	Left Abutment	

5.134 The memoranda reveal three types of problems relevant to sliding stability that arose more than once: 1) the problems with segregation, compaction and low densities; 2) cleaning and curing issues with lift joint surfaces; 3) the application of bedding mix.

Segregation, compaction and density problems

5.135 Problems with compaction, segregation and low densities arose during construction. LCRCC is prone to segregating,[239] so it was appropriate that the RCC Engineers scrutinised placement works to monitor this unfavourable condition.

5.136 The trial embankment may be put aside. Construction of that non-critical section of the Dam was designed to train the workforce in placement techniques. It was to be expected that there would be issues with segregation in that location.[240]

5.137 Other than in the trial embankment, the construction memoranda identify this issue a number of times. On the occasions listed immediately below, either low densities were remedied or the observations of the RCC Engineers were general and did not relate to a particular section of RCC that required rectification:

a. In Block P on 16 July 2004, low densities were corrected by re-compaction to bring the RCC within specified limits.[241]

b. At the base of the primary spillway and at times between 20 August and 13 September 2004, the small roller was observed vibrating RCC outside the specified time limits.[242] No particular area of RCC was identified as having been, for instance, damaged and requiring remedy. This issue appears to have been raised only generally.

c. On 23 November 2004 when RCC was being placed in the primary spillway, the RCC Engineers documented that removal of some RCC had been required because more than one layer of RCC had been placed at a time. That had

237 Exhibit 32, **SUN.009.002.0203**, .0233.
238 Exhibit 32, **SUN.009.002.0203**, .0237.
239 **TRA.500.002.0001**, .0004 ln 33-38.
240 Exhibit 48, **TRA.510.025.0001**, .0027 ln 3 to .0028 ln 2.
241 Exhibit 31, **SUN.009.002.0147**, .0169.
242 Exhibit 31, **SUN.009.002.0147**, .0177.

resulted in increased layer thickness and late compaction, which affected densities.[243]

d. In May 2005, when the last two layers of RCC were placed at the top of the primary spillway, there were problems with low density, segregation and insufficient compaction. Initially, the RCC Engineers advised that some of the segregated RCC could be left in place, subject to the designers considering measures to increase the factor of safety.[244] Ultimately, the construction memorandum of 13 May 2005 required that all areas of RCC affected by segregation be removed.[245] NCRs 192 and 193 are consistent with that requirement. They speak to the extent to which what the RCC Engineers had said was heeded by construction personnel. Two of the corrective actions recorded on NCR 193 were:[246]

- *Involve RCC Engineers more*
- *Heed RCC Engineers' suggestion.*

e. Between 28 May 2005 and 12 June 2005 (when RCC was placed in the primary spillway apron), the time to compaction in '*some minor areas*' extended beyond specified limits.[247] Areas of RCC that were not properly compacted on 28 May 2005 were removed.[248]

f. On 2 September 2005, low densities were detected in the left abutment. The readings were within the specified density limits for areas compacted by small equipment.[249]

5.138 The following detections of segregated RCC appear not to have been remedied:

a. In Block P on 15 July 2004, low density affected a '*small area*' of an RCC lift. Some of the density readings at the base of the layer were below the minimum permitted measurement. Despite the requirements of the Specification, that material was not removed because Mr Lopez reviewed stress demands in that location and determined that it was tolerable for the material to be left *in situ*.[250] The Specification did not permit that evaluation in respect of low density RCC. Mr Lopez's evaluation was relied on to close out NCR 57 that had been raised in respect of the low density readings.[251]

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243 Exhibit 31, **SUN.009.002.0147**, .0200.
244 Exhibit 32, **SUN.009.002.0203**, .0217.
245 Exhibit 32, **SUN.009.002.0203**, .0221.
246 **SUN.117.004.0220**, .0220.
247 Exhibit 32, **SUN.009.002.0203**, .0225.
248 **SUN.117.004.0187**.
249 Exhibit 32, **SUN.009.002.0203**, .0237.
250 Exhibit 31, **SUN.009.002.0147**, .0165.
251 **DNR.006.3434**, .3517.

b. In Block P on 5 August 2004, density readings below the minimum allowable reading were returned at the bottom of an RCC lift.[252] The RCC Engineers evaluated the stress demands at that location and allowed the material to remain in place.[253]

c. On 24 October 2004, the RCC Engineers said that pockets of segregation could be found along the downstream steps of the primary spillway.[254] It is not clear whether those pockets of segregation were remedied.

d. Compaction issues were raised again on 3 February 2005 in relation to the primary spillway. RCC at the downstream face of the primary spillway, described as a '*critical area of the dam*', was not compacted properly.[255] It is not clear, on the face of the memorandum, if lower quality RCC was removed.

e. On 4 August 2005 when RCC was being placed in the left abutment, segregation was detected in the edges of RCC that had been placed to form access ramps.[256] It is not clear whether the thin segregated RCC edges were chipped back as required.

5.139 The construction memoranda record only two areas where the RCC Engineers documented their decision to leave RCC with low density in place: in Block P at EL 51.45 m;[257] and in Block P at EL 60.295 m.[258] There are three other locations where low density RCC was detected and it is not clear whether that RCC was removed: along the downstream steps of the primary spillway in October 2004; at the downstream face of the primary spillway in around February 2005; and in localised areas in the left abutment on 4 August 2005.

5.140 These density problems were recorded as being localised. They relate to two areas in Block P, sections of the downstream steps of the primary spillway, and one location in the left abutment. If low density RCC in the Dam was limited to those locations, that probably would not be enough to call the Dam's stability into question.

5.141 The construction memoranda are indicative of ongoing and recurring problems. But, importantly, those problems were either remedied or are not such as to prejudice the Dam's structural integrity and stability. The memoranda show that density problems were raised regularly by the RCC Engineers, but they mostly seem to have been remedied. They do not reveal that the same issues with density and segregation arose with such frequency as would justify inferring that the RCC was systemically affected by these problems.

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252 Exhibit 31, **SUN.009.002.0147**, .0175.
253 Exhibit 31, **SUN.009.002.0147**, .0176.
254 Exhibit 31, **SUN.009.002.0147**, .0181.
255 Exhibit 32, **SUN.009.002.0203**, .0209.
256 Exhibit 32, **SUN.009.002.0203**, .0233.
257 Exhibit 31, **SUN.009.002.0147**, .0163.
258 Exhibit 31, **SUN.009.002.0147**, .0176.

5.142 That is not to say that no concerns about segregation arise based on these memoranda. One concern is the justifications given by the RCC Engineers for leaving segregated material in Block P. It was the same process used to support the initial decision to leave segregated material in the top of the primary spillway.

5.143 The Specification contemplated that if an LJQI score was less than -3, an RCC Engineer might 'occasionally' allow the lift joint to remain based on a review of design stresses at the particular location.[259] However, the equivalent evaluation was not provided for in section 11.9 of the Specification, which dealt with compaction requirements.[260] Therefore, a review of the stress demands at a location was not a permitted means of dealing with segregated RCC. If that was a process that the RCC Engineers adopted elsewhere during construction of the Dam and simply did not document, that process had the potential to affect the sliding stability of the Dam.

5.144 The likelihood of that having occurred is informed by what the RCC QC Reports say about field density measurements. Each of the bi-monthly reports indicated how many readings were below certain specified density requirements. As discussed above, the minimum average of the density readings from the top, middle and bottom of an RCC lift was 92% TAFD for areas compacted by small equipment and 96% where large rollers had been used. The minimum individual reading was 89% TAFD for small equipment areas and 92% otherwise. By way of example, the following table was included in the last RCC QC Report:

Depth (mm)	Number of data	Average density (t/m^3)	% of compaction (Avg.)	% data below compaction requirement:	
				92% TAFD	96% TAFD
Avg (1 June to 31 July 05)	73	2.464	98.0	< 0.1%	10.6%
Avg (1 Aug to 30 Sept 05)	83	2.479	97.7	< 0.1%	10.4%
50	83	2.483	97.9%	< 0.1%	9.3%
100	9	2.499	98.5%	< 0.1%	0.9%
150	74	2.484	97.9%	< 0.1%	16.4%
200	10	2.487	98.0%	3.0%	20.3%
250	67	2.472	97.4%	4.5%	28.5%
300	6	2.407	94.8%	5.7%	96.5%

Table 8. RCC field density summary – Main Spillway - 1 June to 30 Sept 05

*Figure 5.15 – Field density summary table from the RCC QC Report for August and September 2005. (Exhibit 38, **SUN.110.003.0001**, .0050)*

259 Exhibit 21, **DNR.003.8385**, .8467.
260 Exhibit 21, **DNR.003.8385**, .8465 to .8466.

5.145 As the example above shows, the RCC QC Reports recorded:

a. the percentage of averaged readings for the reporting period that were below the minimum average requirements of 92% TAFD and 96% TAFD respectively (see the second row of reported data in the table above)

b. the percentages of individual readings at each depth that were below the two minimum average requirements (i.e. 92% and 96%) (see rows 3 to 8 of reported data in the table above).

5.146 It is not clear why the Alliance compared individual readings with the minimum average. Individual readings were permitted to be lower: 89% for areas in which small compaction equipment was used and 92% for large compaction equipment areas. In explaining why the reports showed that readings were below 92% TAFD, Mr Herweynen said that those low readings were due to small or manual compaction equipment having been used.[261] The difficulty is that the reports do not explain which readings are from areas where small equipment was used and which are not. The reader of them has no way of knowing whether the readings are from such areas or not. Mr Herweynen was asked to identify where in the RCC QC Reports that was made clear. He was unable to do so, but pointed to general statements in the reports that the field densities complied with technical specifications.[262]

5.147 The reporting does indicate the lift depths at which the low densities were being detected.

5.148 The reported percentage of data lower than the 92% TAFD in the RCC QC Report for:

a. July and August 2004 – 21% at 150 mm deep and 37% at 300 mm deep (where these high percentages can be understood because the RCC Engineers reviewed stress demands regarding low density RCC in Block P during this reporting period)[263]

b. September 2004 – 2% at a depth of 250 mm[264]

c. October and November 2004 – 20% at 200 mm deep, 5% at 250 mm, and 16% at 300 mm[265]

d. December 2004 and January 2005 – 4.5% at 350 mm deep, and 8.2% at 300 mm deep[266]

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261 **TRA.500.014.0001**, .0021 ln 23-25.
262 **TRA.500.014.0001**, .0022 ln 1-15.
263 **SUN.128.002.0001**, .0029
264 Exhibit 101, **SUN.110.002.0158**, .0192.
265 **SUN.110.002.0279**, .0326.
266 **SUN.110.001.0001**, .0055.

e. February and March 2005 – 6.1% at 250 mm deep, 7.4% at 300 mm, and 6.2% at 350 mm deep[267]

f. April and May 2005 – were exactly the same as in the preceding report, suggesting that the relevant entries may not have been updated because other information in the table had changed from the preceding report[268]

g. June and July 2005 – 4.5% at 250 mm deep, 5.7% at 300 mm, and 5.0% at 350 mm deep[269]

h. August and September 2005 – 3% at 250 mm deep, 4.5% at 300 mm, and 5.7% at 350 mm (although the reporting period was from 1 June to 30 September 2005).[270]

5.149 During July and August 2004, and again in October and November 2004, the percentage of low density readings was higher than at other times. Typically, the percentage of readings lower than 92% TAFD was approximately 5% at the lower depths of RCC. Given these low percentages for the majority of the reporting periods, Mr Herweynen's explanation that the low readings related to areas where small compaction equipment was used may well be correct. If that be so, the RCC Engineers would not have frequently confronted RCC outside the density requirements of the Specification. Therefore, it is unlikely that the RCC Engineers would have commonly used a review of local stress demands to justify leaving segregated RCC in place. That type of demand may only have occurred on the three occasions evidenced in the construction memoranda.

Cleaning and curing issues

5.150 Issues with lift surface preparation, particularly cleaning and curing, arose periodically in the construction memoranda:

a. On 23 October 2004, when RCC was being placed in the base levels of the primary spillway, the RCC Engineers became aware that a member of the construction team thought that it was acceptable to clean a lift surface in the morning and not place RCC there until the afternoon.[271] The memorandum does not reveal if that surface was prepared again before RCC was placed.

b. On 11 November 2004, Mr Embery wrote that equipment was being procured so that cleaning could be done with air and water.[272] This suggests that the advice from the RCC Engineers about cleaning methods was implemented by construction personnel.

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267 **SUN.110.001.0174**, .0234.
268 **SUN.110.001.0365**, .0415.
269 **ALC.001.001.0658**, .0709.
270 Exhibit 38, **SUN.110.003.0001**, .0050.
271 Exhibit 31, **SUN.009.002.0147**, .0181
272 Exhibit 31, **SUN.009.002.0147**, .0197.

c. On 19 January 2005, the RCC Engineers pointed out that rollers had damaged the surface of compacted RCC by traversing back over it while rolling down the leading edge of RCC during a rainfall event.[273] That is noted as having increased cleaning requirements suggesting that the extra cleaning was done.

d. On 4 August 2005, while placing RCC in the left abutment, Mr Montalvo identified unauthorised traffic on a lift joint, which meant that more cleaning had been required.[274]

5.151 Of the cleaning and curing issues identified in the following memoranda, remedial action either did not take place or could not be confirmed from the documents before the Commission:

a. A memorandum on 28 October 2004 identified issues arising since RCC delivery with the all-conveyor system had started[275] on 22 October 2004.[276] The problems related to RCC being placed in the lower levels of the primary spillway, including with contaminated lift surfaces, and delays to and inadequate cleaning.[277] Those problems were said to be affecting RCC quality, which suggests that the problems were not rectified. The memorandum stated that curing was not being done properly because there were not enough workers.[278]

b. On 23 November 2004, when RCC placement was progressing in the primary spillway, the RCC Engineers observed that poorly graded aggregate, inadequate curing, and excessive moisture in the RCC mix were generating debris which increased cleaning requirements.[279]

c. On 14 January 2005, it was observed that RCC was being left dry for long periods.[280] Lack of curing increased cleaning requirements. Although cleaning was generally satisfactory, the vacuum truck was spraying water onto the lift surface to clean. That created a slurry across the surface that affected the bonding between layers.[281]

5.152 Problems with cleaning generally appear to have been rectified. Three memoranda identify deficiencies in curing when RCC was being placed in the primary spillway. The memoranda indicate that curing problems were raised at isolated times. For instance, there is no trend in the documents and no language indicating that longstanding curing problems needed to be raised repeatedly (for instance, because construction personnel were ignoring QA personnel). This is consistent with the

273 Exhibit 32, **SUN.009.002.0203**, .0207.
274 Exhibit 32, **SUN.009.002.0203**, .0233.
275 Exhibit 31, **SUN.009.002.0147**, .0187.
276 Exhibit 38, **SUN.110.003.0001**, .0030.
277 Exhibit 31, **SUN.009.002.0147**, .0186.
278 Exhibit 31, **SUN.009.002.0147**, .0187.
279 Exhibit 31, **SUN.009.002.0147**, .0200.
280 Exhibit 32, **SUN.009.002.0203**, .0203.
281 Exhibit 32, **SUN.009.002.0203**, .0205.

evidence of the RCC Engineers that the problems they identified were rectified at the time that attention was called to them.[282]

5.153 This strongly suggests that curing deficiencies were not systemic and, also, are not so widespread as to call into question the stability of the Dam.

Bedding mix issues

5.154 Bedding mix performed a fundamental role in providing bonding and therefore cohesion and tensile strength to lift joints. Any problems in its application are directly relevant to the shearing resistance of lift joints.

5.155 On 9 November 2004, the RCC Engineers recommended changes to the bedding mix to bring it within specified requirements.[283] The RCC QC Report for October and November 2004 indicates that the modifications were made to the bedding mix.[284]

5.156 The following problems with bedding mix do not appear to have been remedied:

a. In relation to RCC in the base of the primary spillway, the RCC Engineers said on 24 October 2004 that bedding mix had been placed on top of loose material. While not a frequent occurrence, the practice was said to be 'worrying'.[285]

b. On 28 October 2004, the RCC Engineers raised problems with bedding mix not being applied to its minimum thickness, nor in a uniform thickness. Some areas of the fillet along the upstream precast panels had been left with inadequate bedding mix or without any bedding at all.[286]

c. On 23 November 2004, bedding mix was applied too thickly. The exposure time of the bedding mix also needed to be kept within time limits and placing bedding mix too far ahead of RCC placement needed to be avoided, especially in critical zones. The memorandum said that improvements were needed in the field, but there is no indication whether those improvements were ever made.[287]

5.157 Problems with the application of bedding mix are concerning because of its importance in providing cohesion. However, only three construction memoranda raise such problems over the six months during which the primary spillway was built. This suggests that there were not systemic issues with the quality of bedding mix application.

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282 **TRA.500.006.0001**, .0029 ln 25- 29; Exhibit 324, **LOJ.003.0001**, .0014.
283 Exhibit 31, **SUN.009.002.0147**, .0195.
284 **SUN.110.002.0279**, .0355.
285 Exhibit 31, **SUN.009.002.0147**, .0181
286 Exhibit 31, **SUN.009.002.0147**, .0187.
287 Exhibit 31, **SUN.009.002.0147**, .0199.

Difficulty knowing the extent to which problems were remedied

5.158 It is difficult to know whether and the extent to which some of the problems identified in the construction memoranda were remedied. There are clear cases where the matters raised in the memoranda were dealt with, and properly evidenced.[288] For instance, low density RCC was removed.

5.159 At times, NCRs were raised regarding issues identified in the construction memoranda. However, the reason why some issues raised an NCR and others did not is not clear. It may have something to do with the division of responsibilities between the RCC Engineers and other QA personnel. Mr Montalvo recalled that Mr Frazer was responsible for managing NCRs.[289] Mr Lopez said that generating NCRs was not part of his responsibilities.[290] That may explain why NCRs were not more frequently raised when issues were detected, identified and documented by the RCC Engineers in the construction memoranda.

5.160 There were other examples of problems being identified but not remediated on the express basis that they affected, for example, localised areas only. Where a quality control issue was not the subject of an NCR, it can be difficult to see whether and how the issue was dealt and whether, importantly, it was remediated. Where there appears to be no documentary evidence that an issue was rectified, the only other evidence is the accounts given to the Commission. Those included evidence of Mr Montalvo, who said that problems were corrected:[291]

> *[M]y memory of that is that we would bring the issue up, and then things would improve, and then we would hit the same snag again, and then we would reiterate our problem with the activities, and then they would get corrected again.*

5.161 Mr Montalvo gave the following explanation of how the RCC Engineers had dealt with quality control problems during construction:[292]

> *Q. When you saw problems or issues, how did you bring them to people's attention or get them fixed?*
>
> *A. The first thing to do - if we're talking about during placement, you talk to the foreman, to the placement foreman, for him to correct the issue that you're pointing out.*
>
> *Q. Did you do that regularly?*

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288 See, for example, Exhibit 31, **SUN.009.002.0147**, .0169, which explains that RCC was re-compacted to bring densities within specified limits; **SUN.117.004.0187** which was the NCR raised in respect of Exhibit 32, **SUN.009.002.0203**, .0225 and that showed segregated RCC was removed.

289 **TRA.500.006.0001**, .0029 ln 36-41.

290 Exhibit 324, **LOJ.003.0001**, .0012.

291 **TRA.500.006.0001**, .0029 ln 25-29.

292 **TRA.500.006.0001**, .0027 ln 6-41.

A. Yes.

Q. Did you get a good response, in your view, to any issues you raised?

A. From what I remember, yes, yes, we did - or I did. I shouldn't say 'we'.

…

Q. If a problem wasn't being dealt with to your satisfaction, how did you escalate or raise that to ensure your position was heard?

A. If that happened, there were different people throughout the project - we would go to the construction manager, and if we couldn't solve it, then we would go to the project manager. That didn't happen often.

5.162 Mr Lopez's evidence was that:[293]

> *Paradise RCC dam is not the first RCC dam that had some problems regarding construction procedures followed by the construction team, especially at the beginning of its construction considering that learning application of appropriate construction procedures of RCC technology usually takes some time. Building a hydraulic project, especially an RCC dam without some problems during construction is impossible. Generation of memos about QC and applied construction procedures is a normal condition that happens on this type of projects. Problems were detected on time giving alert in order to solve them ASAP, providing potential solutions to the construction team. This was part of our duties and responsibilities. Problems detected decreased over time as it is usual, and they were solved by the Construction team.*

5.163 Mr Embery was of the view that the issues raised in the memoranda were not repeated problems.[294] The problems raised were 'absolutely' addressed during the construction of the Dam and generally before the memoranda were written.[295]

5.164 Mr Hamilton spoke of the role of Mr Lopez and Mr Montalvo and what became of the issues that they raised:[296]

> *MR HAMILTON: They were largely independent, if that makes sense. They wrote reports. That's what they did. They were very, very clearly there to make sure that we got that job right.*
>
> *MR HORTON: We see they would do memoranda from time to time, and you were one of the addressees?*
>
> *MR HAMILTON: Yes. Often, often.*

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293 Exhibit 324, **LOJ.003.0001**, .0014.
294 **TRA.500.009.0001**, .0103 ln 21-25.
295 **TRA.500.009.0001**, .0121 ln 9-14.
296 Exhibit 309, **TRA.510.022.0001**, .0017 ln 3 to .0018 ln 29.

MR HORTON: Often?

MR HAMILTON: Yes, which is what they were supposed to do, I mean, absolutely what they were supposed to do. If it was wrong, we fixed it.

MR HORTON: Let's focus on that. How did that work? A memo would be generated saying there was an issue?

MR HAMILTON: Yes, it would be a memo for record, normally, 'This is what happened. This is what we did. This is how we fixed it', or, 'You guys need to change this. It's not working'.

I think the other thing to understand, too, is that this is a massive sort of production event, putting down RCC. There could be a couple of hundred men that are just involved in doing this. Obviously we wanted to get that right, so if it wasn't, we stopped or we had to pull stuff out. They were basically training along the way to make sure that that production was as efficient and as effective as it could be.

…

MR HORTON: What we're interested in is then how is what they say considered and remediated or not, as the case may be? Who's the decision-maker on what happens with that?

MR HAMILTON: I think it largely just was what they said, we did. Sometimes - yes, it wasn't always happy families there. I'm sure there was some conflict, and remember this is a 24-hour operation, so those guys are there, or one of them is there, whenever we're placing RCC. They're sleeping there. If there were issues, they were always elevated. If there were issues that they felt weren't resolved or we weren't getting the productivity that we thought, they were elevated. They were elevated to Bruce or myself or to Ernie, to Richard, Tim.

MR HORTON: Where will we see those? Are there records of the elevation and how issues are dealt with?

MR HAMILTON: I don't know. I mean, they could be in minutes of the alliance management team meetings. They could have been discussions. They would have flowed through in some of our QA-type reports and recordings.

5.165 The Commission did not have minutes of the Alliance Management Team meetings. In the same interview as the exchange above, Mr Hamilton was asked how satisfaction could be obtained that problems raised by the RCC Engineers were rectified:[297]

MR HORTON: … I think you're saying to me, well, first, they're dealt with practically on the spot; second, to the extent they're not dealt with, we'd find

[297] Exhibit 309, **TRA.510.022.0001**, .0033 ln 2 to .0034 ln 24.

them in a non-conformance. You were the project manager. Did you have other ways that you were satisfied, because you seemed satisfied, that that process was working at a practical level? I want that insight from you.

MR HAMILTON: Sure, sure, absolutely. One thing we tried to do was develop a culture of excellence on that job. …

I would say at every level of that operation, there was an intent to do a great job, from the guys that were operating the posi-tracks, to the shovel, to the guys that were in the batch plant, and that culture permeated, I believe, through everybody that was there.

…

The reality is I'd come in in the morning and I'd look at the figures for the night, and I'd go, 'Well, guys, what happened?' They'd say, 'Well, Robert and Jose just stopped the operation because we weren't doing it right.' I'd say, 'Let's have a meeting and discuss why we were not doing it right, and let's get it right'.

MR HORTON: But I think Mr Embery had difficulty with them taking those sorts of actions and warned them off doing that?

MR HAMILTON: I don't think that's true. Bruce has an angry disposition sometimes, as you might have found out. Those guys would actually know that if Bruce had overridden anything, I'd have been all over Bruce, all over him, and I would support them, because they were the experts and they were doing it. Yes, I'm not saying that there was never any friction between Bruce and those guys, and they would have been a pain for Bruce, quite frankly, but that was their job.

MR HORTON: But you're an experienced project manager… There must be ways that you satisfied yourself that what was being done - not only just intent and culture, but you seemed to have a judgment in the laying of the RCC here, according to what you knew and your knowledge, that it was being done well?

MR HAMILTON: Well, absolutely, and we were following, to the best of our ability, that intent of the specification. There's no doubt about that. If there was a problem, it was rectified. Why do I have that confidence? Because I know that Roberto or Jose would have said, 'This is wrong, Mark', and they would have gone straight to Ernie. They would have been on the phone to Ernie that night, going, 'This is wrong. These boys aren't doing it properly.' They would have rung me up in the middle of the night and said, 'Bruce has come out and said, 'Keep going', and we're not happy', and I would have come out. I can't remember an instance where that happened.

Bedding mix as the standard remedy

5.166 Relevant to the assessment whether construction problems were remedied is the reliance on bedding mix to remedy a range of quality control problems.

5.167 Removing a layer of RCC was drastic remedial action. Before taking that step, the sufficiency of other practical engineering measures was considered. Applying bedding mix to poor quality lift joints became the common practical measure used.[298] Bedding mix was applied to increase the LJQI score.[299] As is discussed in Chapter 4, the addition of points for bedding mix was not consistent, was not based upon the Specification, and effected a doubling up of points where up to 6 points was already built into the LJQI to reflect the treatment of joints with bedding mix.

5.168 Along with Dr Schrader, Mr Lopez had been involved in the design of an RCC dam in Colombia called the Miel I Dam. Mr Lopez's role included developing the RCC mix trial program, which included '*exhaustive*' laboratory testing of different RCC mixes.[300] Testing was conducted to determine the impact on cohesion and friction of different lift joint maturities, using retarder admixtures, undamaged and damaged lift surfaces, and bedding mix treatment. The results were said by Mr Lopez to show that the '*effect of bedding mix on the cohesion of the lift joint was very big as it is possible to see in the following figure*':[301]

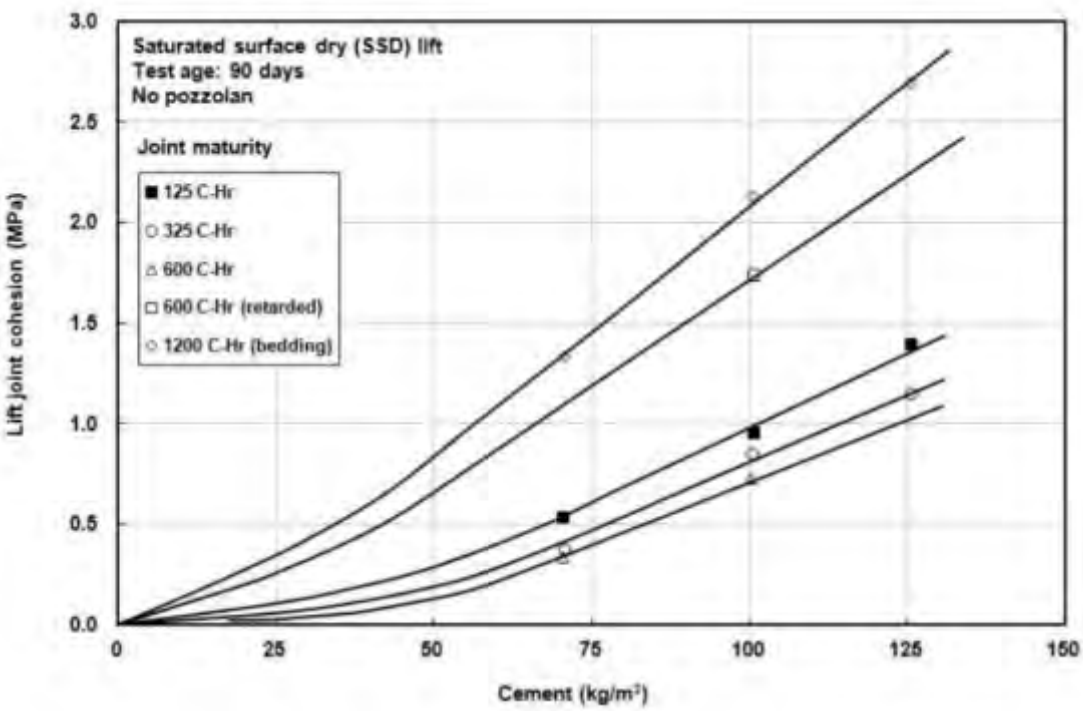

*Figure 5.16 – Shear test summary results showing cohesion vs cement content for different lift joint maturity and lift joint treatment. (Exhibit 324, **LOJ.003.0001**, .0018)*

298 Exhibit 324, **LOJ.003.0001**, .0019.
299 Exhibit 247, **TRA.510.007.0001**, .0031 ln 30-33.
300 Exhibit 324, **LOJ.003.0001**, .0018.
301 Exhibit 324, **LOJ.003.0001**, .0018.

5.169 The chart shows that cohesion was higher for a more mature lift joint treated with bedding mix than for an untreated less mature lift joint. For instance, taking the results for an RCC mix with 100 kg/m^3 of cementitious material, cohesion for a lift joint with maturity of 1200 ºC-Hr treated with bedding mix was approximately three times that of an untreated lift joint with maturity of 600 ºC-Hr.

5.170 Mr Lopez also relied on the chart below as showing that different cementitious content, lift joint maturities and the application of bedding mix had minimal impact on the friction angle. This is similar to Dr Schrader's evidence that, based on his personal database, '*the type of mix, lift joint quality, maturity, and degree of cleaning ... had little or no impact on [friction] angle*'.[302]

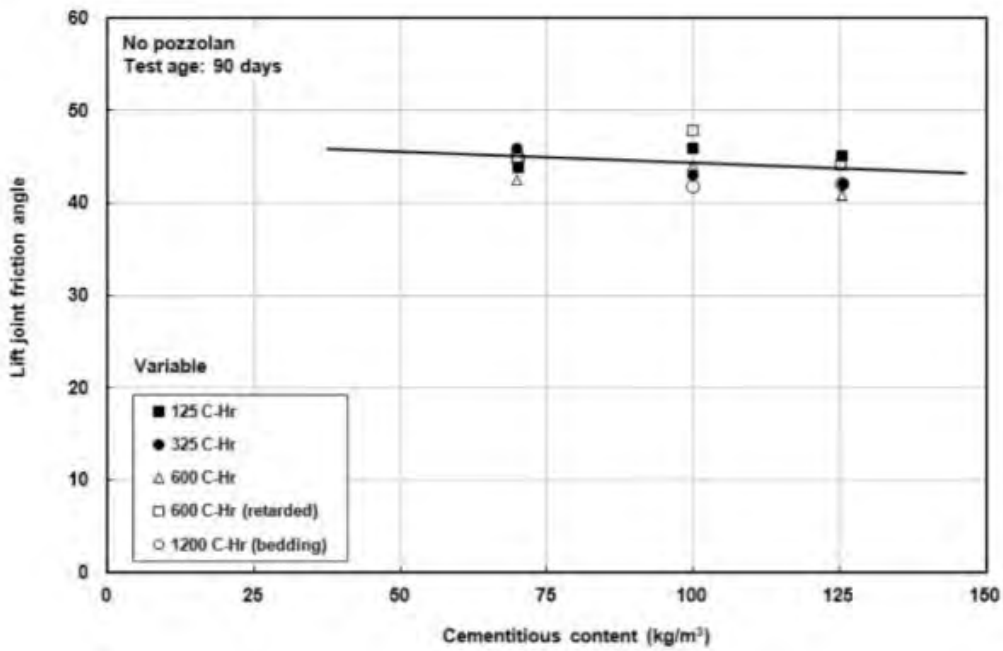

*Figure 5.17 – Shear test summary results showing friction angle vs cement content for different lift joint maturity and lift joint treatment. (Exhibit 324, **LOJ.003.0001**, .0019)*

5.171 On the basis of those results from testing for the Miel I Dam, Mr Lopez stated that:[303]

> *[I]nstead of removing the RCC layer affected by superficial segregation, a typical instruction provided to the construction team at the dam site, was clean the affected area applying air jet with low pressure and covering it with bedding mix before being covered by the following RCC layer (see Figures below). In this way, shear parameters were not affected when this issue happened during construction of Paradise RCC dam, increasing the cohesion of the affected area regarding the cohesion that could be obtained in a cold joint type I and type II indicated in the Technical Specification. Verification of the application of this procedure on small and isolated segregated areas by the RCC Engineers was done during Paradise RCC dam construction.*

302 Exhibit 109, **SCE.019.0001**, .0005.
303 Exhibit 324, **LOJ.003.0001**, .0019.

5.172 Mr Lopez's evidence did not explain how the application of bedding mix was so verified. There are scant records reflecting where bedding mix was applied. The quality assurance forms originally devised to record that detail were either not properly used or were amended so as to remove the burden on the RCC Inspectors (Mr Brampton and Mr Rickert) of having to make the record at all.

5.173 The following figure from Mr Lopez's witness statement shows how he instructed construction personnel on treating areas of RCC that were segregated or where it '*was not possible to clean properly with the target to avoid affect construction schedule*'.[304]

*Figure 5.18 – Instructions provided to construction personnel by Mr Lopez on treating areas of segregation with bedding mix. (Exhibit 324, **LOJ.003.0001**, .0020)*

5.174 Mr Lopez refers to using bedding mix where proper cleaning could not be done without affecting the schedule. That suggests scheduling pressures may have compromised quality, which the Specification sought to avoid.

304 Exhibit 324, **LOJ.003.0001**, .0019 (errors in original).

5.175 The use of bedding mix is not a substitute for the proper cleaning of lifts. Dr Schrader said as much in a memorandum written on 18 November 2004 (which appears to concern the practice to which Mr Lopez referred):[305]

> *I understand from Jose that when you have an area of clean-up that would otherwise hold up production, or if it is marginal, you just put bedding on it. This is a good solution to keep [production] going, providing that the bedding is available. However, if you have a skim of mud over the surface, the mud must be first removed.*

5.176 Mr Tarbox said that if lift joints were not sufficiently clean before the next layer of RCC was placed:[306]

> *[I]t is questionable whether use of the bedding mix would have improved the bond between the RCC layers. The bedding mix is not like a glue: that is not what it is supposed to do. One is not gluing the layers together. The purpose of that technique is to apply an enriched material that has a much higher slump than the RCC. It can be put down on the surface and broomed into the previous layer. When new RCC is applied, a boundary exists between the new and the old that is rich in cementitious material (unlike the RCC). If that surface is not saturated surface-dry or if it has been contaminated with dirt, oil, mud, etc. and is not clean, the bedding mortar simply falls on that unprepared surface. It is debatable whether it would provide the expected or design level of shear strength in that circumstance.*

5.177 Mr Tarbox said that account could be taken for the shearing resistance contributed by bedding mix on the upstream portion of lift joints when designing a dam.[307] When asked whether bedding mix was also able to overcome poor construction practices on lift surfaces, Mr Tarbox replied:[308]

> *It has a certain capacity to contribute towards that, I guess, but I certainly would not ascribe to nor recommend that that's what you are doing it for. If you are intending to create a more positive barrier to seepage water entering the dam along the lift lines and therefore you are putting the bedding mortar there, and if you want to take credit for that, for contributing some cohesion and resisting shear, that's all well and good. If, along the way, it compensates for some oversight in the construction practice, so be it. But I certainly wouldn't do, nor would I recommend that one adopt that as a standard practice for how to design a dam.*

5.178 Many of the lift joints in the Dam were cold joints. Mr Tarbox gave the following evidence about whether applying bedding mix to an upstream portion of the lift was

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305 Exhibit 197, **DNR.011.1381**, .1382.
306 Exhibit 100, **TAG.001.0001**, .0014 [46].
307 **TRA.500.007.0001**, .0032 ln 35-46.
308 **TRA.500.007.0001**, .0033 ln 11-22.

enough to overcome lift joint quality problems on the unbedded remainder of the lift:[309]

> *If the downstream portion of that same lift, let's say, for example, was contaminated with a vehicle that was driven on to the surface that was intended not to be on that surface or was to be restricted from being on the surface, and they contaminated that lift surface so that you could not get good frictional resistance, or let's say that it lay out in the hot sun for many, many hours so that it depleted or deteriorated the capacity of that lift surface to have a good coefficient of friction, then that lift - even though the portion upstream that was intentionally called a cold joint and treated, the whole 75 or 80 per cent of the rest of the lift downstream could have been compromised by virtue of a different one of the construction practices that I pointed out that had not been religiously and consistently followed 24/7.*

5.179 Whether, in those circumstances, the construction of the Dam was adequate had to be considered in light of the design of the Dam as a whole. Relevant to this issue was the specified treatment for cold joints.

Cold joint treatment with bedding mix

Specified requirements

5.180 Dr Schrader wrote[310] what became section 11 of the Specification. It provided that:[311]

> *It is the intent of this contract to place the entire roller-compacted concrete mass with sufficient continuity so that it hardens and acts within each monolith as one block without discontinuous joints or potential planes of separation. … When the time or maturity limits between successive placements of RCC layers exceeds those described below, a cold joint will be considered to have occurred, and the procedures described below for cold joints shall be followed.*

5.181 Cold joints were, by the Specification, to be treated with bedding mix with a nominal thickness of 25 mm over the upstream 25% portion of the lift surface for Type I cold joints, and over an extra 20% of the upstream face for Type II cold joints, i.e. a total of 30% of the lift surface for a Type II cold joint. The upstream face of lift joints always required a strip of bedding mix to be placed. The original specified width of that strip was 500 mm[312] but the later-dated construction drawings increased that to 600 mm.[313]

309 **TRA.500.007.0001**, .0031 ln 4-18.
310 **TRA.500.010.0001**, .0010 ln 5-10.
311 Exhibit 21, **DNR.003.8385**, .8466 [11.10.1].
312 Exhibit 21,**DNR.003.8385**, . 8470
313 Excerpt from **DNR.006.0001**, .0017.

Dr Schrader's relaxation of the specified requirements

5.182 In November 2003, during the stage 2 design, Dr Schrader was asked what the time to trigger a cold joint was likely to be.[314] Dr Schrader responded by saying:[315]

> *Do not worry much about cold joints!! With our design they are not a real technical problem. I put something in the spec about cold joints because we needed to address it in a way that would seem credible, but Richard can confirm for you that, except for a few critical areas this is not a worry with our design.*

5.183 On 2 August 2004, Dr Schrader sent a memorandum to Mr Herweynen and Mr Hamilton. Because the Dam '*essentially achieves stability with current friction values alone*', Dr Schrader reduced the requirement for bedding mix from 25% to 10% on the upstream side of a lift surface for a type I cold joint and from 30% to 15% for a type II cold joint.[316]

5.184 Both recipients of the memorandum could not recall whether the relaxed requirement was implemented, although Mr Hamilton remembered the width of the upstream bedding mix being relaxed at one stage.[317] The change that Dr Schrader recommended would have, according to Mr Herweynen, required his authority to implement and a documented formal change. As Mr Herweynen could not recall that occurring, he could not recollect the change being implemented.[318]

5.185 Dr Schrader was more definitive in his evidence. When interviewed, he said that the change was never made: '*It was never reduced*'.[319] However, Dr Schrader later said that he had been in contact with Mr Lopez who had told him that the amount of bedding had never been reduced.[320]

5.186 Other evidence paints a different picture:

a. The inspection point on the Cold Joint Treatment Checklist for bedding mix accorded with the relaxed requirement. That is, the checklist said that bedding mix was to be placed across 10% of the upstream portion of a Type I cold joint, and across 15% for a Type II cold joint.[321]

b. A memorandum from the RCC Engineers dated 13 September 2004 (that was sent to Mr Embery and copied to Mr Hamilton and Mr Herweynen) included a guideline for the application of bedding mix. The stated width of bedding mix for

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314 Exhibit 327, **SUN.010.002.0292**, .0294.
315 Exhibit 327, **SUN.010.002.0292**, .0292.
316 Exhibit 37, **SUN.010.002.0047**.
317 Exhibit 309, **TRA.510.022.0001**, .0035 ln 34 -38.
318 Exhibit 244, **HER.001.0001**, .0038 [180].
319 Exhibit 127, **TRA.510.006.0001**, .0010 ln 26-29.
320 Exhibit 127, **TRA.510.006.0001**, .0014 ln 35-45.
321 Exhibit 117, **DNR.020.014.4624**, .4626.

cold joints in the guideline was in accordance with Dr Schrader's relaxed requirements.[322]

c. The relaxed requirement was restated in a later memorandum from Dr Schrader to Mr Herweynen on 23 September 2004.[323]

d. Mr Lopez recalled that almost all the joints were cold joints and that the amount of bedding mix used was 10% for a Type I cold joint and 15% for a Type II cold joint.[324]

e. Mr Montalvo remembered Dr Schrader's memorandum reducing the percentage of bedding mix and recalled that:[325]

> *Yes, I received it. I remember there was a discussion about it, and from the photos, I believe we acted on it. I don't have the exact or definite memory of acting on it, but I believe so.*
>
> *Q. Did you act on other memoranda from Dr Schrader in the course of the project?*
>
> *A. Yes, of course. If we received it, we would act on it.*

5.187 The contemporaneous documentary evidence and the recollections of the RCC Engineers are to be preferred over the accounts of Mr Herweynen and Mr Hamilton and Dr Schrader. Mr Lopez and Mr Montalvo, who were more closely involved with RCC placement, gave evidence that the requirement was implemented. The contemporaneous documents corroborate those recollections. It is more likely than not that the change was implemented.

The basis for relaxing the cold joint treatment

5.188 The stated basis upon which Dr Schrader relaxed the requirements for cold joint treatment was that the Dam '*essentially achieves stability with current friction values alone*'.[326] That was not the first time Dr Schrader had said that. He prepared a draft Specification for the Dam dated 27 June 2003.[327] At that time, the design was in stage 2. The draft Specification said that:[328]

> *Although maximum compressive stresses are only on the order of about 2 MPa,* ***friction between lift joints essentially provides sliding stability with no cohesion****, and maximum tensile stress is on the order of about 0.1 MPa for minimal isolated areas, the estimated mix design given below for the vast*

322 Exhibit 31, **SUN.009.002.0147**, .0178.
323 Exhibit 181, **DNR.011.1232**.
324 **TRA.500.011.0001**, .0007 ln 3-17.
325 **TRA.500.006.0001,** 0038 ln 36-44.
326 Exhibit 37, **SUN.010.002.0047**.
327 Exhibit 23, **ALC.002.001.1176**.
328 Exhibit 23, **ALC.002.001.1176**, .1176 to .1177 (emphasis added).

majority of the dam is expected to result in strengths on the order of about 12 MPa at 1 year.

5.189 When the Section 11 of the Specification was approved for issue on 22 April 2004, it retained the same comment as quoted above in section 11.3.2.[329]

5.190 Mr Neumaier was familiar with the use of the word '*essentially*' in the quoted passage. In evidence, he said that the use of the word was '*unfortunate*' but accepted that it was written in the Specification which he had reviewed.[330]

5.191 On 12 November 2003, Mr Herweynen asked for Dr Schrader's opinion, and a basis for it, about what shear strength parameters the final design should be based on.[331] In Dr Schrader's response, he said:[332]

> *When I reviewed the stresses and factors of safety for our design, it was apparent that we do not need anywhere near perfect lift joints. Friction (phi) stays about the same even with very poor quality joints. Cohesion drops with lesser joint quality, but if I remember correctly* ***cohesion is not even needed for much of our dam, or we only need a little****. So, I took this into account when I prepared the specifications.*

5.192 That passage suggests that Dr Schrader had conducted a considered review of the stresses and factors of safety for the Dam. However, he testified that:[333]

> *There isn't anything in my files that I can find that shows that I even- that I had notes about it. But you wouldn't need to necessarily do that. If I was shown a table and it showed that factors of safety were substantially high with the excellent conditions, they were quite acceptable with good conditions, and even with poor conditions, then the factors of safety were going to be suitable, then that's all you have to look at and say, 'Well, it appears that we're okay'.*

5.193 When it was suggested to Dr Schrader that it was '*a dangerous thing, a risky thing and inconsistent with good engineering practice to have advised on this matter without having done proper, recorded, repeatable analysis*', he responded: '*Talk to the designer*'.[334] Hydro Tasmania and Mr Herweynen were relying on Dr Schrader's expertise and advice about what the shear strength values should be so as to satisfy the shear friction factors of safety.

5.194 On 18 January 2004, Dr Schrader sent a memorandum to Mr Herweynen about the material properties of various RCC mixes. Dr Schrader wrote, '*We do not need*

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329 Exhibit 21, **DNR.003.8385**, .8446.
330 **TRA.500.015.0001**, .0016 ln 9-21.
331 Exhibit 88, **DNR.005.4886,** .5139.
332 Exhibit 88, **DNR.005.4886**, .5141 (emphasis added).
333 **TRA.500.010.0001**, .0019 ln 47 to .0020 ln 8
334 **TRA.500.010.0001**, .0052, ln 7-11.

cohesion, but this also is a beneficial property, especially just below the spillway crest'.[335]

5.195 When the Detail Design was being finalised, Dr Schrader wrote to Mr Herweynen and Mr Lopez saying that:[336]

> *The lower lifts where the dam has a lot of weight are not as critical. They will have good friction even if the surface is not very good … because the weight will provide resistance to sliding.*

5.196 That statement was not based on a considered analysis of whether sufficient sliding resistance would be provided by the geometry of the Dam.[337] However, it is open to read that passage as offering that assurance to the designer and the RCC Engineer who was tasked with evaluating the design stresses in particular locations of the Dam as relevant to poor quality lift joints.

5.197 Dr Schrader helped prepare a presentation on 21 September 2004. One of the slides says that the RCC cannot fail by sliding even if there is no bond between the RCC layers. The '*slight exception*' to that was at the top of the spillway where steel anchor bars would be installed.[338]

5.198 Despite Dr Schrader's repeated contentions that the Dam was 'essentially' stable without cohesion, Mr Griggs gave evidence that Hydro Tasmania did not check whether the Dam was stable without cohesion.[339] During the tender design, he said, '*sliding factor assessments*' were carried out that showed that the majority of the Dam achieved stability using friction alone. The exception was the upper part of the Dam, which required bedding mix to bring it up to standard.[340]

5.199 On the basis of that evidence, Hydro Tasmania submitted that there was a reasoned basis for statements that the Dam essentially achieved stability with friction alone.[341] Dr Schrader could not recall whether he checked whether the Dam was in fact stable without cohesion.[342] He accepted that an assessment of whether sliding stability was essentially achieved with friction alone went to the fundamentals of the assessment of the Dam's stability.[343] However, his evidence was that such an analysis was not necessarily required, as revealed by the quote in paragraph 5.192 above.[344]

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335 Exhibit 146, **DNR.011.1361**, .1362.
336 Exhibit 164, **SUN.010.002.0356**, .0356.
337 **TRA.500.010.0001**, .0022 ln 24-25.
338 Exhibit 282, **SUN.020.003.6480**, .6486.
339 **TRA.500.014.0001**, .0086 ln 23-25.
340 **TRA.500.014.0001**, .0086 ln 39 to .0087 ln 5.
341 **HYT.008.0001**, .0033-.0034 [99]-[102].
342 **TRA.500.010.0001**, .0051 ln 25-28.
343 **TRA.500.010.0001**, .0051 ln 30-39.
344 **TRA.500.010.0001**, .0020 ln 2-8.

5.200 According to Mr Herweynen, the Dam needed cohesion for its sliding stability.[345] Mr Neumaier was aware that the sensitivity analysis in the Detail Design Report relied on cohesion[346] and said that '*there was probably not an assessment based on friction alone*'.[347] Neither Mr Herweynen nor Mr Griggs pointed to any record of their having taken issue with Dr Schrader's assertion. This is despite the memorandum relaxing the cold joint treatment requirements asserting that the matter had been discussed and agreed with Mr Herweynen.

5.201 The design of the Dam was never altered to rely on friction alone. There was always a requirement to apply bedding mix to the upstream face of every lift joint. Some bedding mix was to be applied to cold joints. So much may be accepted. However, the understanding that the Dam was essentially stable without cohesion seems to have provided sufficient comfort to the design team to (approximately) halve the amount of bedding mix needed on cold joints.

5.202 When asked about the justification for reducing the amount of bedding mix to be used, Dr Schrader's evidence was that:[348]

> *[T]he numbers in the spec were about what's been used on many other projects up to that point, it was kind of the state of the art, and it made good sense where some projects had done some testing. But I looked at these factors of safety - this is what I think transpired. I looked at these factors of safety from Richard and I said, you know, 'We can back off a little bit on the amount of bedding mix that we're using'.*

5.203 Dr Schrader could point to no such analysis. There was no evidence to explain whether the original specified treatment of cold joints was based on a mathematical analysis of how much additional cohesion was needed on cold joints due to the reduction in quality and, presumably, in the shear strength able to be achieved on a cold joint. In light of Dr Schrader's evidence that the numbers in the Specification reflected what had been used on other projects, it is possible that no reasoned analysis was done for the original requirement. The likely absence of those calculations may explain why no one thought to update the stability analysis when the bedding mix requirement was relaxed.

5.204 There is no evidence that the relaxation of the cold joint treatment method was reasoned and analysed, whether by Mr Herweynen, Dr Schrader or anyone else involved in the design of the Dam. That is despite the Specification saying that:[349]

> *After taking into account the effects of normal load which will be applied by the mass of RCC above any given layer at any point on the layer, adequate shear strength will be achieved if the specification requirements are followed.*

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345 **TRA.500.013.0001**, .0014 ln 29-33.
346 Exhibit 24, **GHD.002.0001**, .0156.
347 **TRA.500.015.0001**, .0016 ln 30-35.
348 Exhibit 127, **TRA.510.006.0001**, .0031, ln 22–29.
349 Exhibit 21, **DNR.003.8385**, .8521.

5.205 As is discussed below, Mr Herweynen believed that the shear strength properties in table 5-4 of the Detail Design Report had been achieved (at least in part) because of the treatment of cold joints with bedding mix.[350] He looked at the quality control records and proceeded on the basis that placing bedding mix would provide the necessary cohesion to achieve the intended stability.[351] A comparison between cohesion design values for treated and untreated lifts shows that adding bedding mix was thought to increase cohesion by a factor between 7 and 8, depending on the quality of the lift joint. However, there appears to be no evidence about what cohesion a cold joint might achieve, with or without bedding mix.

5.206 Mr Herweynen said that the RCC placement schedule never envisaged the number of cold joints that eventually developed. That may be understood as meaning that when the Dam was designed, the eventual prevalence of cold joints was not contemplated. However, once it came to be built and cold joints became frequent, heavy reliance was placed on the boost to shear strength that it was assumed bedding mix would provide. However, no documentation was presented that would confirm that a quantitative estimate of that boost was made. Sufficient cohesion may have been achieved. But the designers could not have been sure of that without testing that was not carried out.

5.207 In Dr Schrader's evidence, he suggested that Mr Herweynen did the calculations to justify the relaxed bedding requirement.[352] However, Mr Herweynen could not recall whether he did such an assessment at the time,[353] and agreed that if there was less bedding mix, it could lead to inadequate shear strength.[354]

5.208 To assist the Commission, Mr Herweynen calculated whether sufficient cohesion would have been provided by the relaxed bedding mix requirements. He was satisfied that applying bedding mix across 10% of a lift joint would provide average cohesion across the lift joint in excess of the 325 kPa design value for a good lift joint. He said:[355]

> *But one thing that I did do a quick check on is even if we had 10 and 15 per cent, based on the parameters that Dr Schrader gave us for 'with bedding mix', which was 2,400kPa, if we adopt 2,400kPa and the average that we need is 325kPa, if we applied 2,400 over a 10 per cent lift joint, you would get 325kPa on average.*

5.209 This appears to be inconsistent with what Mr Herweynen later said:[356]

> *I already gave that calculation just before, that if you adopted the 2,400kPa and you wanted to then have an average of 325, you only have to have 7.4 per cent of the lift surface covered in bedding mix to get the 325kPa.*

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350 **TRA.500.013.0001**, .0057 ln 33 to .0058 ln 12.
351 **TRA.500.013.0001**, .0058 ln 14-19.
352 **TRA.500.009.0001**, .0005 ln 11-29.
353 **TRA.500.013.0001**, .0019, ln 11-12.
354 **TRA.500.013.0001**, .0019, ln 1-3.
355 **TRA.500.013.0001**, .0019, ln 3-8.
356 **TRA.500.013.0001**, .0068 ln 32-35.

5.210 That Mr Herweynen undertook that assessment recently, and in the face of doubts about the Dam's stability, begs the question of whether that check was done at the time the bedding mix requirement was relaxed and, if not, why not. The Commission, in any event, has doubts about the correctness of the calculation. A simple area ratio calculation gives effective cohesions of 240 kPa and 360 kPa for 10% and 15% coverage, respectively, of bedding mix with a cohesion of 2,400 kPa.

Conclusions about construction problems

5.211 All things considered, the issues raised in the construction memoranda tended to be about isolated problems. The issues were not repeated with the frequency needed to give rise to serious misgivings about the quality of RCC placement in the Dam. Problems seem to have been detected by the RCC Engineers, raised with construction and design personnel and (as can be seen on many of the construction memoranda) remedied. Even where remediation is not documented, the problems are not so widespread as to justify concern for the Dam's stability. As Mr Tarbox said, the Dam has a reserve capacity to accommodate some undefined proportion of deficiencies including, for example, isolated patches of segregation or other poor practice without impacting on the stability of a lift (i.e. the Dam as a whole).[357]

5.212 The construction memoranda were part of a quality assurance system. That part seems to have been adequate to remediate quality control issues as the RCC Engineers detected them during construction. Although these problems raised were of concern, they were not manifestly widespread. That assessment is material for present purposes.

5.213 Other ways in which quality assurance issues were raised were in RCC Placement ITPs and LJQI Scorecards, NCRs and during discussions on site.[358] Regardless of how issues were raised, the application of bedding mix came to be used as the standard remedy for quality issues. However, bedding mix could not remedy all problems. For instance, when applied to an area of a lift that had not been properly cleaned, it was questionable whether bedding mix would improve the bond.

5.214 While many of the issues raised in the construction memoranda do appear to have been remedied, there were only 80 of those documents. By contrast, there were hundreds of RCC Placement ITPs and LJQI Scorecards. Those forms documented the daily inspections of lift joints and were completed by RCC Inspectors. The forms were seldom endorsed by the RCC Engineers at the time of RCC placement. In many cases countersignatures of the RCC Engineers were not made for substantial periods of time after the relevant RCC had been placed and well after an opportunity to remedy any issues had been lost. When asked whether it was possible that problems with segregation were not rectified, Mr Lopez agreed that was a possibility.[359]

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357 **TRA.500.007.0001**, .0024, ln 44 to .0025 ln 1.
358 **TRA.500.009.0001**, .0080 ln 34-38.
359 **TRA.500.011.0001**, .0005 ln 37 to .0006 ln 6.

5.215 The documents before the Commission do not provide evidence that all construction problems were remedied. The only evidence of that are the assurances offered by witnesses during Commission hearings, including Mr Montalvo, Mr Lopez, Mr Hamilton and Mr Embery.

5.216 No criticism is made of the substantial effort that must have been required of the QA personnel to produce the substantial volume of documentation that prompted observations that 'exceptional' records were kept.[360] However, in light of the structural deficiencies in the quality assurance and quality control system mentioned in Chapter 4, it cannot be concluded that remedial measures addressed all the problems encountered. That matters because the choice to build with LCRCC meant that '*fairly rigorous adherence to good construction practices*' was required.[361]

Certification

5.217 The then Department of Natural Resources and Mines granted Development Permit 176904 for construction of the Dam on 30 October 2003.[362] Condition DS6 concerned as-constructed documentation. Item 3(i) of that condition required:[363]

> *The as constructed documentation must include… [c]ertification by a registered professional engineer under the Professional Engineers Act 2002 (RPEQ) that the works have been constructed in compliance with all appropriate engineering standards including signed statements from the dam designer that principal components of construction have been inspected and approved. Such components shall include:*
>
> · *Dam foundation and foundation treatment.*
>
> · *Test results of the concrete used in construction.*
>
> · *Adequacy of any joints and waterstops in the concrete.*
>
> · *Structural adequacy of all principal elements.*

5.218 A full-time design presence was maintained on site during construction.[364] Mr Griggs was the on site design presence from March 2004 until July 2005, when William Curlewis took his place.[365] Mr Herweynen explained the expectations of the design presence in a memorandum dated 27 February 2014.[366] They included, relevantly, to provide sufficient assurance that what had been constructed was in accordance with the drawings, the Specification and the intent of the design. Evidence of that

360 Exhibit 53, **SUN.020.003.6637**, .6637.
361 Exhibit 127, **TRA.510.006.0001**, .0036 ln 36–42.
362 Exhibit 28, **DNR.003.7173**.
363 Exhibit 28, **DNR.003.7173**, .7181.
364 Exhibit 244, **HER.001.0001**, .0045 [204].
365 Exhibit 244, **HER.001.0001**, .0045 [205].
366 Exhibit 88, **DNR.005.4886**, .4944

assurance was to be in the form of a completed design inspection and test plan for each component of the Dam. The memorandum said that:[367]

> *Richard Herweynen – Design Team Leader, who is a RPEQ, will sign off on each of these Design Inspection & Test Plans.*

5.219 In a presentation prepared for a meeting with the Regulator on 19 February 2004, the following was said in relation to Mr Herweynen's involvement in the on site design presence:[368]

> *Third of my time will be on site at key times of construction to see foundation preparation, trial embankment, start of the RCC placement along the main dam, start of spillway crest concrete.*

5.220 On 25 November 2005, Mr Herweynen certified that, subject to particular remedial work being done, the dam was at '*Practical Completion*'.[369] That certification referred to an earlier letter of 5 October 2005. In that letter, Mr Herweynen had provided the following design sign-off before impoundment:[370]

> *I certify that the works as constructed have been undertaken in a manner which meets the design requirements for the dam.*

5.221 In support of the sign-off before impoundment, the memorandum said that design inspection and test plans had been established to ensure that the design intent was achieved for all aspects of the Dam, including RCC.[371]

5.222 While acknowledging that inspection and test plans were an important part of the quality control.[372] Mr Herweynen did not look at them during the project. Instead, he relied on the monthly summaries of quality control[373] and the existence of inspection and test plans.[374]

Satisfaction that construction complied with engineering standards

5.223 When Mr Herweynen certified the Dam as, firstly, ready for impoundment and, secondly, as at practical completion, the corehole required by the Specification had not been drilled.[375] Despite the importance of shear strength in an RCC dam because each lift joint is a potential failure surface,[376] there was no quantitative verification that the design shear strength values had been achieved. Mr

367 Exhibit 88, **DNR.005.4886**, .4945.
368 Exhibit 278, **DNR.020.019.2606**, .2641.
369 Exhibit 306, **DNR.005.0584**, .0828.
370 Exhibit 4, **SUN.126.001.0001**, .0001.
371 Exhibit 4, **SUN.126.001.0001**, .0001.
372 **TRA.500.013.0001**, .0051 ln 35-38.
373 **TRA.500.013.0001**, .0050 ln 36-39.
374 **TRA.500.013.0001**, .0051 ln 35-38.
375 Exhibit 21, **DNR.003.8385**, .8477.
376 **TRA.500.002.0001**, .0012 ln 8-13.

Herweynen's evidence was that there was never any intent to do a validation test on shear strength.[377]

5.224 Instead Mr Herweynen satisfied himself that the Dam had been built in accordance with design intent based on:

a. his visual confirmation of the quality assurance and control system being implemented[378]

b. the full time on site presence of Mr Griggs[379]

c. Mr Herweynen's own site inspections during visits to the site approximately every two months for around two weeks at a time[380]

d. Mr Herweynen's review of quality control records (including the RCC QC Reports)[381]

e. discussions with construction personnel.[382]

5.225 In relation to the achievement of shear strength parameters across lift joints, Mr Griggs's presence on site was not relevant. Mr Griggs was not involved in assuring that placement of RCC was in accordance with the design.[383] That role fell to Mr Lopez and Mr Montalvo, neither of whom were involved in designing the Dam. However, Mr Lopez and Mr Montalvo had some familiarity with the design requirements. For instance, Mr Lopez was sent an email by Mr Griggs on 6 June 2004 attaching a table that showed '*the sensitivity of stability to lift joint quality at various levels & chainages in the dam*'.[384] The table set out the shear friction factors of safety for a 'good' and 'poor' lift joints at various levels in the Dam in the primary spillway, the second spillway – main section, the secondary spillway – ridge section, and the left abutment. The RCC Engineers also prepared the RCC QC Reports, the last of which referred to the design parameters for shear strength and linked them to a summary of the LJQI scores that had been returned.

How was the LJQI used?

5.226 In section 7 of the last RCC QC Report, Table 26[385] summarised the LJQI scores across the project using the ratings from the Specification:

377 **TRA.500.013.0001**, .0056 ln 21-22.
378 Exhibit 244, **HER.001.0001**, .0042 [193].
379 Exhibit 244, **HER.001.0001**, .0045 [204] and [205].
380 Exhibit 244, **HER.001.0001**, .0045 [206].
381 Exhibit 244, **HER.001.0001**, .0045 [206].
382 Exhibit 244, **HER.001.0001**, .0045 [207].
383 Exhibit 244, **HER.001.0001**, .0046 [209]-[210].
384 HYT.600.007.0003.
385 Exhibit 38, **SUN.110.003.0001**, .0101; see also Exhibit 21, **DNR.003.8385**, .8467.

LJQI		
Design & Specification Basis		
Acceptable		+1 to -3
Occasional w/RCC Eng Approval		-3
Occassional w/RCC & Design Eng Approval		-5
Nothing Ever Allowed		< -5
ACTUAL		
Excellent	> +1	9%
Good	+1 to -1	75%
Fair	-1 to -3	15%
Poor	-3 to -6	1%
Bad	< -6	0%

Table 26. LJQI SUMMARY

5.227 Tables 27 and 28[386] presented '*information related to the stability of the dam which is affected by the LJQI. It can be seen that all construction estimates exceed the design basis*'. Tables 27 and 28 provided the following:

Bedding Mix Used	Proposal Estimate	Design Basis	Construction Start Estimate	Current Estimate
No	0.40*	0.32	0.40*	0.40*
Yes	2.80*	2.60	3.10*	2.80*

*Note: Includes Reduction of Estimate for Not Having Site Specific Full Scale Tests

Table 27. Lift Joint Cohesion Estimate (MPa)

Bedding Mix Used	Proposal Estimate	Design Basis	Construction Start Estimate	Current Estimate
No	45*	40	45*	45*
Yes	45*	42	45*	45*

*Note: Includes Reduction of Estimate for Not Having Site Specific Full Scale Tests

Table 28. Lift Joint Friction Angle Estimate (Degrees)

5.228 These are the same as the figures presented by the Alliance to SunWater during a due diligence review in August 2005[387] that was attended by Mr Herweynen.[388] His evidence was that he did not know the basis of the estimated shear strength values in the presentation and deferred to Dr Schrader and Mr Lopez.[389] However, Mr Herweynen also said that the basis of the estimates would have been the design values that Dr Schrader had provided.[390]

5.229 Dr Schrader attended the due diligence workshop in August 2005. Mr Montalvo recalled that Dr Schrader presented the part of the presentation about the shear strength properties.[391] Dr Schrader also said that the estimates about cohesion and the friction angle were 'most likely' his.[392] Asked about the basis of the estimated

386 Exhibit 38, **SUN.110.003.0001**, .0101.
387 Exhibit 39, **ALC.001.001.1874**, .1882.
388 **TRA.500.013.0001**, .0037 ln 39-46.
389 **TRA.500.013.0001**, .0039 ln 30 to .0040 ln 12.
390 **TRA.500.013.0001**, .0039 ln 2-11.
391 **TRA.500.006.0001**, .0040 ln 2-11.
392 **TRA.500.010.0001**, .0047 ln 44 to .0048 ln 4.

values, he denied having used the LJQI scores to derive them. However, he was unable to articulate what the basis of the estimates was.[393]

5.230 Dr Schrader's evidence was that the LJQI scores could not be used to establish shear strength parameters on site:[394]

> *It seems that somehow somebody got the erroneous idea that the LJQI is used to establish or calculate values for cohesion and friction.* ***This is absolutely not the case.*** *The LJQI is simply a tool that can be used to help assure thorough and appropriate inspection and QA/QC of lift joints, and also to be sure that lift joint inspection is well documented. Without the LJQI, lift joint inspection can slip through the cracks, the basis for acceptance is often arbitrary or not clear, and documentation has been lacking.*

5.231 Dr Schrader's closing submissions stated that:[395]

> *The LJQI is simply a tool used to help assure that thorough and appropriate inspections are done, and also to be sure that lift joint inspections are documented. There also is a table that provides suggested "adjustment" factors that can be applied to cohesion and friction for different qualities of joints.*

5.232 The Detail Design Report had set out assumed shear strength properties for 'excellent', 'good' and 'poor' lift joints. Those values were based on Dr Schrader's advice about what percentage reduction should be applied to 'probable' values to arrive at 'design' values for cohesion and friction. Different percentage reductions were recommended for 'excellent' and 'poor' lift joints. On 12 November 2003, Mr Herweynen sent an email to Dr Schrader attaching a document that contained information that Dr Schrader had previously given to the Alliance.[396] In the following terms, the document described what was meant by the 'excellent' and 'poor' lift joint classifications:[397]

- *'Excellent' relates to excellent construction practice with an all conveyor system, no rain, clean surfaces, good inspection, compacted within 30 minutes of mixing, average temperatures at the time of compaction <24, surfaces are near SSD when RCC is placed.*

- ***'Poor' relates to poor constructions practice where RCC Lift Joint Quality Index is less than – 3 (refer ES Spec)*** *due to segregation, rain, poor inspection, late delivery, higher temperatures, drying of surface and other issues.*

5.233 That explanation of 'poor' lift joint suggests a connection between LJQI scores and the shear strength parameters that the lift joint could be expected to achieve. A

393 **TRA.500.009.0001**, .0053 ln 38 to .0054 ln 23.
394 Exhibit 109, **SCE.019.0001**, .0010.
395 **SCE.036.0001**, .0021.
396 Exhibit 88, **DNR.005.4886**, .5139.
397 Exhibit 88, **DNR.005.4886**, .5140 (emphasis added).

further such link is hinted at by the column headers in tables 5-4 and 5-5 of the Detail Design Report – '*LIFT JOINT QUALITY INDEX*' – beneath which the design values for internal friction and cohesion are set out for poor, good and excellent quality lift joints. Then there is this table from the Specification showing what ratings were attracted depending on the LJQI score given to a lift joint:[398]

RCC Lift Joint Quality Index (LJQI) Guidelines.

RATING		LJQI (sum of points)
1	Excellent	>+1
2	Good	+1 to -1
3	Fair	-1 to -3
4	Poor	-3 to –6
5	Very Bad	<-6

Figure 5.19 – Banding for LJQI scores from the Specification

5.234 As shown in table 26 from the last RCC QC Report, an 'acceptable' LJQI score was in the range from -3 to +1. Seemingly on the basis of 99% of the LJQI scores being higher than -3, the estimates of cohesion and the friction angle across lift joints accorded with the values from tables 5-4 and 5-5 for 'excellent' joints.[399]

5.235 The notion that the LJQI was to be used to correlate scores on site with design parameters is bolstered by other considerations.

5.236 First, if no link was to be made between the LJQI scores and the design parameters for lifts of different ratings, the different ratings for lift joints seem unnecessary. A minimum LJQI score would have sufficed. In addition, the labels given to the ratings align with the labels the designers used to define the parameters for lift joints of varying quality.

5.237 Secondly, Dr Schrader's 1999 article explained how the LJQI was used to evaluate whether (and what percentage of) design strength values had been achieved on site. Reading from the relevant graph in Figure 16, an LJQI score of -2 would have '*a probable cohesion that is 90 per cent of the design basis with a probable friction angle that is 98 per cent of the design basis*'.[400]

5.238 Thirdly, Mr Hamilton wrote a letter dated 8 September 2003 that explained the increase in design cohesion values during the design stage:[401]

Load Combinations

398 Exhibit 21, **DNR.003.8385**, .8467.
399 Exhibit 24, **GHD.002.0001**, .0141.
400 Exhibit 124, **PDI.040.0001**, .0025.
401 Exhibit 306, **DNR.005.0584**, .0714 to .0715 (emphasis added).

The value adopted for cohesion at RCC lift joints was generally 100kPa, which is a conservative estimate of cohesion for a lift joint without bedding mix. In the upper parts of the dam, increased cohesion was required between lift joints for flood loading, which was achieved by placing a bedding mix between lift joints. Where bedding mix is specified the cohesion adopted in the stability analysis was 800kPa. These design parameters were conservative estimates provided by Ernie Schrader early in the Stage 2 period, for the lean mix design.

Ernie Schrader has extensive material property data for various mix combinations obtained from extensive trial mix programs for the many projects he has been involved in. Based on this data, he has developed regression relationships between measured parameters in the trail mix program and demonstrated material properties. Therefore the design parameters that Team 1 has adopted have already been validated by actual data from similar mixes.

*Based on Team 1's Trial Mix program and these regression relationships, complete material properties were provided for the proposed RCC mix. These properties are listed in Table 3 of Appendix D —Volume 2. The design cohesion values are obtained by multiplying the probable value given in Table 3 with the percentages given in Table 4 for various ages and lift joint quality. For a lift joint with and without bedding mix, the design cohesion values estimated for our proposed 70kg mix are as follows (**the range given is the bounds between poor lift joint and excellent lift joint — based on the quality index rating given in our specification**):*

Age (Days)	Without Bedding Mix	With Bedding Mix
7	50 - 200 kPa	280 – 1120 kPa
28	125 - 300 kPa	700 – 1680 kPa
90	200 - 350 kPa	1120 – 1960 kPa
180	250 - 400 kPa	1400 – 2240 kPa
365	275 – 400 kPa	1540 – 2240 kPa

*On the award of the contract, Team 1 will extend its trial mix program, including a trial crushing program. This trial mix program will provide additional validation of the material properties. In addition to this **Team 1 has specified a fairly extensive quality control program, which will also provide data to validate the material properties.***

5.239 The use made of the LJQI was consistent with Dr Schrader's 1999 article. That is, the LJQI was used to provide a framework for inspections of lift joints as well as to estimate whether the design parameters had been achieved.

5.240 As is discussed in Chapter 4, the LJQI was a useful framework for inspections. However, the LJQI suffered from flaws, in its design and in its application at the Dam.

Fundamentally, it did not assess compaction to the bottom of an RCC lift.[402] It was a subjective system of assessment dependent on the judgment of the inspector assigning the LJQI score. In some cases, the scoring on site may have doubled the numerical upside obtained from applying bedding mix. These problems beset the reliability of the scoring system. The use of the LJQI as an indication of whether the design parameters had been achieved must be called into question. Allocating a numerical score to what were essentially qualitative assessments seems to have beguiled the RCC Inspectors, RCC Engineers and the design team into thinking that the LJQI was more useful than it was.

Corehole sampling in January 2006

5.241 A core sample was taken from the Dam in January and February 2006. It was not tested for shear strength, only tensile and compressive strength. The Specification did not require that shear strength testing be undertaken. It required '*the quality of bedding and RCC between layers*' to be determined by coring and permeability testing.[403] No shear strength testing was undertaken on RCC and lift joints until 2015.

5.242 This section explains the circumstances of the 2006 coring, the tests and inspections of that core, decision making about shear strength testing immediately before and after the core was taken, as well as the use made of the results of this coring in GHD's assessment of the Dam's sliding stability.

5.243 The discussion is relevant to SunWater's involvement in the Dam's construction and commissioning of the Dam, to the uncertainty about the Dam's sliding stability and to the relative difficulty that attends the shear strength testing of LCRCC.

5.244 SunWater initiated a 'due diligence' of the Dam upon becoming aware it might have ultimate ownership and responsibility for it.[404] As part of that, members of the Alliance made a presentation to SunWater on 21 September 2004.[405] Part of the presentation concerned the properties of the RCC being used to build the Dam. It was said that '*More Detail on QC & Test Result To Date Will be Presented Later'.*[406]

5.245 Mr Herweynen and Dr Schrader discussed shear strength testing. In an email dated 14 May 2005,[407] Mr Herweynen asked for Dr Schrader's advice on the validation coring and whether shear strength testing should be undertaken. Dr Schrader recommended that a core be taken:[408]

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402 **TRA.500.009.0001**, .0046 ln 18-22.

403 Exhibit 21, **DNR.003.8385**, .8477.

404 Mr Paton (a member of SunWater's due diligence team) said it was undertaken because '*Sunwater was going to inherit Paradise Dam following completion of the construction of the dam*': Exhibit 91, **PAR.001.0001**, .0002 [4]

405 **TRA.500.010.0001**, .0045, ln 38-40; Exhibit 283, **SUN.020.003.6480**.

406 Exhibit 283, **SUN.020.003.6480**, .6487.

407 Exhibit 215, **SCE.023.0001**, .0002.

408 Exhibit 215, **SCE.023.0001**, .0001 [emphasis added].

> *Richard, I know this is a cost and may seem like a pain to those who just want to get the RCC in place and go home, but it is important to us. Think of it this way- (and remind Mark) that the* ***cost of doing the core is MUCH less than the cost of any one of the lifts that we could have taken out (and arguably should have per spec), but we left in place****. I think we owe it to ourselves, the project, the industry* ***and the future owners*** *to prove that what we constructed and left in place is acceptable –* ***especially since we said we would do it as a basis for getting the job****, and we left some potentially 'marginal' material in-place.*

5.246 SunWater conducted a 'due diligence workshop' on 11 and 12 August 2005. Mr Paton, Mr Brigden and Peter Richardson comprised the due diligence team. The Dam was by then nearing completion. The workshop was held at the Dam site. Dr Schrader and Mr Herweynen were also present. During the workshop, Mr Paton observed RCC being placed at the top of the left abutment. He said he '*had no reason to be concerned about the way that RCC was being placed*'.[409]

5.247 Immediately after the workshop, Mr Paton recorded his thoughts and may have circulated them in draft to the other members of the due diligence team. That draft is headed '*Burnett Dam, SunWater Due Diligence Workshop, 11 and 12 August 2005, Civil Group'.*[410] Two of the due diligence actions requiring completion were stated as being:[411]

- *A review of the achieved material strength properties (tensile capacity and shear strength) of the RCC.*
- *A review of the results of a proposed investigative core hole through RCC.*

5.248 Mr Paton wrote this because, he said, shear strength is an important design assumption in an RCC dam and he was interested in the outcome of testing.[412] Mr Brigden said the due diligence team had wanted to see shear strength test results.[413] There was no evidence of which Mr Paton was aware that shear strength testing had been undertaken before that workshop.[414] During the workshop, he became aware that a core was to be taken.[415] He presumed that the results of the coring would have provided information about the quality of the RCC.[416]

5.249 Mr Paton had limited experience with LCRCC at the time, and would have felt 'more comfortable'[417] if SunWater had been provided with hard data to confirm shear strength.

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409 Exhibit 91, **PAR.001.0001**, .0002 [7].
410 Exhibit 92, **SUN.016.010.4219**.
411 Exhibit 92, **SUN.016.010.4219**.
412 Exhibit 91, **PAR.001.0001**, .0003 [10].
413 **TRA.500.002.0001**,.0014 ln 19 to .0015 ln 34, .0025 ln 26 – 39.
414 Exhibit 91, **PAR.001.0001**, .0003 [10].
415 Exhibit 91, **PAR.001.0001**, .0003 [10].
416 Exhibit 91, **PAR.001.0001**, .0003 [10].
417 Exhibit 91, **PAR.001.0001**, .0003 [11].

5.250 Mr Brigden noted the absence of any shear strength test results. He was:[418]

> *[L]ooking for the material strength properties, particularly tensile capacity and shear strength, of the RCC and [he] could not find any hard data. It appeared at that stage that … there had been no real testing for the shear strength properties*.

5.251 His expectation at the time was that a core could be taken for the purposes of shear strength testing:[419]

> *[T]he RCC had reached an age where perhaps not the top five or ten lifts but certainly down to depth would have been coreable. It was part of the specification and it was specified as 150mm core, and my expectations were that an angle hole would retrieve core, particularly at depth, and would demonstrate whether or not there had been a good bond established across the lifts and the hole was sited such that it would cover quite a few variables - one underneath a conveyor belt where you usually get a lot of spill and it's very hard to lift your conveyor belt; one where trucks were turning and screwing with their tyres; cold joints with mortar; hot joints without. That was my expectation. It was one way of getting a look of what the joints in the dam were like.*

5.252 During the due diligence process, Mr Brigden's expectation was that the core extracted from the Dam would be tested for shear strength. The core proposed to be taken was to intercept '*many, many lift joints*'. Over its length, that core would represent many joints encompassing all variables including the different materials and construction processes.[420]

5.253 Mr Paton wrote in his notes about the workshop[421] that *'Parent and lift joint shear strength tests have not been completed presumably due to the prohibitive cost*'. He could not recall why he had made that notation. He believes now that it would not have been expensive '*if all that was required was the shear strength testing on core samples*'.[422] Mr Brigden spoke also of the relative difficulty of shear strength testing LCRCC: shear strength testing, he said, *'is possible, but quite expensive, and cannot be performed on lean RCC, low paste RCC, until the concrete has sufficiently hardened to be able to support the forces of the drilling*'.[423]

5.254 Mr Paton could not recall if there had been agreement at the workshop whether shear strength testing would be done. He did, however, recall that no shear strength testing was performed on the cores obtained in early 2006. That understanding is correct, as shown below.

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418 **TRA.500.002.0001,** .0011 ln 39-46.
419 **TRA.500.002.0001,** .0025 ln 26-39.
420 **TRA.500.002.0001,** .0050 ln 11-42.
421 Exhibit 92, **SUN.016.010.4219**, .4222.
422 Exhibit 91, **PAR.001.0001**, .0003 [13].
423 **TRA.500.002.0001**, .0014, ln 33-36.

5.255 There was discussion within the Alliance immediately after the workshop about shear strength testing. Dr Schrader, in a memorandum about 'RCC Cores' dated 15 August 2005 to Mr Herweynen, Mr Hamilton, Mr Embery and Mr Montalvo,[424] referred to section 11.16.5 of the Specification and its requirement of '*coring and permeability testing of the RCC*'.[425] He noted (correctly) that the only testing contemplated there concerned permeability, and not shear strength, or compressive or tensile strength. He suggested, however, that a core be taken and testing be done, including because the Alliance had said to SunWater's due diligence team that it would be forthcoming. He suggested detailed steps for taking and handling the core, and proposed that it be inspected to see if the lifts were bonded:[426]

> *However, coring and core testing should be accomplished for other reasons. These include: (1) To verify for ourselves that adequate quality has been achieved, with insitu properties that exceed the design requirements; (2) Because the 'Due Diligence' team asked about cores and core information, and we advised them (without providing details to them about what this would consist of) that it would be forthcoming (3) At the time that the proposal was developed, the Alliance team (designers) met with the dam regulator, who concurred that it was justifiable to reduce the amount of testing (QC cylinders/cubic meter of RCC) from what would normally be done providing that we had good quality control/inspection of mixing and placing, and with the understanding that we would take cores after completion of the project for verification[427] (we did this); and (4) As a professional contribution to the industry.*
>
> *It is usually impossible to obtain good core from lean RCC mixes at less than 180 to 365 days, and then only with very careful drilling done at an angle, with at least a nominal 6 inch core, and a split inner sleeve. Because we perceive to have done one of the best lean RCC jobs ever, with a god mix, it may be practical to obtain reasonable core with drilling starting about October, especially if it is accomplished from the older right abutment, angling at about 30 degrees (or flatter) into the spillway section. To the extent achievable, the hole should be*

424 Exhibit 213, **ALC.002.001.0936**, .0937.

425 Exhibit 21, **DNR.003.8385**, .8477.

426 **ALC.002.001.0936**, .0937 - .0939.

427 The correctness of this assertion is doubtful. The Department of Natural Resources, Mines and Energy (DNRME) representatives, when asked for their view on this, said they could not find any records of any such meeting. Mr Ryan, who then worked with the Regulator, was said to recall attending one meeting only at the Dam but it was unrelated to this issue. Mr Allen's health precluded enquiries being made of him directly. DNRME's position was that neither the Department, nor its officers, had any official role in approving or signing off on such matters. On the one occasion when the documents do suggest the Dam Safety Regulator met with representatives of the Alliance, it was on 19 February 2005 [**DNR.005.4886**, .4888] and the notes do not suggest the issue of QC testing was raised. Dr Schrader seems to be placing reliance upon what he may have been told rather than from direct knowledge. He is not recorded as having attended the meeting on 29 February 2004 with the Dam Safety Regulator. Nothing in the notes is consistent with the Regulator having the alleged concurrence. The point is not material, however, because the type of testing concerned was not controversial in this Inquiry.

skewed downstream (plan view) so that it goes as much as practical towards centre of the dam. The hole does not necessarily need to extend all the way to the foundation providing that the following goals can be achieved.

- *The core diameter is greater than at least 4+ inches. For a nominal 6 inch drill hole diameter, and with an inner sleeve, the core will probably end up being about 5 inches diameter.*
- *Reasonably consistent quality and intact care is obtained. An exception may be the upper portion of the core where the mix is younger and may not have been compacted as quickly and completely as during the majority of mass placement.*
- *The core includes samples from the conveyor backswing area as well as samples from the area of dam placed more quickly by all-conveyor, without trucks.*
- *The core includes a reasonable representation of RCC lift joints with bedding and without bedding.*

If, after coring into the mass of the spillway, the quality of core is poor, drilling should probably be stopped. Either a change needs to be made in drilling equipment and procedures, or the RCC simply needs more maturity.

If practical, two pressure tests should be done in RCC with bedding and two should be done in RCC without bedding. Preferably this will include one test in the area placed by total conveyor and one test in the area placed with truck in the back-swing. The length of each pressure test should be about 3 meters. A long packer will be needed to seal against what probably will be a rough surface with some "ravel' where the hole crosses lift surfaces. Pressure testing can be done with a single packer as the drilling progresses (preferred) or with a double packer after the entire hole has been drilled. The pressure head should be on the order of 2.5 times the vertical distance from the top of the hole to the test location. The test should be continued for some time, probably on the order of about 15 minutes, until the water take reaches a stable rate. Notes should be taken of water take per each minute. It is not unusual that there is an initial higher water take that slows after the area being tested has been saturated.

The core should be placed immediately into boxes after removal from the inner sleeve, It should not be allowed to dry or roll around on the dam surface. It should be marked with permanent marker with relevant information including an arrow for the up direction. If they are clearly discernible, lift joints should be marked and numbered according to the same numbering system used in the dam. A note should be made with regard to whether each lift was bonded or non-bonded when the core was first retrieved and viewed in the split sleeve. The person doing the logging should be able to tell by inspection of any separated lifts as to whether the lift was initially bonded but fractured due to drilling or handling, or if it was unbonded in-situ. The bonded and fresh lift surfaces should show clean slightly fractured fresh surfaces with fractured mortar or aggregate

when viewed under a hand lens. Unbonded surfaces will not have this appearance, and probably will have traces of calcium carbonate along the lift.

The core should be photographed wet, wrapped with wet burlap … hessian … and bubble wrap, and set into the box, with the box sized so that the wrapped sample fits without force, but does not roll around. The lid (hinged) should then be closed. It is best if the boxes have rope or other handles at the ends for handling and transport. This minimises the chance for dropping the boxes during handling.

Selection of exactly what tests will be done on which core samples should be done carefully by an appropriate Engineer, preferably after all of the core is available for review. The Engineer should supplement basic notes that were taken at the time of sampling, making additional markings on the core as appropriate, and rephotographing the samples wet after these markings are added. This has been done effectively in the past by marking on the core box lid adjacent to the core, or directly on the core, and re-photographing. I will offer to do this at no cost for travel, with billing only for one day on-site, or in Brisbane lab, regardless of how long it actually takes.

At least 20 cores (preferably closer to 30 for statistical purposes)) should be tested in compression (with stress-strain curves on at least 8 samples). At least 8 cores should be tested for split tension. This should be split equally between cores in the backswing area and cores in areas placed directly by conveyor with no backswing. When we did this at Mujib dam, there was a very clear difference in strength and quality in the two areas, if there is adequate core and budget, it would be desirable to test enough cores so we can make a further distinction between cores with bedding and cores with no bedding, although for compression and split tension I do not expect a notable difference.

Each test specimen should be weighed in air and in water (displaced water volume determined) so an accurate saturated density can be determined regardless of the roughness and size of the core. Water displacement will establish the exact volume of the sample.

The ends of the samples will need to be carefully cut with a diamond saw on table. Main Roads has a set-up for doing this. They then should be capped with sulfur compound for compression testing. No capping is needed for split cylinder testing.

Longer samples should be reserved for compression testing, with shorter ones used for split tension. The L/D ratio should be between 2.2:1 and 1.8:1 for compression, and anything greater than 1:1 for split tension.

Shipping needs to be careful done so that the samples do not dry out or bang around. I have done this in the past by setting the boxes on a bed of damp sand in a truck (and sometimes driving the truck myself). We do not want a wild driver destroying good cores, nor do we want unsupervised laborers or technicians dropping heavy core boxes when they move them.

5.256 Dr Schrader's comments illustrate the difficulty of taking cores of LCRCC. If the cores are not taken and handled carefully, the reliability of testing or inspecting them is susceptible to challenge on the basis that separation in the lift joints might have occurred in drilling or handling and they may not be representative of the *in situ* condition.

5.257 SunWater continued its due diligence, but as a 'fitness for purpose' review. Two documents record the progress in that regard: one in November 2004,[428] the other in December 2005.[429] Mr Paton explained that the due diligence became a fitness for purpose review once SunWater had acquired the Dam by acquiring the shares in Burnett Water.[430]

5.258 The draft reports dated November 2004 and August 2005 were never finalised. They express opinions about the quality of the Dam's construction, the QC Process and other matters. The views expressed in those documents are of little value: those reports are drafts and they lack the analysis that would have been required if the views expressed there had been intended to be relied upon. As working drafts, their content does not materially advance the work of the Commission, other than as a record of SunWater's fitness for purpose review, which was never completed. Mr Brigden could not recall who told him to stop or what he was told, but someone *'would have told him'* to cease the work.[431] Mr Paton could not remember if he was instructed to stop or whether the due diligence team was waiting to see the coring report before finalising its work.[432]

5.259 The taking of the core proceeded. Mr Montalvo was responsible for it. In preparation, he spoke with Dr Schrader and '*the team a lot … to see what was needed, to make sure that we knew what the plan was and what exact tests we needed to perform*.'[433] The core was taken on a diagonal axis between 27 January and 11 February 2006. It was at Chainage 724 at RL 77.655. The alignment was inclined at 30 degrees from vertical and 5 degrees from the axis of the dam in the downstream direction. The top of the hole was offset downstream from the upstream face by 3.3 m. It had a depth of 35.5 m. There were 15 drilling runs.[434]

5.260 Mr Herweynen communicated with Dr Schrader in March 2006 regarding the testing to be undertaken on the extracted cores. Dr Schrader recommended forgetting all about shear tests.[435] Mr Herweynen abided by Dr Schrader's recommendation not to conduct shear strength testing, despite that being the means of confirming that the shear strength design values had been achieved.[436]

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428 Exhibit 54, **SUN.016.014.1266**.
429 Exhibit 55, **SWA.505.001.0014**.
430 **TRA.500.006.0001**, .0062 ln 25-35.
431 **TRA.500.002.0001**, .0019 ln 16-27.
432 **TRA.500.006.0001**,.0062 ln 2-17.
433 **TRA.500.006.0001**, .0030 ln 27-30.
434 Exhibit 95, **ALC.001.001.1683**, .1683.
435 Exhibit 217, **SCE.025.0001**; Exhibit 221, **SCE.029.0001**.
436 **TRA.500.002.0001**, .0029 ln 31 to .0030 ln 21.

5.261 In an email to Mr Herweynen and others on 13 September 2006, Dr Schrader noted that '*it is extremely difficult to get good cores when crossing lift joints in lean mixes. We expected this'*.[437]

5.262 Mr Montalvo wrote a report '*Preliminary Comments on RCC Coring at Paradise Dam*' dated 14 September 2006[438] about the coring, the results of testing on the core, and of his observations. That report states that it is a starting point for the '*Final Report to be issued shortly after review … by 'Mr Herweynen … and [Dr] Schrader …'.*[439] No such report was before the Commission and there is no record of it ever having been prepared.

5.263 Mr Montalvo's report makes no mention of permeability testing. Dr Schrader gave reasons in his memorandum[440] for why it was unnecessary. Permeability testing is not relevant for present purposes.

5.264 Mr Montalvo's report stated that core samples were carefully wrapped and transported. He noted '*some damage from drilling*' to the RCC cores and RCC lift joints.[441] The test conducted were for compressive strength only. No shear strength tests were undertaken, but Mr Montalvo's observations of the lift joints upon extraction of the core are detailed.

5.265 There were 15 drilling runs in total. The location for extraction was:[442]

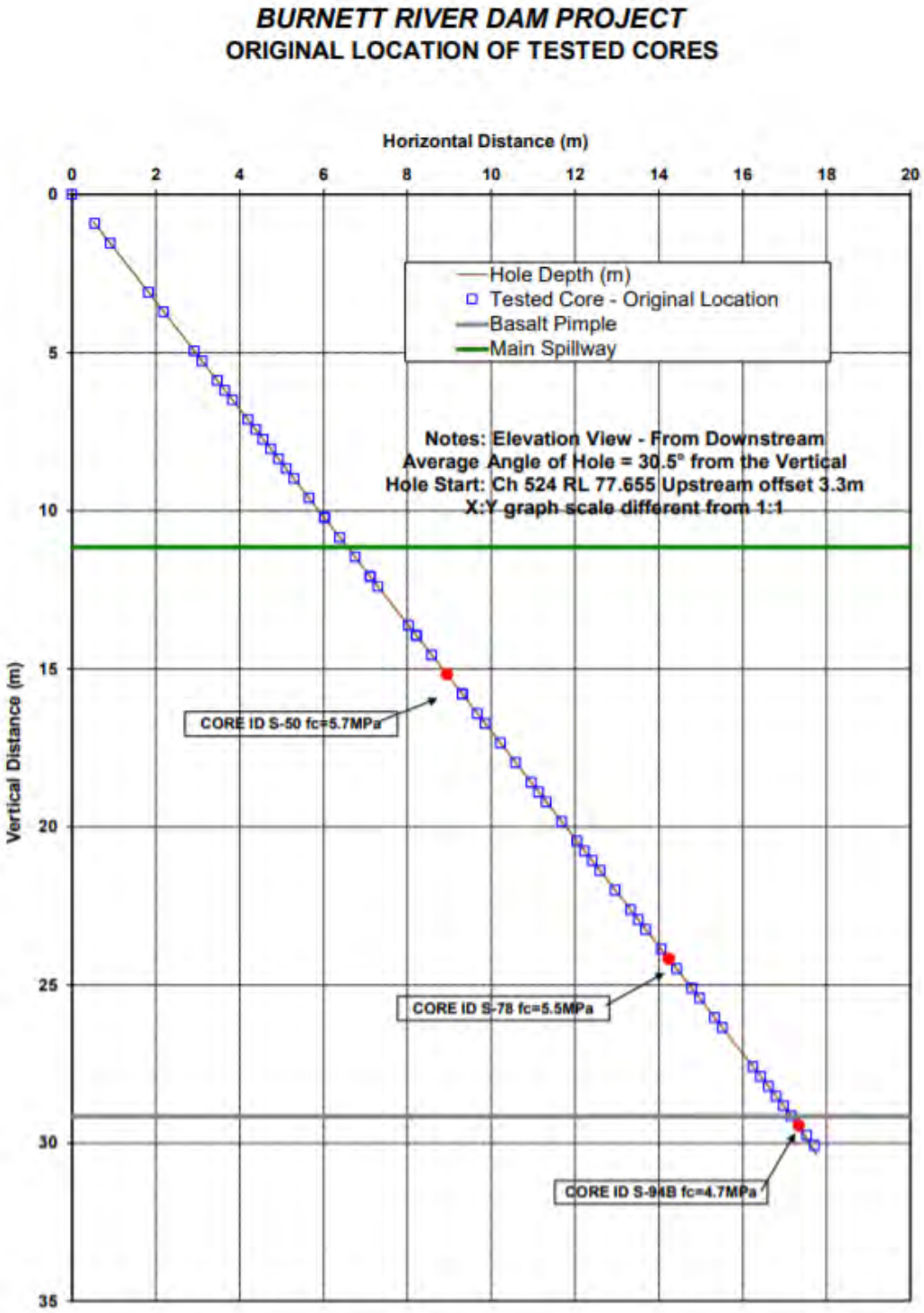
> *[S]elected with the aim of obtaining test data for the following placement scenarios that took place during construction:*
>
> - *All Conveyer placement (with/without bedding mix)*
> - *Placement with trucks or other (with/ without bedding mix)*

437 Exhibit 223, **SCE.031.0001**.
438 Exhibit 95, ALC.001.001.1683.
439 Exhibit 95, **ALC.001.001.1683**, .1683.
440 Exhibit 213, ALC.002.001.0936.
441 Exhibit 95, **ALC.001.001.1683**, .1683.
442 Exhibit 95, **ALC.001.001.1683**, .1683.

5.266 Mr Montalvo's report included the following photograph of the drilling rig in place on the right abutment with the diagonal alignment of the corehole.[443]

443 Exhibit 95, **ALC.001.001.1683**, .1693.

5.267 The following graphic summary of the locations from which samples were taken was presented in Mr Montalvo's report.[444]



444 Exhibit 95, **ALC.001.001.1683**, .1699.

5.268 Mr Montalvo expressed these conclusions in the report:[445]

> ***Conclusions***
>
> *In summary, due to disturbance of the RCC cores the low results recorded during testing exaggerate the worst-case scenario of RCC compressive strengths, and even then the strength is well above design requirements. Issues documented during placement, especially in difficult areas, did somewhat decrease the quality of the RCC, but not in a general sense as most of the RCC in the dam does not fall in that category.*
>
> ***As designers, we knew that the lean mix would not be totally watertight at non-bedded lift joints, or at bedded lift joints that had some construction difficulty and for this reason adopted the upstream membrane, which is performing as expected.***
>
> ***As expected, the core strength results are generally lower than the cylinder results as a result of disturbance of the RCC material from the drilling process. However, in all cases the strength results are significantly greater than what is required within the dam based on stress analysis. Therefore from a design point of view the core results have validated that the RCC within the dam has acceptable strength properties and the dam should behave as intended.***

5.269 The last sentence records the core results as having shown the RCC to have 'acceptable strength properties'. That conclusion was not based upon laboratory test results for shear strength because no such testing occurred. Mr Montalvo, however, visually inspected the cores and expressed opinions about which contained lifts that appeared to be bonded and not.

5.270 His 'Core Test Summary' attached to the report[446] records 64 total lifts having been extracted in the cores, and 30 of them having been bonded. The number of cores and lifts examined is not entirely clear. As Mr Willey pointed out:[447]

> *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*.

5.271 The observations about bonding (or the lack of it) are derived from spreadsheets called 'coring logbooks' which Mr Montalvo created and completed.[448] There were 15 coring logbooks in total, each numbered with the 'run' to which it related.[449]

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445 Exhibit 95, **ALC.001.001.1683**, .1685 (emphasis original).
446 Exhibit 95, **ALC.001.001.1683**, .1732.
447 Exhibit 56, **WYJ.001.003.0001**, .0012 [39].
448 **TRA.500.006.0001**, .0032 ln 43.

5.272 Mr Montalvo compiled picture reports of each of the 'test runs'.[450] They show the position of the drill rig, the downstream appearance before and during drilling, the extracted cores, and different parts of the extraction process.

5.273 The photographic reports are consistent with Mr Montalvo having visually assessed whether the lifts extracted in cores were bonded.[451] He explained the process as follows:[452]

> *[W]hen you extract a core and the lift is bonded, then it's joined. When it was not bonded, then it would just separate and then you would have an independent core for each lift.*

5.274 Mr Montalvo followed the directions in Dr Schrader's memorandum[453] when taking notes and inspecting the lift joints for bond status.[454] His notes in the core logbooks were intended to capture that information.[455]

5.275 Mr Montalvo's report was not provided to SunWater by the Alliance. SunWater in its submissions listed the times it had sought the results of the 2006 core testing.[456] That report was provided to SunWater by Mr Herweynen in August 2019.[457]

5.276 Mr Montalvo's report (including the coring logbooks and the picture reports) was reviewed by GHD[458] in preparing the Stability Memorandum.[459]

5.277 Mr Willey interpreted the information in the coring logbooks as follows:[460]

> *What they have logged there is the RCC layer thickness; the elevation of the lift, or they have said the layer elevation, but I guess we've interpreted that as the lift joint, but it is not particularly relevant. The RCC layer number there refers to the - I believe that refers to the layer number as listed on the original construction drawings. They have interpreted an age, or an approximate age, of the RCC at the time it was extracted, so obviously from the construction records, at a certain*

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449 Exhibit 59, ALC.002.001.0717, ALC.002.001.0718, ALC.002.001.0719, ALC.002.001.0720, ALC.002.001.0721, ALC.002.001.0722, ALC.002.001.0723, ALC.002.001.0724, ALC.002.001.0725, ALC.002.001.0726, ALC.002.001.0727, ALC.002.001.0728, ALC.002.001.0729, ALC.002.001.0730, ALC.002.001.0731.

450 Exhibit 60, ALC.001.001.1589, ALC.001.001.1740, ALC.001.001.1809, ALC.001.001.1657, ALC.001.001.1580, ALC.001.001.1773, ALC.001.001.1503, ALC.001.001.1672, ALC.001.001.1789, ALC.001.001.1511, ALC.001.001.1846, ALC.001.001.1823, ALC.001.001.1618, ALC.001.001.1540.

451 **TRA.500.006.0001**, .0031 ln 34-41.

452 **TRA.500.006.0001**, 0031 ln 44-47.

453 Exhibit 213, **ALC.002.001.0936**.

454 **TRA.500.006.0001**, .0032 ln 13-19.

455 **TRA.500.006.0001**, .0032 ln 21-25.

456 **SUN.026.0001**, .0007-.0008, [32].

457 **SUN.026.0001**, .0008, [33].

458 Exhibit 56, **WYJ.001.003.0001**, .0010 [36]-[37].

459 Exhibit 16, **GHD.005.0001**.

460 **TRA.500.003.0001**, .0047 ln 20-39.

level and location they know roughly when that material was placed, and obviously the difference between that time and the time at which they were coring gives the age. I guess more importantly is the 'Cause and depth of core break'. So the people logging the hole were specifically requested to inspect the core and identify where lift joints were bonded or where they were not bonded, and for those that were not bonded, they were requested to inspect the core and determine whether the core had been broken by drilling.

5.278 Mr Willey read Dr Schrader's 15 August 2006 memorandum[461] as the instructions to undertake the inspection of the core and identification of the bonded lifts.[462] Mr Willey explained his interpretation of the 'cause and depth of core break' and 'remarks' columns in the coring logbooks in this way (to which there was no challenge):[463]

[W]here there is an X in the column, that is their interpretation from the person logging the hole as to the condition of the lift joint. So, for example, core ID S2, the second row in that sheet, there is an X under 'No bond', and the comment over on the right-hand side was 'Not bonded'. If you look at the one above, they note that it was 'Bonded', with 'good bond - bedding mix not apparent'. So it appears they inspected each lift joint quite diligently and classified them on that basis.

5.279 Mr Montalvo said that it was difficult reliably to assess whether the '*lifts were initially bonded but fractured due to drilling or handling or if something was unbonded in situ*', and he relied on advice from the drilling crew to do so.[464] It was not of concern to him that a large percentage of the lift joints were unbonded, '*because there was an expectation that some of them were not going to be, or in that location they were not going to be bonded*'.[465] The exchange in evidence with Mr Montalvo was as follows:[466]

Q. And whether something is broken by machine. Are you making an assessment there, at least in part, of whether lifts were initially bonded but fractured due to drilling or handling or if something was unbonded in situ?

A. Yes.

Q. Is that something you think you could reliably do?

A. No, if it was bonded and then unbonded by the drilling operation, that is really hard to do, so that's why I was relying on the advice from the drilling crew more than mine. I found that pretty hard to do.

Q. Did they tell you how they could know if it was unbonded in drilling?

461 Exhibit 213, **ALC.002.001.0936**.
462 **TRA.500.003.0001**, .0047 ln 41-43.
463 **TRA.500.003.0001**, .0048 ln 12-21.
464 **TRA.500.006.0001**, .0033 ln 6-16.
465 **TRA.500.006.0001**, .0034 ln 22-25.
466 **TRA.500.006.0001**. 0033 ln 6-23.

A. I don't remember. I guess if I put that in here, it was on their advice. But if I put it in here, it's because I took it and so I own it. I would have asked them. I just don't remember what they said.

5.280 As to whether he was able to determine whether the cores were 'unbonded' through the extraction process or subsequent transportation, Mr Montalvo said:[467]

Subsequent transportation is easy, because if they were bonded when extracted and they get to point B unbonded, then something happened during transportation. The damage or unbonding through extraction - that one, I relied heavily on the crewman that was extracting the cores.

5.281 Mr Montalvo's assessment derived from his visual assessment, necessarily a subjective judgment, and his reliance on the views of others.

5.282 The cores were sent to the Road System and Engineer Group at the then Department of Main Roads (**DMR**) on 28 September 2006.[468] DMR provided a report dated November 2006.[469]

5.283 The format for testing was said to be:[470]

6 Direct Tensile for cores marked as type cb (conveyor bonded)

6 Direct Tensile for cores marked as type c (conveyor / not bonded)

1 Direct Tensile for core marked as type t (truck / not bonded)

After Direct Tensile testing is performed, recover the longest section from each test and perform the following:

6 Indirect Tensile for cores marked as type cb (conveyor bonded)

6 Indirect Tensile for cores marked as type c (conveyor / not bonded)

Indirect Tensile for core marked as type t (truck / not bonded).

5.284 The results of the test make up the remainder of the report, along with a Combined Summary of Cores.[471] The summary shows that of the 13 cores tested, 7 were not bonded.

5.285 Photographic reports were also attached to the DMR report depicting each of the tested samples[472] and the testing process.[473]

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467 **TRA.500.006.0001**, .0032 ln 6-11.
468 ALC.001.001.1456, .1457.
469 ALC.001.001.1456.
470 ALC.001.001.1456, .1457.
471 ALC.001.001.1456, .1465.
472 ALC.001.001.1399.

5.286 Dr Schrader sent an email on 1 January 2007 to Mr Herweynen and Mr Lopez, and copied to (among others) Mr Hamilton and Mr Montalvo which noted:[474]

> *Main Roads has sent me a CD with their report. It is clear. They tested cores with bonded lift joints and cores without bonded lift joints. All cores were subjected to direct tension tests followed by indirect tension tests of the remaining piece.*
>
> *The overall lift joint strength is about where we said it would be. However, there is more to the story than this simple summary, and I would like to get it sorted. It is nothing that the rest of you need to worry about, but Jose and I really do follow through with the finalization of all data and what it means. This has to do mostly with the cores being lower than expected compared to the QC cylinders, especially with regard to split tension.*
>
> *Robert, there is no need to send me anything else with regard to the cores, but can you just verify if the cores that were tested in tension with bonded lift joints* ***did*** *have bedding, or* ***did not*** *have bedding. I have assumed they did not have bedding.*

5.287 Later email correspondence on the topic of whether the lift joints were treated is dated 1 and 4 January 2007. The first, from Mr Montalvo, states '*[r]egarding the presence of bedding in the bonded lift joints, I've been meaning to go through the QA files plus the field notes from coring to provide a precise summary, I'll do this in the next couple of days*'.[475] The email dated 4 January 2007 is from Mr Lopez. He says, '*Roberto has these updated files. We updated daily RCC tests results in our daily reports'*.[476]

5.288 Later, Dr Schrader said not to '*bother*' with shear testing unless enough cores from different angles were taken to get statistically significant data. He told Mr Herweynen and others, variously, to '*probably forget about shear tests*',[477] to '*forget about shear tests*',[478] and '*you will not be able to afford to do proper shear tests, so forget about them*'.[479]

5.289 LCRCC presents particular difficulty in producing cores which are reliable for such testing. That difficulty was greater here because the Dam had not fully cured as at early 2006. Although the right abutment was built first, not all of the parts of the Dam through which the core was taken had reached 180 to 365 days maturity.

5.290 The visual inspection by Mr Montalvo suggested that many lifts were unbonded. That was not, however, conclusive of the lifts having been unbonded *in situ*.

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473 ALC.001.001.1387.
474 Exhibit 219, **SCE.027.0001**, .0001 to .0002.
475 Exhibit 219, **SCE.027.0001**, .0001.
476 Exhibit 222, **SCE.030.0001**, .0001.
477 Exhibit 215, **SCE.023.0001**, .0001.
478 Exhibit 221, **SCE.029.0001**, .0002.
479 Exhibit 217, **SCE.025.0001**, .0001.

GHD's work

5.291 SunWater retained GHD to conduct engineering and technical studies on the Dam between 30 January 2013 and 30 November 2019.[480] Initially, GHD was involved in reviewing and developing future improvement works during the preliminary business case process.[481] In late 2017, as part of that process, GHD assessed the stability of the Dam to ensure that the proposed scope of works covered all potential failure modes.[482]

5.292 GHD was engaged by SunWater in late 2018 to undertake the preliminary design of two improvement works options for the Dam as part of SunWater's Dam Safety Improvement Project. GHD's work in that respect continues.[483] As part of that engagement, GHD prepared three memoranda relevant to the stability of the Dam.[484] Mr Willey was the key author. His memoranda were progress updates to SunWater.[485]

Shear Strength Memorandum

5.293 In a memorandum dated 5 September 2019, GHD presented the findings of a review into the shear strength of the RCC lift joints of the Dam based on existing test data. GHD also made recommendations about the shear strength parameters that should be adopted in analysing the Dam's stability (**Shear Strength Memorandum**).[486]

5.294 GHD's assessment was based upon previous stability analyses, shear strength test results from 2015 and 2019, and information about the post-construction corehole taken by the Alliance in 2006.[487] An evaluation of the 2015 shear strength test results and logs resulted in GHD concluding:[488]

> *[T]here typically appears to be minimal difference between peak and residual strength. Also, it is noted that assessment of the borehole logs and core photographs from holes through the RCC indicate that it is common for lift joints to be unbonded (80-90% of lift joints unbonded in DD600 and DD601) and therefore the sliding friction strength or residual strength is considered appropriate and cohesion should not be included.*

5.295 The 2019 shear strength test results were also discussed. Based on core logs, photographs and data from investigations of coreholes by optical and acoustic

480 **GHD.047.0001**, .0002 [1].
481 **TRA.500.003.0001**, .0013 ln 32-37.
482 **TRA.500.003.0001**, .0013 ln 30-40.
483 **GHD.047.0001**, .0002 [1].
484 Exhibit 15, **DNR.001.2363**; Exhibit 16, **GHD.005.0001**; Exhibit 61, **SUN.009.004.0037**.
485 Exhibit 56, **WYJ.001.003.0001**, .0008 [27].
486 Exhibit 15, **DNR.001.2363**.
487 Exhibit 15, **DNR.001.2363**, .2363.
488 Exhibit 15, **DNR.001.2363**, .2368.

televiewer, GHD determined that there was '*a high likelihood of debonded or segregated zones extending across lift joints*'.[489]

5.296 As for the 2006 corehole, GHD noted that 108 lifts were encountered of which 78% were unbonded. Those unbonded joints were typically smooth and '*segregation was commonly evident in core photos*'.[490]

5.297 Based on the prior investigations, of the order of 60% - 90% of lift joints were unbonded. Given the likely smooth nature of RCC lift joints, GHD suggested that peak unbonded shear strength values were not appropriate. Residual strengths should be adopted instead. GHD recommended that a friction angle of 39.3º and no cohesion should be used for the stability analysis of the Dam for normal stresses up to 600 kPa.[491] Those values compared to the design parameters of a 40.4º friction angle and a cohesion value of 325 kPa.[492]

Stability Memorandum

5.298 The second memorandum provided an update on GHD's review of the stability of the monoliths under different flood loadings (**Stability Memorandum**).[493] In accordance with the Shear Strength Memorandum, a friction angle of 39.3º and no cohesion were adopted in assessing the Dam's stability against acceptance criteria for a residual strength scenario in the 2013 *Guidelines on Design Criteria for Concrete Gravity Dams,* published by Australian National Committee on Large Dams (**2013 ANCOLD Guidelines**).[494]

5.299 The Stability Memorandum set out the input data and assumptions upon which the assessment was based. In the result, GHD concluded that:[495]

a. the stability of the primary spillway monoliths was considered 'marginal' at the peak of the flooding in 2013 (**the 2013 event**)

b. the main reason for the poor assessment of the Dam's stability was the significantly lower shear strength of the lift joints than had been assumed in the design.

5.300 Also significant were the different assumptions that GHD had made compared to those the designers had adopted in relation to uplift pressures beneath and within the Dam. The Stability Memorandum included a number of figures comparing the factors of safety for different monoliths in the primary spillway to the acceptance

489 Exhibit 15, **DNR.001.2363**, .2384.
490 Exhibit 15, **DNR.001.2363**, .2385.
491 Exhibit 15, **DNR.001.2363**, .2387.
492 Exhibit 24, **GHD.002.0001**, .0141.
493 Exhibit 16, **GHD.005.0001**.
494 Exhibit 16, **GHD.005.0001**, .0005, .0013.
495 Exhibit 16, **GHD.005.0001** .0019-.0020.

criteria in the 2013 ANCOLD Guidelines. The figure for monolith H is reproduced below.[496]

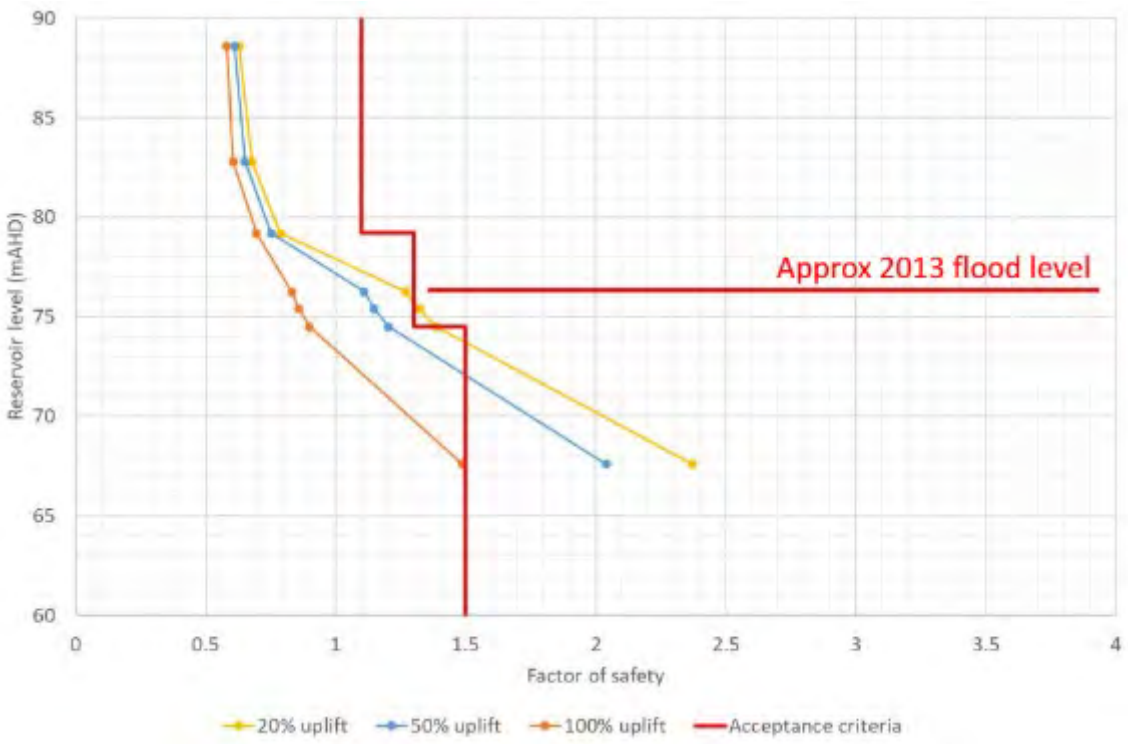

Figure 5.20 – GHD's assessment of the stability of monolith H

Updated Shear Strength Memorandum

5.301 The third of the memoranda, dated 28 February 2020, was an update to the Shear Strength Memorandum to include the results of a second round of testing on 2019 corehole samples (**Updated Shear Strength Memorandum**).[497] GHD remained of the view that there were likely to be widespread zones of debonding and segregation on the lift joints.[498] However, some of the detail was updated to reflect the additional test results.

5.302 GHD recommended that a friction angle of 38.2º (down from 39.3º) and no cohesion be used for normal stresses up to 600 kPa.[499]

5.303 The Updated Shear Strength Memorandum included the following summary of intrusive investigations and sampling with the dam by monolith. The summary included details of the testing undertaken on the samples retrieved.[500]

496 Exhibit 16, **GHD.005.0001**, .0015.
497 Exhibit 61, **SUN.009.004.0037**; Exhibit 56, **WYJ.001.003.0001**, .0008 [29].
498 Exhibit 61, **SUN.009.004.0037**, .0066.
499 Exhibit 61, **SUN.009.004.0037**, .0066.
500 Exhibit 61, **SUN.009.004.0037**, .0040.

Monolith	Coring/drilling	Shear strength testing
B	2019 D/S toe horizontal core (noting there are two at this location RCC-B and RCC-B1 as the first was abandoned)	RCC-B – Five triple stage tests in total – Triple-stage direct shear on two sections with three samples from first section (1.49-1.94m) and two samples on second section (4.65-4.95m)
C	2015 vertical core from crest (DD600) 2019 D/S toe horizontal core (RCC-C)	DD600 – Two triple stage test on lift joint samples at depths of 10.2 m and 12.2 m RCC-C – not tested
G	2014 D/S toe horizontal core	Understood to be three separate samples tested with each tested at different normal stress to obtain peak strength and then run three times for residual
L/K	2006 post-construction inclined hole	Not tested
N	2015 near-vertical core from crest (DD601) 2019 D/S toe horizontal core (noting there are two at this location RCC-N and RCC-N1 as the first was abandoned)	DD601 – Two triple stage test on lift joint samples at depths of 23.32 m and 24.8 m RCC-N & RCC-N1 – not tested
P	2019 D/S toe horizontal core (RCC-P)	RCC-P – Three triple stage tests in total – Triple-stage direct shear on one section with three samples (1.55-2.0m)
Q	2019 D/S toe horizontal core (RCC-Q) 2019 4 x vertical or near-vertical core from crest (PD-04, and PD-10 to PD-12)	RCC-Q – Three triple stage tests in total – Triple-stage direct shear on one section with three samples (4.1-4.87m) PD-04, and PD-10 to PD-12 – not tested
R	2015 vertical core from crest (DD602) 2019 D/S toe horizontal core (RCC-R)	DD602 – not tested RCC-R – not tested
S	2019 D/S toe horizontal core (RCC-S) 2019 inclined core from crest (PD-05 – 60° to horizontal)	RCC-S – Six triple stage tests in total – Triple-stage direct shear on two sections with three samples from first section (2.4-3.15m) and two samples on second section (1.3-1.75m) PD-05 – not tested
U	2015 vertical core from crest (DD603) 2019 D/S toe horizontal core (RCC-U)	DD602 – not tested RCC-U – not tested
V	2019 inclined core from crest (PD-06 – 60° to horizontal)	PD-06 – not tested

Figure 5.21 – GHD's summary of intrusive investigations and testing

5.304 The following plan of those corehole locations was provided to the Commission:[501]

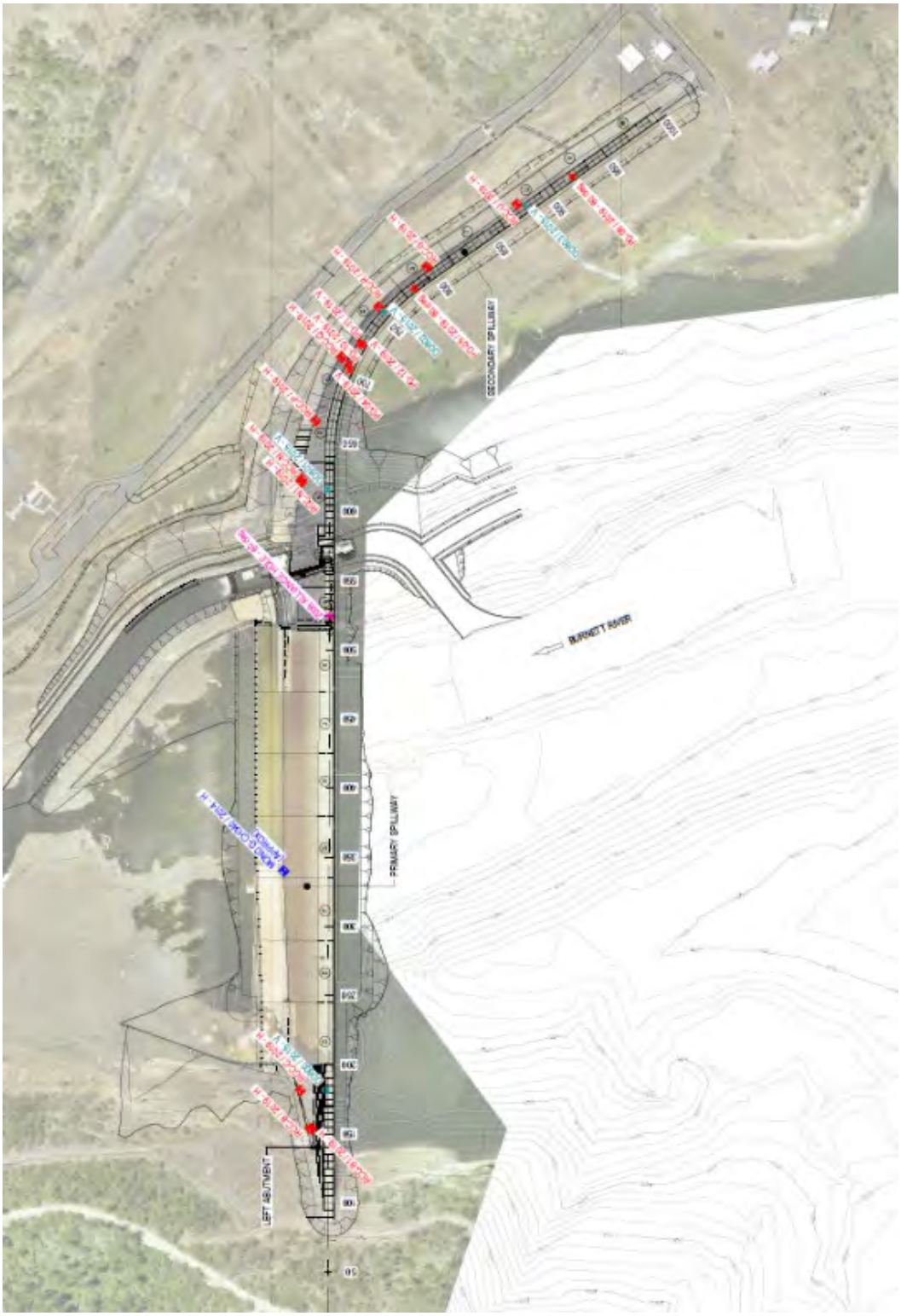

Figure 5.22 – GHD plan of corehole locations

501 Exhibit 65, **GHD.040.0001**.

5.305 GHD used all test results to derive the following shear strength results on a statistical approach. That approach sought to produce strength parameters with the 'reasonable certainty' of which the 2013 ANCOLD Guidelines speak. GHD explained that the statistical approach was:[502]

> *[U]ndertaken by determining a lower confidence limit such that there was a certain probability that the population mean was greater than the selected strength. This … method takes into account the variability in the sample results rather than accepting that the sample results truly reflect the distribution of the population. In this case, a confidence level of 97.5% was adopted.*

5.306 Based on the statistical assessment, GHD derived the following shear strength results using all shear strength test results from cores taken from the Dam:[503]

Parameter	Cohesion and friction	
	c (kPa)	ϕ (°)
"Peak bonded" – 200-600 kPa	150	34.8
"Peak unbonded" – 200-600 kPa – c & ϕ	138	35.7
"Peak unbonded" – 200-600 kPa – ϕ only	0	44.8
"Residual unbonded" – 200-600 kPa – c & ϕ	80	31.6
"Residual unbonded" – 200-600 kPa – ϕ only	0	38.2

Figure 5.23 – GHD's derived shear strength results using a statistical approach

2014 coreholes

5.307 In mid-2013, the first Technical Review Panel (**first TRP**) was formed to review SunWater's design and construction of phased remedial works on the Dam after the 2013 event. A report of the first TRP dated October 2013 discussed the Dam's stability and how the Dam might be threatened from flood flows through the spillway and over the right and left abutments in very large inflow floods.[504] Sliding failure mechanisms would control the Dam's stability. RCC lift joints were identified as potential failure planes to be included in any comprehensive stability review. The first TRP said that some field and laboratory investigations may have to be done.[505]

5.308 Horizontal cored holes were drilled in November 2014.[506] Cores were taken from the toe of monoliths F and G from downstream face. The holes were drilled through the reinforced conventional concrete cap and extended nearly 3 m into RCC.[507] The

502 Exhibit 61, **SUN.009.004.0037**, .0048.
503 Exhibit 61, **SUN.009.004.0037**, .0060.
504 Exhibit 7, **IGE.017.0001**, .0026.
505 Exhibit 7, **IGE.017.0001**, .0025-.0026.
506 Exhibit 10, **IGE.020.0001**, .0051.
507 Exhibit 10, **IGE.020.0001**, .0052.

cores were 142 mm in diameter.[508] The approximate location of the corehole from monolith G is depicted in blue in the plan above. The location of the corehole taken from monolith F is unknown[509] and is therefore not depicted in the diagram.

5.309 Graeme Bell was a member of the first TRP. He inspected the cores that had been retrieved and made the following comments at the time:[510]

> *The concrete facing cores suggested that the quality of the facing concrete was very good, but immediately behind the first of the RCC was definitely not of good quality. This RCC tended to break up on drilling, which does suggest that it was not well-compacted and looked as though it may have dried out quite quickly. I must admit that it reminded me very much of the apparent poor quality of the downstream face on both the right abutment and the left abutment dams. …*
>
> *According to SunWater's report on the drilling, the concrete facing was more than 1 m thick, up to 1.7 m in places. The poor quality RCC seems to have been 0.5-0.6 m thick in places. Thereafter, from the successful holes, the RCC cores could be extracted intact, although one broke along a layer boundary surface.*

5.310 Despite those observations, Mr Bell went on to state that:[511]

> *In all of the initial samples, the quality of the RCC in each layer on either side of the layer boundary surface looks to be very good and I for one would have no concerns about the shear strength of the intact RCC.*

and:[512]

> *I do believe that the peak strength parameters for the bonded layer boundary of 45°/200-300 kPa would be acceptable for the whole RCC mass.*

2015 coreholes

5.311 As part of geotechnical investigations reported by SunWater in 2016, two vertical coreholes were taken and subjected to shear strength testing.[513] Vertical coreholes of 83 mm in diameter were drilled in monoliths C (corehole number DD600), N (corehole number DD601), R and U.[514] The 2015 cores are depicted by the green points in the plan above.

5.312 Based on an assessment of the corehole logs and core photographs, GHD considered that it was common for lift joints to be unbonded.[515] Borehole imaging was compared against the core photographs to assess whether the retrieved core

508 Exhibit 61, **SUN.009.004.0037**, .0043.
509 Exhibit 56, **WYJ.001.003.0001**, .0014 [46].
510 Exhibit 10, **IGE.020.0001**, .0052.
511 Exhibit 10, **IGE.020.0001**, .0053.
512 Exhibit 10, **IGE.020.0001**, .0054.
513 Exhibit 61, **SUN.009.004.0037**, .0042.
514 **TRA.500.003.0001** .0060 ln 45 to .0061 ln 3.
515 Exhibit 15, **DNR.001.2363**, .2368.

was representative of the core *in situ*. GHD concluded that the condition of the core was generally representative of *in situ* conditions, although breakage of the core might be greater than the extent of unbonded lifts in the Dam.[516]

5.313 The following photograph shows a 3 m section of the core taken from monolith C in the left abutment:[517]

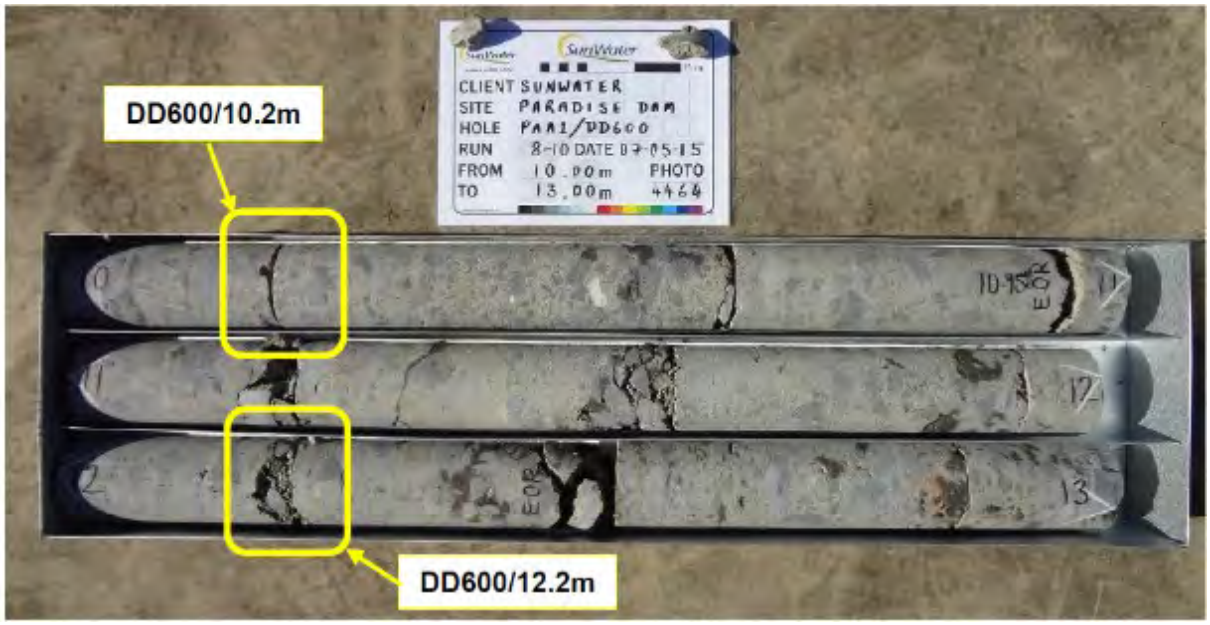

Figure 5.24 – Core retrieved from the left abutment in 2015

5.314 Sections of the core taken from monoliths C and N were tested for shear strength. The test results were included in the Updated Shear Strength Memorandum.[518]

5.315 In a report dated 15 December 2015, the first TRP expressed the view that '*over a full width of a section we should have a strength of at least 45º*'.[519]

GHD's 2019 investigations

5.316 In reviewing SunWater's preliminary design for dam improvement works, GHD reconsidered the results of the 2015 shear strength test results. GHD derived a shear strength of 37-38º.[520] Using that shear strength, a re-evaluation of the stability of the Dam indicated that it was not acceptable when compared with ANCOLD acceptance criteria. GHD recommended that additional sampling and testing be done.[521]

516 Exhibit 15, **DNR.001.2363**, .2372 to .2373.
517 Exhibit 15, **DNR.001.2363**, .2366.
518 Exhibit 15, **DNR.001.2363**, .2368; Exhibit 61, **SUN.009.004.0037**, .0042; Exhibit 56, **WYJ.001.003.0001**, .0013 [46].
519 Exhibit 10, **IGE.020.0001**, .0026.
520 Exhibit 56, **WYJ.001.003.0001**, .0014 [47].
521 Exhibit 56, **WYJ.001.003.0001**, .0015 [49]-[50].

5.317 Core samples were drilled and taken by SMEC on behalf of SunWater in 2019.[522] The 2019 cores are depicted by the red points in the plan above. Horizontal and vertical cores were taken from the secondary spillway and left abutment.[523] Holes were cored in 12 of the 20 monoliths in the Dam.[524] No samples were taken from the primary spillway. The curved crest means that temporary works would be required to install a drill rig.[525] Sampling from the downstream face of the primary spillway is also not without difficulty because the location of the lift joints is masked by the reinforced concrete.[526] Relevant to the lack of coring from and testing of RCC in the primary spillway, Mr Willey gave evidence that the design does not distinguish between the primary spillway, the secondary spillway and the left abutment. The assumed strength parameters and the Specification were the same across those locations.[527]

5.318 Testing was undertaken on cores from seven of the 20 monoliths, including 21 triple stage direct shear tests and three single stage tests.[528] The method of selecting samples for testing was set out in the Updated Shear Strength Memorandum, along with photographs of the sections of core selected as suitable for testing.[529] During hearings, four RCC experts were asked whether the samples were suitable for testing. All agreed that they were.[530]

5.319 The methodology for direct shear testing on sections of core taken in 2019 was set out in the following detail in the Updated Shear Strength Memorandum:[531]

- *Select three samples from each section of core to be tested and prepare as required*
- *Normal stresses recommended for testing are 200 kPa (σ_{n-1}), 500 kPa (σ_{n-2}) and 1,000 kPa (σ_{n-3})*
- *Sample 1 from each section of core is to be tested as follows:*
  - *Test at σ_{n-1} to obtain 'peak bonded' shear strength and stop test as soon as failure is achieved*
  - *Test at normal stress of σ_{n-1} to obtain 'peak unbonded' shear strength and stop test as soon as failure is achieved*
  - *Test at normal stress of σ_{n-2} to obtain 'peak unbonded' shear strength and stop test as soon as failure is achieved*

522 Exhibit 56, **WYJ.001.003.0001**, .0015 [51].
523 Exhibit 15, **DNR.001.2363**, .2375; **TRA.500.003.0001**, .0063 ln 40-45.
524 **TRA.500.003.0001**, .0078 ln 5-7.
525 **TRA.500.003.0001**, .0064 ln 6-10.
526 **TRA.500.003.0001**, .0064 ln 12-19.
527 **TRA.500.003.0001**, .0066 ln 4-22.
528 **TRA.500.003.0001**, .0078 ln 10-14.
529 Exhibit 61, **SUN.009.004.0037**, .0052-.0055.
530 **TRA.500.008.0001**, .0080 ln 5 to .0081 ln 41.
531 Exhibit 61, **SUN.009.004.0037**, .0057.

- *Test at normal stress of σ_{n-3} to obtain 'peak unbonded' shear strength and stop test as soon as failure is achieved*
- *Test at normal stress of σ_{n-1} to obtain 'residual' shear strength*
- *Reset the sample or reverse the direction of shearing and retest at normal stress of σ_{n-1} to obtain check on 'residual' shear strength*
- *Test at normal stress of σ_{n-2} to obtain 'residual' shear strength*
- *Reset the sample or reverse the direction of shearing and retest at normal stress of σ_{n-2} to obtain check on 'residual' shear strength*
- *Test at normal stress of σ_{n-3} to obtain 'residual' shear strength*
- *Reset the sample or reverse the direction of shearing and retest at the same normal stress as the 'peak bonded' test above to obtain check on "residual" shear strength*

- *Sample 2 from each section of core is to be tested as follows:*
  - *Test at σ_{n-2} to obtain 'peak bonded' shear strength and stop test as soon as failure is achieved*
  - *Undertake tests for 'peak unbonded' and 'residual' shear strength as listed for Sample 1*
- *Sample 3 from each section of core is to be tested as follows:*
  - *– Test at σ_{n-3} to obtain 'peak bonded' shear strength and stop test as soon as failure is achieved*
  - *– Undertake tests for 'peak unbonded' and 'residual' shear strength as listed for Sample 1.*

GHD's stability analysis

5.320 GHD's assessment, based on the available data, is that for some loading cases the Dam does not meet the acceptance criteria in the 2013 ANCOLD Guidelines for a residual strength scenario.[532]

5.321 Although Mr Willey agreed that the 2013 ANCOLD Guidelines are not intended to be a prescriptive code of practice,[533] he gave evidence that when engineers are

532 **TRA.500.003.0001**, .0062 ln 9-11, .0065 ln 28-46.
533 **TRA.500.003.0001**, .0073 ln 43-46.

assessing a dam, it is always assessed against the current standards of the day.[534] It is important to do so because methods and approaches change over time.[535]

5.322 GHD's assessment is sensitive to key input assumptions, including as to uplift and tailwater.[536] There was some difference between the assumptions that GHD adopted in its stability assessment and those used in the original design. The differences are discussed in more detail below.

5.323 The assessment was also sensitive to whether peak or residual strength parameters were used. According to the 2013 ANCOLD Guidelines, different acceptance criteria apply depending on which strength is used.

5.324 A key consideration for GHD in identifying the appropriate shear strength parameters to be used in assessing the Dam's stability was the likelihood and extent of unbonded lift joints.[537] The 2013 ANCOLD Guidelines provide that unless there is strong evidence to support bonded lift joints or investigations using cored samples are undertaken, lift joints should be considered unbonded.[538] GHD concluded that 60 - 90% of lift joints were unbonded based on a review of information about and tests results from cores taken from the Dam in 2006, 2015 and 2019.[539] The Updated Shear Strength Memorandum said that there could be '*no argument that some of the lift joints within the Dam [were] unbonded*'.[540] That led GHD to the conclusion that, according to the 2013 ANCOLD Guidelines, all the lift joints needed to be considered unbonded.[541] Therefore, the residual strength scenario was used, together with the applicable acceptance criteria.[542]

5.325 GHD did not consider it appropriate to adopt a weighted average strength to rely in part on cohesion from bedding mix placed along the upstream face. That was because of 'strain incompatibility', i.e. the uncertainty or perhaps even the unlikelihood of mobilising the full cohesion in the bonded portion of the lift and the full frictional strength over the remainder of the lift at the same shear displacement.[543]

The work of the second TRP

Role of the TRP

5.326 A second TRP was engaged by SunWater to provide comments, advice and guidance as part of the upgrade works prepared by GHD for the Dam Safety Improvement Project.[544] SunWater is not bound to accept the TRP's advice.[545] The

534 **TRA.500.003.0001**, .0019 ln 14-15; .0071 ln 43 to .0072 ln 2.
535 **TRA.500.003.0001**, .0019 ln 15-19.
536 Exhibit 16, **GHD.005.0001**, .0014.
537 Exhibit 56, **WYJ.001.003.0001**, .0009 [34].
538 **GHD.047.0001**, .0003 [9]; Exhibit 35, **ACD.001.0001**, .0026.
539 Exhibit 15, **DNR.001.2363**, .2387.
540 Exhibit 61, **SUN.009.004.0037**, .0044.
541 **GHD.047.0001**, .0003 [9]; Exhibit 35, **ACD.001.0001**, .0026.
542 Exhibit 56, **WYJ.001.003.0001**, .0017 [59].
543 Exhibit 61, **SUN.009.004.0037**, .0066.
544 Exhibit 73, **LOF.001.0001**, .0002 [6].

members of the TRP were initially, Mr Foster, Mr Lopez (no relation to the RCC Engineer of the same surname) and Jonathon Reid and Dr Pells.[546] They were later joined by Mr Tarbox and John Young.[547] The TRP members bring the following expertise:

a. Mr Foster, in dam safety and design[548]

b. Mr Lopez, in structural analysis of concrete dams[549]

c. Dr Pells, in computational fluid dynamics modelling and hydraulic studies, along with scour predictions[550]

d. Mr Reid, in dam safety and risk management perspective;[551]

e. Mr Tarbox, in RCC technology[552]

f. Mr Young is the lead geotechnical advisor on the TRP.[553]

5.327 The TRP has produced three reports following workshop meetings in May 2019, September 2019 and November 2019.[554]

5.328 The focus of the TRP's work has been on options for the improvement works being considered by GHD. However, that has involved considering the stability of the Dam in its current configuration. Relevant to that work is the shear strength of the RCC lift joints and GHD's assessment of appropriate shear strength parameters to adopt in assessing the Dam's stability.

TRP Report No. 1

5.329 Before the TRP's first meeting, SunWater engaged Mr Lopez to review GHD's stability analysis and shear strength estimation as they stood in late 2018.[555] An appendix to TRP Report No. 1 contains Mr Lopez's report. Of the stability analysis, Mr Lopez discussed changes to assumptions GHD had made about uplift, tailwater depths and crack formation (among other things).[556] In relation to GHD's estimate of shear strength, Mr Lopez said that:

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545 Exhibit 67, **PTF.001.001.0001**, .0002 [4].
546 Exhibit 11, **SUN.009.003.0613**, .0630.
547 Exhibit 12, **IGE.051.0001**; Exhibit 13, **SUN.009.002.0001**; Exhibit 67, **PTF.001.001.0001**, .0002 [5].
548 Exhibit 67, **PTF.001.001.0001**, .0003 [8].
549 Exhibit 73, **LOF.001.0001**, .0001 [1], .0002 [6].
550 Exhibit 72, **STP.001.0001**, .0007 [26].
551 Exhibit 71, **JTR.001.001.0001**, .0002 [8].
552 Exhibit 100, **TAG.001.0001**, .0002 [5].
553 Exhibit 76, **YOJ.001.001.0001**, .0002 [6].
554 Exhibit 67, **PTF.001.001.0001**, .0002 [4]-[5], .0003 [6].
555 Exhibit 11, **SUN.009.003.0613**, .0620.
556 Exhibit 11, **SUN.009.003.0613**, .0659.

a. the design shear strength parameters appeared conservative based on a comparison with data from a number of RCC dams in Brazil, the United States and Vietnam;[557]

b. as there was little difference between peak and residual shear strength data, and because testing had been conducted on only four specific locations from the Dam, more stringent acceptance criteria for the 'peak strength – not well defined' scenario should be considered in lieu of the 'residual – well defined' criteria;[558]

c. a shear strength of 37º and no cohesion for normal stresses up to 600 kPa based on an assumption that the lift joints were unbonded appeared reasonable until more testing was conducted.[559]

5.330 Mr Lopez's report and TRP Report No. 1 were prepared before any testing results from the cores taken in 2019 were available. Indeed, when the TRP visited the site on 28 May 2019, a drill rig was on site drilling and coring from the right abutment.[560] Based on the shear strength data that was available to GHD at that time, the TRP stated in Report No. 1 that:[561]

> *Testing of core from two holes drilled from the dam crest on the left and right abutment indicate that the lift surfaces have strengths below those assumed in the original design with a potential to have a 20th percentile strength or 97.5% confidence level strength in the range of 37 to 38˚, which is less than typical values for unbonded joints in mass concrete gravity dams. Further sub-horizon[t]al holes have been drilled from the spillway toe to intersect lift joints. [T]he drilled horizontal cores have shown that the lift joints have both bonded and unbonded contacts, which makes the estimation of the shear strength quite challenging, and the strength estimated by GHD might be a bit too conservative. On the other hand, the TRP noted that the estimation of the shear strength was based on a linear regression after plotting all available shear tests together, even though the tests did not follow all the same testing standards and were sourced from both vertically and horizontally drilled cores. GHD acknowledged this, and said that they took some conservatism in the estimation, which would cover for such uncertainty in the quality of the test results. It is suggested that future shear strength testing (i.e. current investigation) be conducted using the same standards, and that the shear strength used for design be adjusted accordingly.*

5.331 The TRP noted that GHD's stability analysis had adopted the minimum factors of safety for '*residual-well defined parameters*' from the 2013 ANCOLD Guidelines.[562] In line with Mr Lopez's views above, the TRP recommended that GHD consider the

557 Exhibit 11, **SUN.009.003.0613**, .0662.
558 Exhibit 11, **SUN.009.003.0613**, .0660, .0664.
559 Exhibit 11, **SUN.009.003.0613**, .0664.
560 Exhibit 11, **SUN.009.003.0613**, .0634.
561 Exhibit 11, **SUN.009.003.0613**, .0621.
562 Exhibit 11, **SUN.009.003.0613**, .0623.

appropriateness of that scenario when the test results were from localised samples and when there was little difference between the peak and residual shear strength results to date.[563]

TRP Report No. 2

5.332 The second TRP report was dated 23 September 2019.[564] In relation to RCC lift joint shear strength, the TRP discussed the original design of the Dam and the specified requirements for placing bedding mix on the upstream face and on cold joints. Dr Schrader's memorandum of 2 August 2004 relaxing the bedding mix requirements on cold joints was included in full in TRP Report No. 2.[565] In relation to Dr Schrader's memo, the report said that:[566]

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

5.333 The second report set out the conclusions that GHD had drawn from the results of shear strength testing in 2015 and 2019:[567]

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

5.334 The TRP supported GHD's recommendation to use the residual shear strengths to assess the Dam's stability.[568] The TRP also supported GHD's updated estimation of shear strength parameters:[569]

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

563 Exhibit 11, **SUN.009.003.0613**, .0623.
564 Exhibit 12, **IGE.051.0001**, .0002.
565 Exhibit 12, **IGE.051.0001**, .0010.
566 Exhibit 12, **IGE.051.0001**, .0011.
567 Exhibit 12, **IGE.051.0001**, .0011.
568 Exhibit 12, **IGE.051.0001**, .0012.
569 Exhibit 12, **IGE.051.0001**, .0012.

5.335 Having expressed conclusions about the shear strength of the RCC lift joints, section 3.2 of TRP Report No. 2 went on to provide an explanation for the 'poor condition' of the Dam. The section became significant to the Commission's work and so is set out in full below:[570]

> *From construction records, specifications and recent communications with the original dam designers, it appears that the poor condition of the dam is not the result of using low paste RCC per se, but rather the result of poor construction practices for the treatment of the numerous cold joints in the dam, which were not rectified in spite of evidence that poor bonding and segregation were identified on the early RCC lift joints.*
>
> *A review of construction photographs made available to the TRP during meeting #2 suggests that the organization of equipment, personnel and RCC placing operations had the potential of contributing to the poor lift surface bond strengths. An orderly array of equipment operating at a single front for delivery of RCC, spreading by dozer and immediate compacting is the preferred organization on the active lift surface. Placing and dozing away from forms and abutments versus spreading and dozing toward forms and abutments should be avoided. There was evidence that multiple layers had been discontinued at several different locations which created multiple shoulders susceptible to poor compaction and the potential for "rock pockets" (uncompacted RCC) to be covered over and not properly prepared to receive the next lift. There were obvious large expanses of dried out surfaces with little or often no water curing visible. There were also areas where the old lift surfaces were being contaminated by traffic, dust and debris. Whether or not these conditions were mitigated before new RCC was placed on those surfaces cannot be ascertained from photos but suggests the possibility of those areas not having been properly prepared.*
>
> *It has been difficult to find evidence of bedding mix on cold joints in recent and previous drilled RCC cores. Additionally, horizontally drilled cores extracted in the plane of horizontal lifts observed during a laboratory visit on 27 August 2019 exhibited dust and/or mud on the RCC joints suggesting little or insufficient surface preparation. Based on the observed condition of recovered horizontal lift surfaces and taking into consideration that the specifications did not require thorough cleaning of previously placed RCC lift surfaces that required bedding mix, it is questionable whether or not the application of bedding mix would have improved the bond between RCC layers.*

5.336 The reference to communications with the Dam designers in the passage above was a matter of interest during Commission hearings. Section 3.2 was drafted by Mr Tarbox,[571] although it was included in a report signed by all members of the TRP. Mr Tarbox did not have discussions with the original dam designers.[572] However, Mr

570 Exhibit 12, **IGE.051.0001**, .0013.
571 **TRA.500.004.0001**, .0050 ln 20-23.
572 **TRA.500.007.0001**, .0048 ln 11-22.

Foster did have a discussion with Mr Herweynen over the telephone before the second TRP workshop. After that call, Mr Herweynen sent the TRP members a number of documents including part of the Detail Design Report and some RCC QC Reports.[573] The reference in section 3.2 of TRP Report No. 2 is a reference to Mr Foster's discussion with Mr Herweynen.

5.337 In terms of the Dam's stability assessment, using GHD's analysis, the TRP noted the sliding stability deficiencies of the Dam:[574]

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

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

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

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

5.338 The TRP also reviewed a further stability memorandum from GHD regarding the stability analysis for monoliths C, N and H, noting that the each of those monoliths did not meet ANCOLD acceptance criteria in certain circumstances.[575]

5.339 TRP Report No. 2 concluded that the Dam *'in its present state has a risk profile that plots well above the ANCOLD tolerable risk criteria and does not meet industry standards for gravity dam stability'.* [576]

TRP Report No. 3

5.340 TRP Report No. 3, dated 9 December 2019, was prepared after a workshop on 19 November 2019.[577] Mr Tatro attended the workshop after being commissioned (along with his colleague, Mr James Hinds) to review GHD's assessment of lift joint shear strength and implications for shearing failure through a lift joint.[578] Subsequent to that workshop, on 25 November 2019 Mr Tatro and Mr Hinds finalised a report commenting on GHD's evaluations (**Tatro Hinds Report**).[579]

5.341 The TRP did not make extensive comment on the Tatro Hinds Report but highlighted some noteworthy points. Having summarised the multi-stage testing regime set out in detail above, Report No. 2 said:[580]

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573 Exhibit 67, **PTF.001.001.0001**, .0015 [42].
574 Exhibit 12, **IGE.051.0001**, .0014-.0015.
575 Exhibit 12, **IGE.051.0001**, .0015.
576 Exhibit 12, **IGE.051.0001**, .0021.
577 Exhibit 13, **SUN.009.002.0001**, .0002.
578 Exhibit 13, **SUN.009.002.0001**, .0003.
579 Exhibit 66, **IGE.028.0001**.
580 Exhibit 13, **SUN.009.002.0001**, .0003-.0004.

Total shearing can be in the order of 30 to 40 mm at the end of this testing regime and [Tatro Hinds] has expressed an opinion that this testing is very aggressive and, in their opinion, adversely affected the test results, creating lower values than if the testing were done by a different methods. They considered this repetitive testing to be questionable. If the testing is to represent residual strength once a dam block beings to slide on an unbonded lift it will require large displacements to define residual strength. The TRP opinion is that "large displacements" is a loose term, possibly a shortcoming of the [2013 ANCOLD Guidelines], which is leading to some contradicting interpretation by TH and GHD. Ideally each test should have a peak and residual test regime completed for a single normal stress. CDA Dam Safety Guidelines (2005) recommend for shear strength tests that horizontal cores (minimum 150 mm diameter) be taken from the joints and samples are prepared for direct shear tests under the prevailing normal stresses. It appears from the test results that if only the first residual test results were used and the 2nd and 3rd stages were not considered there would only be results available for a normal stress of 200 kPa.

5.342 The Tatro Hinds Report expressed concern about the limited number of samples. The TRP observed that while more sampling was desirable, *'on the basis of test results to date it [was] difficult to see that strengths will increase such that conclusions on stability and risk reached by GHD will change markedly'.*[581]

5.343 The third TRP report confirms the conclusions stated in the earlier Report No. 2 and expresses the view that the Dam has safety concerns if left in its current condition.[582] The report concluded that the Dam's risk profile was above the ANCOLD tolerable risk criteria and did not meet industry standards for gravity dam stability.[583] According to the TRP, the Dam was susceptible to a shear sliding failure mode on the unbonded lift joints within the RCC.[584]

Concerns about the reliability of the 2019 test results

5.344 Of the four RCC experts who gave evidence concurrently, Dr Schrader,[585] Dr Rizzo,[586] and Mr Tatro[587] did not think that the information available to GHD was sufficiently reliable to found the stability analysis. The concerns raised by those three experts included:

a. about the testing methodology, including appropriateness of multi-stage testing

b. that the samples were not representative of the RCC in the Dam, and there were not enough samples

581 Exhibit 13, **SUN.009.002.0001**, .0004.
582 Exhibit 67, **PTF.001.001.0001**, .0003 [7].
583 Exhibit 13, **SUN.009.002.0001**, .0016; Exhibit 67, **PTF.001.001.0001**, .0003 [9].
584 Exhibit 13, **SUN.009.002.0001**, .0016.
585 **TRA.500.008.0001**, .0066 ln 14-15.
586 **TRA.500.008.0001**, .0066 ln 34-38.
587 **TRA.500.008.0001**, .0067 ln 10-13.

c. about the method of sampling, namely whether blocks should have been taken rather than cores.

Multi-stage testing

5.345 In the Tatro Hinds Report, Mr Tatro and Mr Hinds expressed the following reservations about multi-stage testing:[588]

> *It is our opinion the method of repetitive testing for unbonded peak and residual strengths is problematic. This method appears to degrade the sample surface and may negatively affects the sliding friction strength and residual shear strength test results. It is possible this method is more appropriate for hard rock of uniform consistency and not for a concrete composite of high strength rock within a low strength matrix. Testing of residual strength of individual samples will provide a reasonable comparison to evaluate the effect of repeated shearing.*
>
> …
>
> *The photographs of test samples after test show shear surfaces that are obviously highly degraded due to repetitive shearing and appear to be more representative of a crushing and grinding failure as opposed to a shear failure.*
>
> *The extreme shear surface distress seems to have occurred on all test samples thus strongly suggesting that repetitive loading as specified in the test method is not appropriate. Our conclusion is that the test method is overly aggressive in the application of load and repeated re-application and is not suitable for this low strength matrix and RCC in general.*

5.346 Mr Tatro told the Commission that:[589]

> *I take issue with any test result that was done in a multiple - I forget the term Tim used in his report, but in many passes on the same sample. I take issue with that. I don't think those are reliable.*

5.347 In a letter dated 13 February 2020, Dr Rizzo said that repetitive testing was not appropriate[590] and agreed with the criticisms of the method in the Tatro Hinds Report.[591] Dr Schrader said multistage testing was not appropriate for LCRCC and that is why other equipment and testing methods had evolved.[592]

5.348 Mr Dolen did not agree with the three other experts:[593]

> *I am not aware in the past that there have been criticisms of these procedures, and that goes back to the beginning of doing the shear box testing in the mid to*

588 Exhibit 66, **IGE.028.0001**, .0007-.0008.
589 **TRA.500.008.0001**, .0018 ln 24-27.
590 Exhibit 103, **RIZ.001.001.0001**, .0003.
591 **TRA.500.008.0001**, .0043 ln 13-17.
592 **TRA.500.008.0001**, .0043 ln 41 to .0044 ln 18.
593 **TRA.500.008.0001**, .0043 ln 23-28.

late 1970s. This has been a common practice used on our concrete gravity dams. It has been done and reported to other agencies, these results of multistage testing.

5.349 Mr Dolen considered that degradation of the samples subjected to testing was reflective of quality rather than the method of testing. Where only half of a surface showed substantial degradation, Mr Dolen interpreted that as primarily attributable to the quality of the RCC at that surface.[594] On that point, Mr Tatro agreed that a segregated sample of RCC would degrade 'pretty quickly'. However, Mr Tatro said that he had never seen samples come out of a mould in such a poor condition as these samples. That struck him as a red flag that the testing procedure may not be appropriate.[595] A photograph of a sample before and after shear strength testing is included below:

Figure 5.25 – Photograph of an RCC sample before and after testing.
*(Exhibit 61, **SUN.009.004.0037**, .0234)*

5.350 Mr Dolen's view was that the degradation of samples reflected that which would occur under displacement in the Dam.[596] Mr Foster said that the displacement to which samples were subjected in testing was representative of what would happen '*if you need large shearing to fail the dam, because you will be degrading the surfaces as it goes along with time, and that strength is what you're going to end up with*'.[597]

594 **TRA.500.008.0001**, .0047 ln 29-36.
595 **TRA.500.008.0001**, .0048 ln 28-31.
596 **TRA.500.008.0001**, .0058 ln 5-6.
597 **TRA.500.004.0001**, .0051 ln 18-26.

5.351 In a report prepared for the Commission, Mr Dolen said that multi-stage testing was an accepted standard used around the world to test for shear strength and friction properties of cores drilled from RCC and CVC dams. In support of that view, Mr Dolen's report said:[598]

> *McLean and Pierce discussed the appropriateness of multistage testing in their paper Comparison of Joint Shear Strengths for Conventional and Roller Compacted Concrete, (McLean and Pierce, 1988). They quote as follows:*
>
> > *'Of particular interest in the testing of jointed materials, either concrete or rock, is the condition of the joint at the initiation of the test. The impracticality of economically obtaining and testing sufficient specimens to allow each data point to be determined from a "virgin" specimen usually dictates that multi-stage tests be performed. Under these conditions, the test is started under a selected normal force and carried out to the point of interest, i.e., breakbond, or peak strength, then stopped. The normal load is then increased, and the test restarted. Several of these stages can be performed until the condition of the sheared surface, displacement of the specimen, or limits of testing device cause termination of the test series'.*
>
> *Multi-stage testing is the accepted practice for determining shear sliding strength parameters on concrete and rock joints. This test method has been used by the USBR and for testing done by USBR for USACE; including tests requested by and reported to Mr. Tatro (Gleason and Dolen, 2003).*
>
> *Multi-stage testing is a part of the both the ASTM D5607-16 (Section 8.6.2.4 Multi-Stage Shear) and the USBR Standard 4915 (Sections 10.10 to 10.13); both standards follow the same procedures. The results of these tests have been accepted in dam design and stability analysis for at least forty years.*
>
> *I have performed more than twenty shear testing programs of laboratory cast cylinders, test sections, and from cores drilled from RCC dam construction.*
>
> *I have performed additional test programs on laboratory cast specimens and cores drilled from conventional concrete dams and analyzed the tests on every shear test by USBR up to 2011; more than 400 individual multi-stage tests.*

5.352 While Mr Dolen could point to two industry standards that provided for multi-stage testing, the other three experts did not identify an alternative testing procedure that had been adopted by any agency as a standard. Mr Tatro gave evidence about a bespoke standard that he had developed with Mr Hinds for a project that Dr Rizzo worked on. Dr Rizzo said that procedure had worked quite well. Dr Schrader referred to procedures that he had developed for clients on request.

598 Exhibit 104, **GHD.006.0001**, .0014 to .0015.

5.353 ASTM D5607 – 16 is in evidence. Section 8.6.2.4 of it provided:[599]

> *Multi-Stage Shear—Repeat [the procedures headed 'Normal Load' and 'Shear Load'] on the same specimen under several normal loads. Two possible techniques are available for performing the multi-stage shear: without repositioning the specimen to its natural position between normal loads before each shearing stage, or with repositioning the specimen to its natural position between normal loads before each shearing stage. Typically, at least three to five different normal loads are required to define the strength envelope. 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.*

5.354 This passage shows that the standard recognises the potential for repetitive shearing to affect the results of the latter stage tests. Despite that potential, the procedure is endorsed as an industry standard and includes recommendations on how to conduct the testing so as to reduce the potential for specimen degradation. Mr Willey's evidence was that multi-stage shear testing was undertaken in accordance with that standard.[600]

5.355 Mr Willey's statement referred to another industry standard that recognises multi-stage testing.[601] The Electric Power Research Institute entitled '*Uplift Pressures, Shear Strengths, and Tensile Strengths for Stability Analysis of Concrete Gravity Dams*' (**EPRI 1992**)[602] is the source of recommended shear strengths in the 2013 ANCOLD Guidelines.[603] EPRI 1992 says the following of multi-stage testing:[604]

> *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.*

5.356 In any event, even if the residual strength obtained on the first stage test was used, Mr Willey said that the stability assessment would still fall short of the ANCOLD acceptance criteria.[605]

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599 Exhibit 98, **WLJ.001.001.0001**, .0006 to .0007.
600 Exhibit 56, **WYJ.001.003.0001**, .0016 [56].
601 Exhibit 56, **WYJ.001.003.0001**, .0018-.0019 [68].
602 WYJ.001.004.0001.
603 Exhibit 35, **ACD.001.0001**, .0026.
604 **WYJ.001.004.0001**, .0003-.0004.
605 Exhibit 56, **WYJ.001.003.0001**, .0021 [80].

Cores or large blocks?

5.357 The Tatro Hinds Report said that:[606]

> *[C]ores, compared to large blocks, offer a small area to test where edge effects and aggregate size can dominate the observed performance. Hence larger core are better than smaller core. Our shear testing is done nearly exclusively on sawn blocks (nominal shear surface ranging in size from 250mm x 250mm to 300mm x 300mm) in order to achieve a more consistent and representative sample. Although this is not convenient for post construction evaluations, it should be noted that larger samples are more representative and can be more effective in defining shear parameters for lower strength and less well-bonded samples.*

5.358 In a letter dated 13 February 2020, Dr Rizzo agreed that the best type of test is a shear test on sawn blocks.[607] While giving evidence, Dr Rizzo said that the results based on cores were not informative at all because the data was flawed by the sampling technique.[608]

5.359 The Tatro Hinds Report had stated that block samples were not convenient post-construction.[609] Dr Rizzo was asked whether it was impractical to take block samples now and gave evidence that it was 'very practical' to cut blocks from the secondary spillway with a diamond saw.[610]

Representative nature of samples

5.360 Mr Dolen was of the view that the sample size was sufficient. A total of 7 samples in 2015 plus 17 samples in 2019 was comparable to USBR RCC core testing programs. It was also comparable to a recent evaluation that Mr Dolen had participated in to investigate the lift joint shear friction properties for a mass concrete dam in Canada. Mr Dolen described that as the '*most comprehensive testing program*' that he had ever participated in.[611]

5.361 Mr Dolen pointed to the balance between further testing and delaying mitigation of risks to the downstream population.[612] He referred to the Tatro Hinds Report, which stated:[613]

> *[A]dditional testing as recommended may narrow the range of variability and improve assurance in the test results.*

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606 Exhibit 66, **IGE.028.0001**, .0006-.0007.
607 Exhibit 103, **RIZ.001.001.0001**, .0003.
608 **TRA.500.008.0001**, .0051 ln 6-14.
609 Exhibit 66, **IGE.028.0001**, .0006-.0008.
610 **TRA.500.008.0001**, .0071 ln 24-26.
611 Exhibit 104, **GHD.006.0001**, .0020.
612 **TRA.500.008.0001**, .0064 ln 31-35, .0071 ln 41-44.
613 Exhibit 66, **IGE.028.0001**, .0002.

5.362 Mr Dolen agreed with that and used the available shear strength data to illustrate the point.[614] As the Updated Shear Strength Memorandum explains, using data from the 2015 tests, the friction angle was 37-38º.[615] When the first set of 2019 test results were included, the friction angle went up to 39.3º.[616] Including the second set of 2019 test results brought the angle back down to 38.2º.[617]

5.363 Mr Foster's evidence was that a sufficient number of samples had been taken to provide reasonable certainty in residual shears strengths. The key thing, as far as Mr Foster was concerned, was that GHD retrieved larger horizontal cores in 2019 to validate the testing on the smaller vertical cores in 2015. Mr Foster said that was important because:[618]

> *… it removes a lot of confusion or doubt or suspicion that the breaks in the core in the vertical one were caused just by the drilling. The samples that were taken in the abutments where you could actually see where the joint was, and they drilled subhorizontally knowing that they would capture the joint in the sample, you could see that some of them were bonded and some of them were unbonded. They tested both lots, went through and got residual from all of them, and it has given a lot more confidence to the numbers that have come out of them – the 38-39 degree range for residual strength.*

5.364 Mr Tatro's evidence was that there was not a sufficient number of samples tested to date.[619] As to the location of the cores, Mr Tatro said that:[620]

> *As I hear more and more testimony and read more and more of this stuff you send me, I question how representative this coring is of the material in the dam. So, we can talk forever about this, when, instead, maybe we should just make a conscious effort to go out and representatively core that structure and know for sure. Otherwise, we're just speculating.*

5.365 Dr Rizzo believes that more shear tests should be conducted. The small number conducted to this point '*does not accurately quantify actual joints conditions*'.[621]

5.366 Dr Schrader thinks that the testing to date does not represent the real situation in the Dam. More samples are needed, including samples through the areas where bedding mix was supposed to have been used.[622]

5.367 The three experts who were unsatisfied with the testing to date were asked what more should be done to provide reasonable certainty.

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614 **TRA.500.008.0001**, .0064 ln 37 to .0065 ln 12.
615 Exhibit 56, **WYJ.001.003.0001**, .0014 [47].
616 Exhibit 16, **GHD.005.0001**, .0005, .0013.
617 Exhibit 61, **SUN.009.004.0037**, .0066.
618 **TRA.500.004.0001**, .0047 ln 32 to .0048 ln 1.
619 **TRA.500.008.0001**, .0064 ln 7-17.
620 **TRA.500.008.0001**, .0049 ln 7-14.
621 Exhibit 103, **RIZ.001.001.0001**, .0003.
622 **TRA.500.008.0001**, .0058 ln 40-46.

5.368 Mr Tatro said:[623]

> *I think we need to do vertical coring, 150mm coring at some angle, and establish the degree of intact versus not intact joints once and for all, and do that fairly quickly, and save those cores. ...*
>
> *And then we have all those cores that have been taken in an appropriate location and not been damaged by drilling, and then do a large number of shear box values for friction angle, not multiple stage but single stage, and do a series of those. And then if there's still concern or question or anomalies in that data, take some larger-diameter cores, use a larger shear box. Then as a last resort, try and get some block samples out, if the larger core doesn't show anything.*

5.369 Dr Rizzo said:[624]

> *[W]e should do some large block testing, 300mm by 300mm, if you will, as the primary basis for determining the friction parameters at the lift joints. They should be supplemented by additional cores to allow for a reasonably good statistical analysis of the variability and uncertainty associated with the results of the shear block tests.*

5.370 Dr Schrader suggested a number of further tests including shear blocks, preparing shear pads on site with buried aggregate from construction and testing that, testing the parent RCC in shear, and digging a trench in the Dam wall to inspect the in situ material.[625]

Magnitude of the results

5.371 Dr Schrader called the shear test results into question because (among other reasons) the friction angles were so low. He said:[626]

> *[T]o my knowledge, no RCC or conventional concrete, with this quality of aggregate, with this type of mix, with similar properties, has ever had a result that low. In fact, many are much, much higher than 45 degrees.*

5.372 As to the relative magnitude of these results, Mr Willey's evidence was that the results were not unreasonable by comparison to the EPRI data set out in the 2013 ANCOLD Guidelines.[627] Section 5.1 of the Guidelines referred to EPRI (1992) having reported that the lower bound of sliding friction strength was '*c' = 0.0 MPa, Φ' = 38º*' but that '*the test data indicate that the **residual shear strength** parameter Φ' is generally 2º to 3º less than the **sliding friction strength**'.*[628]

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623 **TRA.500.008.0001**, .0070 ln 9-25.
624 **TRA.500.008.0001**, .0070 ln 41-47.
625 **TRA.500.008.0001**, .0072, ln 19 to .0074 ln 3.
626 **TRA.500.008.0001**, .0021, ln 2-6.
627 **GHD.047.0001**, .0003 [9]; Exhibit 35, **ACD.001.0001**, .0026.
628 Exhibit 35, **ACD.001.0001**, .0026 (emphasis in original).

5.373 Mr Willey also referred[629] to Dr Schrader's 1999 paper, which speaks of a wide range of RCC joint strengths existing, with extremes for friction angles ranging from 30º to 65º.[630]

5.374 Mr Dolen's opinion was that the results were low compared to other data because these results were representative of a quality of RCC that is not often tested. While Mr Dolen said it was unusual to get such low test values, it is similarly uncommon to have '*this issue of porosity leading to the low values*'.[631]

Conclusions about the reliability of the 2019 test results

5.375 Multi-stage testing is a recognised method that is endorsed in industry standards. The potential for specimen degradation in subsequent stages of the testing is recognised in ASTM D5607 and measures are recommended to reduce that potential. Of the three RCC experts who criticised multi-stage testing, one alternative suggested by Mr Tatro was to obtain residual shear strength results by subjecting specimens to stage one testing only.[632] However, many more samples would be needed to achieve statistically robust results based on individual tests, which then becomes a matter of expense. Otherwise, the only alternatives were bespoke procedures that had been developed by two of the experts themselves. Those procedures have not been adopted by an industry as standard. Multi-staged testing can produce reliable values of residual friction angle, provided the shearing is always in the one direction and that no other disturbance of the specimen occurs between stages.

5.376 The friable nature of the LCRCC with which the Dam was constructed was susceptible to being degraded in testing procedures, especially those that involved multi-stage testing. However, the evidence from Mr Dolen and Mr Foster was that such degradation was reflective of the degradation that would be sustained across lift joints if the Dam were to fail. That proposition was rejected by Dr Schrader, Dr Rizzo and Mr Tatro. Mr Tatro said that the tests were carried out well beyond the limit of what would constitute 'failure'.[633] However, that did not directly respond to the suggestion that the displacement in the multi-stage testing simulated '*what would actually occur in the event the Dam were to fail in sliding*'.[634] Neither Dr Schrader nor Dr Rizzo took the opportunity to elaborate on why they rejected the proposition.[635]

5.377 The opinions of Mr Dolen and Mr Foster are preferred. They were able to articulate the basis of their views and it makes sense that a lift surface would be degraded during the large displacements that would occur if the Dam failed in shear across the lift joint during a flood event. The frictional resistance that lift joints have at large displacements is relevant to the question whether this Dam is stable.

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629 **TRA.500.008.0001**, .0023 ln 9-40.
630 Exhibit 124, **PDI.040.0001**, .0004.
631 **TRA.500.008.0001**, .0024 ln 4-7.
632 **TRA.500.008.0001**, .0047 ln 43 - .0048, ln 18.
633 **TRA.500.008.0001**, .0058 ln 10-13.
634 **TRA.500.008.0001**, .0057 ln 42-45.
635 **TRA.500.008.0001**, .0057 ln 42-47, .0058 ln 17.

5.378 It is recognised that the lift joints in the Dam have almost certainly not undergone any significant shear displacement to this point. Therefore, the residual strengths do not reflect the current state of the Dam. But the current condition of the Dam is not what the residual strength scenario is directed to. As Mr Foster explained:[636]

> *The rationale of the residual strength is that if the bond is not there for whatever reason, you want to be assured that your dam is not going to fail. So the last resort that you have in the strength profile of the test is that if you have broken through the peak and you are heading to a residual strength, your dam is still stable.*
>
> *… So if all the other systems have broken down, the bond has gone, et cetera, you still end up in a situation where you basically can't fail the dam by sliding, because … the residual strength is not going to be exceeded; you have some reserve on that.*

5.379 Some sections of the lift joints will have at least a peak unbonded strength that will need to be overcome before a shearing failure occurred. However, GHD's approach to stability, based on residual strengths, was supported by the TRP,[637] by Mr Dolen[638] and by Mr Tatro[639] based on the observed and anticipated extent of unbonded lift joints. Mr Tatro had accepted the appropriateness of GHD's approach in the Tatro Hinds Report, but seemed to retreat from that position during the hearings, saying that some sections of his report were remnants of an earlier draft. No explanation of what sections had been updated and what were remnant was given, aside from the following sentence being from an earlier draft and 'misleading' as a result:[640]

> *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.*

5.380 In circumstances where it is costly and time consuming to take core samples, multi-stage testing provides more strength data out of the samples retrieved. The samples that were tested as the first stage provide similar indications of residual shear strength as the samples that were tested in the second and third stages at the same normal stress.[641] That may be why the Tatro Hinds Report remarked that the identified issues with testing were not expected to significantly impact on the test results.[642] The reservations about multi-stage testing do not explain the low shear strength results.

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636 **TRA.500.004.0001**, .0061 ln 38 to .0062 ln 9.
637 Exhibit 12, **IGE.051.0001**, .0012.
638 Exhibit 104, **GHD.006.0001**, .0005.
639 Exhibit 66, **IGE.028.0001**, .0002.
640 **TRA.500.008.0001**, .0017 ln 21-25, with the quoted passage from Exhibit 66, **IGE.028.0001**, .0002.
641 Exhibit 56, **WYJ.001.003.0001**, .0021 [79].
642 Exhibit 66, **IGE.028.0001**, .0002.

5.381 The criticisms levelled on the basis of the low test results are not compelling. As Mr Willey pointed out, the results are still within the bounds (albeit at the lower end) of ranges of friction angles published by ANCOLD.[643]

5.382 Whether sufficient testing has been done is a different matter.

5.383 Mr Dolen was conscious that more sampling and testing might delay work to mitigate risks to the downstream population. However, he did not categorically reject the suggestion that more testing should be carried out. The three other RCC experts and Mr Willey all supported further testing. Dr Rizzo said that this was a '*very difficult situation*' that required best efforts to assess the available strength of lift joints.[644]

5.384 LCRCC is more difficult to sample and test than RCC with higher cementitious content. That disadvantage has materially contributed to the lingering uncertainty whether the Dam achieved the design shear strength parameters. Securing reliable test results from cores of LCRCC is inherently difficult, including because of the low strength of LCRCC, which makes it susceptible to damage during drilling, sampling and testing especially if affected by porosity.

5.385 Given those difficulties with coring LCRCC, three RCC experts would test for the achieved shear strength of LCRCC using large block samples. It is not clear whether this can be done at the Dam. Dr Schrader gave evidence explaining how a shear block adjacent to the upstream face might be extracted;[645] however, he accepted that difficulties with that approach were presented by the anchors for the precast concrete panels that are embedded in the upstream side of RCC.[646]

5.386 GHD's Updated Shear Strength Memorandum stated:[647]

> *It is recommended that additional samples be obtained for laboratory testing to provide greater certainty in relation to the lift joint shear strength. Sampling should be undertaken as part of the Essential Works lowering of the primary spillway crest which is currently proposed by Sunwater. This will allow the removal of cut block samples to be obtained from within the section of the crest that is being demolished. The extent of sampling and the testing methodology for these samples will need to be agreed with the Technical Review Panel for the project as well as other relevant independent reviewers.*

5.387 That accords with SunWater's intentions with respect to further testing:[648]

> *Sunwater will perform additional testing as part of the essential works and intends to consult with certain of the technical experts to develop the sampling and testing program. This testing will necessarily be planned and managed so as*

643 **GHD.047.0001**, .0003 [9]; Exhibit 35, **ACD.001.0001**, .0026.
644 **TRA.500.008.0001**, .0051 ln 12-14.
645 **TRA.500.008.0001**, .0082 ln 5-17.
646 **TRA.500.008.0001**, .0082 ln 21-36.
647 Exhibit 61, **SUN.009.004.0037**, .0066-.0067.
648 **SUN.026.0001**, .0019.

to minimise delay to the essential works. The results of the additional testing will inform future decisions about the scope of the remaining Paradise Dam Improvement Project works and assist with designing improvement options.

Sunwater submits that it has made the decision to proceed with the essential works because of the risk that the Dam poses to the safety of the downstream population and based on the best information which it has. This includes relying on the advice of retained experts. That is a complex and difficult decision making process. It is one which necessarily involves time sensitive decisions.

5.388 In circumstances where questions remain about the Dam's stability, further testing is warranted.[649]

Sliding stability analyses of the Dam

Choice of shear strength parameters for design

5.389 The selection of appropriate values of cohesion and friction angle is critical to obtaining an accurate and reliable estimate of the strength of a potential shear plane and therefore of the stability of a dam. Careful consideration must be given to the choice of these parameters by dam designers. Often these choices are made based upon prior experience, but prudence would dictate validating these choices by special shear testing in a laboratory on specimens containing a lift joint, or a concrete-rock interface, preferably on samples extracted from the dam itself, either from drill cores or block samples.

5.390 It is also possible when analysing the sliding stability of a gravity dam to conduct a probabilistic analysis of the likelihood of sliding failure. This type of analysis is able to take into consideration the uncertainty in the values of the strength parameters that should be used in the analysis and the loadings that may be applied to the dam. There is no evidence that a probabilistic approach to structural stability was adopted at the time the Dam was designed. On the contrary, the Factor of Safety approach was adopted, as evidenced by the Final Design Report. Further details of this approach to assessing the stability of the Dam at the design stage, and much later post-construction, are now considered in the following sections.

Factor of Safety analyses

5.391 Sliding stability analyses of the Dam were conducted by the Alliance during the design phase of the Dam and they documented the outcome of their analyses in the Detail Design Report.[650]. Subsequently, GHD conducted an independent sliding stability assessment and documented the outcome in the Stability Memorandum in November 2019.[651]

649 When that testing should occur relative to the lowering of the primary spillway crest is not an issue within the Terms of Reference. That is for others.

650 Exhibit 24, **GHD.002.0001**, .0137 - .0166

651 Exhibit 16, **GHD.005.0001**.

5.392 There are significant differences between these two independent sets of stability analyses. Some of these differences were explored during the hearing and they are identified and discussed specifically in documents supplied to the Commission by GHD and Hydro Tasmania.[652] Essentially, they concern disagreements between these parties about what values should be selected for the strength parameters that ought be assigned to the RCC lift joints in the dam and the assumptions that should be made when conducting deterministic calculations of the factor of safety with respect to sliding of the dam along either its interface with the rock mass upon which it sits or along lift joints within the dam monoliths. These differences are explored further in the following sections, but the stated view of Hydro Tasmania is:[653]

> *The low factors of safety that GHD are calculating are due to the overly conservative parameters they have adopted for tensile strength, shear strength and uplift. The combination of these individual conservatisms is considered to be unreasonable.*

Alliance sliding stability analysis – pre-construction

5.393 In their analysis of sliding stability carried out as an essential component of their design activities, the Dam's designers adopted a number of sets of values of the lift joint strength parameters and made a number of other key assumptions. These warrant further discussion.

5.394 The shear strength properties of the lift joints have been considered in detail elsewhere in this report. At this juncture, suffice to say that the values adopted by the Alliance were much less conservative than the shear strength properties measured by GHD and consequently used in the various sliding stability analyses conducted by GHD. This difference is discussed in more detail later, but it is important to note at this point that the Alliance conducted no strength testing to validate the particular strength assumptions that they made at the design stage.

5.395 One of the key design decisions involved the use of an impermeable membrane attached to the upstream face of the Dam. In addition to the obvious implications for water-tightness, this choice had a number of other implications for the behaviour and stability analysis of the Dam, as will be described below. Section 5 of the Detail Design Report describes how and why the decision was taken to install a membrane on the vertical upstream face of the dam and some of its consequences:[654]

> *Following discussions with the Dam Safety Regulator, a decision was made early in the Stage 2 process to adopt a dam design which incorporates an upstream membrane to ensure the dam is watertight. This decision eliminated the need for grout enriched RCC on the upstream face and allowed the RCC mix design to be based on strength requirements only. The compressive strength required in the RCC was only 2MPa, based on the stability analysis*

652 **HYT.007.0001**, .0016.
653 **HYT.007.0007**, .0007.
654 Exhibit 24, **GHD.002.0001**, .0129.

carried out (see Section 5.5). This strength could be achieved with a lower cementious material than proposed in the preliminary design. The RCC mix design is discussed in more detail in Section 6 of the Design Report.

5.396 The selected membrane was installed between the RCC and concrete facing panels, the latter providing some protection of the membrane from sunlight and mechanical damage. The Detail Design Report also noted the following:[655]

> *Located immediately behind* [downstream of] *the membrane is a drainage system that can collect any seepage should it occur. Drainage pipes collect this seepage and convey it to the downstream face. The design includes a drainage seepage monitoring system so that seepage can be assessed and therefore the integrity of the membrane can be monitored.* ***The dam has been designed to remain safe even if the membrane and drainage system fails*** *(refer to Section 5.5).*

5.397 The significance of the emphasised last sentence is explored in more detail later in this report.

5.398 Section 5.5 of the Design Report includes the following statements:[656]

> *Internal pore* [water] *pressures will act within the rock foundations, at the foundation-dam interface, and within the body of the dam. When acting on the dam-foundation interface or of any section being analysed, these pressures are usually termed 'uplift'.*
>
> *Within the rock foundations and at the foundation-dam interface, the presence of joints in the rock usually ensure*[s] *that uplift pressures develop quickly.*
>
> *In the body of a conventional mass concrete dam uplift pressures may not develop during the life of the dam except near the upstream face, in poor construction joints, in cracks, and in unsound concrete. However, due to the different nature of construction of a roller compacted concrete dam, it is expected that uplift pressure may develop along the horizontal lift joints if not protected by an upstream impervious membrane.*
>
> *The dam design includes a membrane over the upstream face of the primary spillway section, the left abutment and the main section of the secondary spillway. The ridge section of the secondary spillway does not have an upstream membrane, however this section has bedding mix on each layer of RCC that will reduce the permeability of the lift joints. A drainage system is provided behind the membrane, but no gallery or foundation drainage system has been included in the design.*

655 Exhibit 24, **GHD.002.0001**, .0130 (emphasis added).
656 Exhibit 24, **GHD.002.0001**, .0138 - .0139.

5.399 It is inferred from these statements that the Dam's designers saw a number of advantages in the use of an upstream ('Carpi') membrane, including its beneficial effect on reducing pore water pressures within the dam. The absence of such pressures would mean that destabilising uplift forces would not normally have to be included in the analysis of shear sliding along RCC lift joints, or in the worst case values of pore pressure less than the full hydrostatic head would need to be considered resulting in reduced values of uplift. Either their absence or reduced values (below full hydrostatic head) would have the effect of increasing the factor of safety with respect to sliding. But the use of zero or reduced pore pressure in calculations is contingent on the upstream membrane remaining effective as a water seal during the life of the dam. In addition, should the membrane leak any bedding mix placed on lift joints at the upstream face should be capable of providing sufficient water-tightness and preventing cracking.

5.400 In section 5.3.2 of the Detail Design Report it was noted that '[t]*he dam has been designed to remain safe even if the membrane and drainage system fails'.*[657] However, closer examination of the Detail Design Report reveals that membrane failure was only considered for the Usual load case of the reservoir at Full Supply Level (**FSL**). This corresponds to the headwater level at the level of the crest of the primary spillway (El 67.6 m AHD). It is apparent that membrane failure was not considered by the Alliance for any Unusual or Extreme loading events.

5.401 In its submission,[658] and with regard to the possibility of membrane failure, Hydro Tasmania, said *'… it would take a long time for pore pressure to build up within the concrete of the dam so it is unlikely that pore pressure will build up significantly during a flood event'*. As explained later, this opinion is not shared by GHD, who consider that the possibility of membrane failure ought be included for Unusual and Extreme load cases.[659]

5.402 Whether membrane failure should have been considered for all categories of loading in the original design of the Dam has a very important bearing upon the question of dam stability. The Commission heard evidence about the possibility of sliding failure under Unusual and Extreme loading events if the membrane were to fail and allow pore pressures to develop within the dam and specifically along its lift joints.[660]

5.403 The design life of the Dam was stated to be 100 years.[661] It is a fact that polymer geomembranes of the type used in the Dam have not been in existence for the past 100 years and so we do not have the benefit of experience with them and their use in dams over such a long period of time. Experience of them during earthquakes is limited.

657 Exhibit 24, **GHD.002.0001**, .0130.
658 **HYT.007.0001**, .0007.
659 **HYT.007.0007**.
660 See, for eg: Exhibit 325, **GHD.045.0001**; Exhibit 326, **HYT.004.0001**; **HYT.007.0001**.
661 Exhibit 24, **GHD.002.0001**, .0026.

5.404 In making an argument for the adequacy of the upstream membrane as a means of providing a long-term water-tight seal for the Dam and therefore also an aid to structural stability, Hydro Tasmania stated:[662]

> *the Carpi membrane system has a number of features that prevent failure during earthquake loading including a layer of geotextile between the concrete panel and the membrane – so that it would debond under any significant load. The membrane also has a strain capacity of 200% so it can stretch to double its width*.

5.405 This particular claim about the ductility of the membrane may be true now, for a relatively new membrane, but it begs the question whether it would also be true in 100 years or so from the time the membrane was installed. Data indicating the effects of ageing on the behaviour of such membranes is very limited, so this remains an open question.

5.406 Given the uncertainty, at least in the view of this Commission, concerning the behaviour and properties of the Carpi membrane over the next 100 years or so, a prudent, albeit conservative view for a dam designer would be to assume that membrane failure is a possibility within that approximate time frame. The coincidence of membrane failure and loading conditions other than Usual might also be considered as a possibility. If that view were to be adopted as being reasonable, then different assumptions should be made with respect to sliding stability analysis of the Dam. In that case, the possibility of pore water pressures acting on lift joints, some of which may have failed in tension, should be considered. It is worth reiterating the Alliance design, as documented, did not do so for other than the Usual loading condition with the reservoir at FSL.

5.407 As previously indicated, Hydro Tasmania suggested[663] that it would take a long time for pore pressure to build up within the RCC of the dam if the membrane failed allowing water under pressure to enter the dam and its lift joints. In which case, it was submitted, it is unlikely that pore pressure will build up significantly during a flood event.[664] They cited low pore pressure readings recently and during the 2013 event in support of this argument.[665]

5.408 There is no conclusive evidence that the membrane had failed and was leaking significantly at the time of the 2013 event. This line of argument is, therefore, inconclusive.

5.409 Around the time of the flooding in 2010/11 (**the 2011 event**) and during the 2013 event, the Dam spilled water over its spillway crest for 21 months almost continuously after its initial overtopping. This observation probably diminishes the power of the argument that there would be insufficient time for pore water pressures

662 **HYT.007.0001**, .0009.
663 **HYT.007.0001**, .0007.
664 **HYT.001.0007**, .0007.
665 **HYT.007.0001**, .0004 to.0005

to build up if the upstream membrane did rupture during an Unusual or Extreme loading event.

5.410 The Alliance design of the Dam included a drainage system to collect water that might seep through the upstream membrane, should it provide an imperfect seal. If a major leak or leaks did occur as a result of membrane failure, this drainage system and any bedding mix applied at the upstream face would be the only means of restricting the movement of water into the dam and its joints and the development of large pore water pressures on the lift joints of the dam. But what if the drains were blocked or unable to cope with a high rate of seepage into the Dam? If either of these circumstances were to occur the pore water pressures in the dam would rise, potentially destabilising the dam with respect to sliding along lift joints.

5.411 In the Detail Design Report it is asserted: '*The dam has been designed to remain safe even if the membrane and drainage system fails'.*[666] As previously indicated, the Dam's designers attempted to demonstrate that this statement was valid only for the case of Usual loading at FSL, and they did so assuming values of the strength parameters for the lift joints that have since been called into question by some experts.

5.412 On the contrary, on the basis of their different assumptions about conditions in the Dam and their assumed strength parameter values, GHD has demonstrated that the statement about membrane failure and dam stability quoted in the Detail Design Report is not valid, at least for some Unusual and Extreme loading cases. GHD predicted Factors of Safety against sliding that are less than 1 in their Stability Memorandum in 2019.[667]

5.413 A related factor in the design was the assumption of values of 1,000 kPa and 260 kPa for the tensile strength of the lift joints with and without bedding mix, respectively.[668] Adopting these values, the designers were able to demonstrate that tensile failure was not predicted in any of the design load cases they considered. Of course, the validity of this conclusion is directly dependent on the value assumed for the tensile strength of the lift joints. The values adopted by the designers are assumed values that have not been validated by direct testing of lift joints.

5.414 Another important assumption required in the analysis of sliding stability concerns the magnitude of pore water pressures acting on the base of the dam, i.e. the uplift pressures. In the design calculations for sliding stability, a triangular distribution was assumed when the tailwater level was below the level of the toe of the Dam.[669] The downstream value was zero, while the value at the upstream heel of the dam was assumed to be 50% of the hydrostatic pressure due to the full headwater level. When the tailwater level was above the level of the toe of the dam, the pressure distribution on the base of the dam was assumed to be trapezoidal in shape. The uplift pressure

666 Exhibit 24, **GHD.002.0001**, .0130.
667 Exhibit 16, **GHD.005.0001**, .0013, .0019.
668 Exhibit 24, **GHD.002.0001**, .0140 to.0141.
669 Exhibit 24, **GHD.002.0001**, .0143.

at the toe of the dam corresponded to the hydrostatic pressure corresponding to the full depth of the tailwater, and the pressure at the heel of the dam was 50% of the pressure corresponding to the full depth of headwater, provided that value was greater than the pressure at the toe. If the latter condition was not met, the pressure distribution beneath the dam was assumed to be uniform and equal to the tailwater pressure.

5.415 As will be demonstrated in the following section, these assumptions about the uplift pressures differ from those assumed in GHD's analysis of sliding stability. In general, the GHD assumptions are more conservative than the conditions stated here and adopted by the Dam's designers.

GHD sliding stability analysis – post-construction

5.416 In the 2020 Memorandum, GHD investigated the Dam's sliding stability and made assumptions about the strength parameters they suggest are appropriate.[670] They also differed from the designers of the Dam in terms of certain key assumption in the analysis. These key differences can be summarised as follows:

a. GHD assumed only friction on the lift joints, with zero cohesion. It argued that this approach should be adopted in stability analyses for reasons that included evidence that some lift joints in the dam were unbonded and therefore adopting a finite value of cohesion was not warranted and potentially unsafe. Several different values of the friction angle appear to have been adopted at different times in their stability calculations, viz., 41 degrees and 39.3 degrees. The assessment by GHD of the most appropriate value of the friction angle varied over time and was refined as further testing data became available. By contrast, the Alliance assumed a cohesion value of 250 kPa and a friction angle of 35 degrees ($\tan\phi = 0.7$) in the worst case of 'Poor' joint quality and 'Adopted Values' of cohesion and friction angle of 325 kPa and 40.4 degrees, respectively, corresponding to 'Good' joint quality.

b. GHD assumed the lift joints had no tensile capacity, presumably also on the basis that some lift joints were either unbonded or should be assumed as unbonded (following the 2103 ANCOLD Guideline). By contrast the Alliance assumed that the lift joints had a tensile capacity of at least 260 kPa. As a consequence of their assumption, GHD predicted that tensile failure was likely on lift joints near the upstream face of the Dam. In statements and evidence provided to the Commission, Mr Herweynen, Mr Griggs and Dr Schrader indicated that tensile failure of joints near the upstream face was most unlikely because bedding mix was placed on lift joints at the upstream face, in accordance with the design Specification.[671]

670 Exhibit 61, **SUN.009.004.0037,** .0060.

671 Mr Herweynen: **TRA.500.014.0001**, .0016 ln 16-18, .0038 ln 16-22; Mr Griggs: **TRA.500.014.0001**, .0101 ln 37-43, Exhibit 286, **GRT.001.0001**, .0008 to .0009, .0015 to .0017; Exhibit 283, **PDI.090.0001**, .0010 ln 26-45; Dr Schrader: Exhibit 111, **SCE.021.0001**, .0003.

c. GHD assumed that membrane failure was a possibility for all loading events, ie, Usual, Unusual and Extreme. As a consequence of this assumption, and GHD's prediction of tensile failure on some lift joints, full hydrostatic water pressure was assumed by GHD to act on the lift joint, tending to destabilise the dam by providing significant uplift to the section of the dam monolith above the lift joint in question. By contrast, the designers assumed uplift conditions on the lift joints only for the Usual load case with the reservoir at FSL.

d. GHD assumed the pore water pressure distribution under the Dam was either triangular or trapezoidal, depending on the level of the tailwater. Importantly, they argued that in some locations the drainage provided at the toe of the Dam was covered by concrete applied during the remediation works that followed the 2013 event. As a consequence, GHD assumed those drains were ineffective. Moreover, they assumed the uplift pressure under the base of the Dam would reach a value given by the full height of the tailwater at the downstream limit of the spillway apron, rather than at the toe of the Dam. An important further consequence of these assumptions is that the predicted uplift force will generally be larger than would be predicted if the full tailwater pressure was reached at the toe of the Dam.

5.417 On the basis of the assumptions made above, GHD concluded that the Dam did not meet the acceptance criteria recommended in the 2013 ANCOLD Guidelines for all loading scenarios considered. Furthermore, for some lift joints in some monoliths GHD predicted values for the Factor of Safety with respect to sliding on lift joints to be less than 1 under higher flood loads.[672]

5.418 The Dam's designers and other experts[673] challenged the assumptions made by GHD, describing them as overly conservative, and suggested that the cumulative effect of these conservative approaches and assumptions was undue caution.

5.419 In summary, according to the Dam's designers, and based on the assumptions they made, the Dam meets the acceptance criteria for sliding and can be considered as safe.

5.420 On the other hand, according to later studies and analysis by GHD, and based on the assumptions that GHD believes are reasonable, the Dam does not meet all current acceptance criteria as set out in the 2013 ANCOLD Guidelines. Indeed, at higher flood loads GHD predicted that the Dam is unsafe with respect to sliding.

5.421 Who is correct? This question is explored in more detail in the following section.

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672 Exhibit 16, **GHD.005.0001**, .0019.

673 Mr Herweynen: Exhibit 245, **HER.002.0001**, .0008; Mr Griggs: Exhibit 286, **GRT.001.0001**, .0018; Mr Foster: Exhibit 73, **LOF.001.0001**, .0013; Dr Rizzo suggested the analysis undertaken by GHD was reasonably conservative and in some respects, overly conservative: **TRA.500.008.0001**, .0066 ln 46-47-.0067 ln 1. Mr Foster initially suggested that the strength estimated by GHD might be too conservative (Exhibit 11, **SUN.009.003.0613**, .0621) but in examination accepted that GHD's approach was appropriate: **TRA.500.004.0001**, .0044 ln 17-43.

Is the Dam safe with respect to sliding?

5.422 Is the Dam stable in its 'as-built' configuration? Only one RCC expert who provided evidence to the commission, Dr Schrader, was prepared to state that in his assessment '*it is almost certain that the dam is stable*'.[674]

5.423 The question of whether the Dam is stable falls to be addressed with reference to the loads to which the Dam might be subjected from time to time. Also material in that consideration are all uncertainties in the mechanical properties of the RCC, specifically its strength properties, its loadings, and the methods of analysis adopted to assess its stability. The latter include any uncertainty in the various assumptions that a dam designer must make when analysing the question of stability.

5.424 It is important here to reiterate that the reliability of any conclusions about the structural stability of a dam are dependent to a very large degree upon the correctness of all assumptions that are adopted in the stability analysis and the reliability of the input parameters for that analysis, viz., strength parameters and loadings.

5.425 The question of safety is addressed here in terms of the Factors of Safety with respect to sliding calculated using the deterministic method of analysis described previously. Some of the points raised previously are repeated here for emphasis.

5.426 With reference to Figure 5.1 of GHD's Stability Memorandum[675], the GHD assessment, which conservatively assumed tensile failure of the RCC lift joints, membrane rupture and '50% uplift' and adopted a residual friction angle of 39.3° for the lift joints, indicates that monolith H in the Dam has a factor of safety greater than 1 for the Usual loading cases, including FSL and the 1:50 AEP flood load case. For these same assumptions, GHD also calculated a factor of safety greater than 1 for loading in the lower range of the Unusual flood loading cases, viz., 1:100 and 1:2,000 AEP.[676] The computed factor of safety was below 1 for the Extreme flood loading cases considered by GHD, viz., 1:10,000 AED and PMPDF load cases. However, it is noted that the calculated factor of safety was below the acceptance criteria in the 2013 ANCOLD Guidelines for most of the cases listed here. The FSL case is the exception.

5.427 According to the assumptions made by GHD and the deterministic analyses they conducted, and with reference to the current 2013 ANCOLD Guidelines, the Dam has unacceptable computed values of the factor of safety with respect to shear sliding failure. The Dam in its current configuration and subject to Unusual and Extreme loading is structurally unsafe with respect to shear sliding based on the GHD approach to that question.

674 **TRA.500.010.0001**, .0102 In 30-33.
675 Exhibit 16, **GHD.005.0001**, .0014.
676 Exhibit 16, **GHD.005.0001**, .0019.

5.428 The deterministic factor of safety approach is only a guide to safety, and not a guarantee, either of safety or failure. Its reliability is only as good as the reliability of its inputs and assumptions, as mentioned previously.

5.429 In an earlier report,[677] GHD also presented computed results for the probability of sliding failure of a number of different monoliths in the Dam. In many cases the computed values of this probability would be considered unacceptably high by many design guidelines.[678] A higher residual friction angle of 41 degrees was used in those probability calculations. Again, it needs to be emphasised that the reliability of a probabilistic analysis is also highly dependent on the quality and reliability of its inputs. Additional information is required for a probabilistic analysis compared to the deterministic factor of safety approach.

5.430 Clearly, according to GHD and their deterministic and probabilistic analyses, there are feasible conditions under which the Dam should be considered unstable with respect to shear sliding.

5.431 In contrast to the GHD conclusions regarding the possibility of sliding instability are the opinions of the Dam's designers and a number of experts who gave evidence. In particular, the Dam's designers and those experts put forward the view that the low factors of safety computed by GHD were due to what the designers and the experts in question believe were the overly conservative values adopted for parameters such as the tensile and shear strengths of the lift joints.[679] They expressed an opinion that GHD had been overly conservative with respect to the assumption they made about the uplift pressures beneath and within the dam and in particular whether the membrane might rupture allowing water under significant hydrostatic pressure to enter possibly cracked and uncracked lift joints.[680]

5.432 The difference in opinion described above raises fundamental questions about uncertainty and the probability of failure. Clearly the GHD approach to sliding stability is more conservative than that adopted by the Dam's designers. But which is more appropriate (or more correct)?

5.433 When addressing this particular question, it is important to remind the reader that any set of assumptions about material properties and conditions that might apply over time in a dam has its attendant uncertainties. For example, despite the best available knowledge, our estimates of floods and earthquakes and the loads they impose on dams may not be perfect. Upstream membranes may leak. Unusually low strength lift joints could occur in a dam, despite the best construction and quality

677 Exhibit 301, **GHD.021.0001**.

678 For example, above the Limit of Tolerability under ANCOLD, *Guidelines on Risk Assessment. Australian National Committee on Large Dams* (2003).

679 Exhibit 245, **HER.002.0001**, .0006, .0008, .0011, .0013, .0018; Mr Griggs said that GHD's assumption that lift joints have zero tensile strength at the upstream side would have a very low probability: Exhibit 286, **GRT.001.0001**, .0017 and that his friction only *'assessment shows that the stability assessment is sensitive to the overly conservative input parameters that GHD have adopted.'* Exhibit 286, **GRT.001.0001**, .0018.

680 Exhibit 283, **PDI.090.0001**, .0012, .0015 to .0016.

control practices. The essential point being made here is that all activities, including dam design and construction, have attendant uncertainties, a finite probability of an event such as sliding failure occurring, the (possibly unwanted) consequences of any failure and hence the attendant risk.

5.434 This raises a further fundamental question. What level of uncertainty about these matters and therefore what probability of possible failure of the dam is generally acceptable? Furthermore, who should make the decisions about what probability of failure and level or levels of risk are acceptable?

5.435 In answering the first of these last two questions, it needs to be recognised that there are established means of identifying society's tolerance level for certain events to occur. This is often described as a 'risk appetite'. For a structure such as the Dam, this appetite is usually very low, given the likely consequences of failure, often viewed simply in terms of the potential loss of life if the Dam were to fail.

5.436 In answering the second of these questions, it should be recognised that in many engineering projects, the choice of which set of design assumptions should be accepted may ultimately rest with the owner of the project. In general, when there are choices to be made about the level of caution to be exercised, an informed designer should normally ascertain what is the owner's risk appetite. Do they want to take the most conservative line and potentially pay more for the project upfront, or are they willing to accept greater risk and consequently accept the possibility that in the future they may face repair and reparation bills, or worse still, loss of life? This choice may not always seem clear-cut to the general public, some of whom may think no risk should always be the answer. The reality is that all construction activities have attendant risk or finite probability of failure. Zero risk or zero probability of failure is impossible to achieve.

5.437 It follows that some probability of failure always attends construction activities, and a dam is no exception. How much uncertainty or risk are we as a society or as an owner of an asset under construction prepared to accept? That decision usually bears heavily on the project cost.

5.438 In principle, it is ultimately for the owner of the Dam to make the key decisions about risk appetite, recognising that they should always act in the public interest for a piece of major infrastructure such as a dam and especially as failure of the dam is most likely to put human lives at risk. In the present case, do they wish to accept the assumptions tendered by GHD, that indicate the Dam could be potentially unstable under certain conditions, or the assumptions of the Dam's original designers that led to their conclusion that the Dam should be safe according to the acceptance criteria adopted at the time the Dam was designed?

5.439 As indicated previously, the experts who gave evidence on this issue are divided in their opinions on which assumptions are most reasonable when analysing the sliding stability of the Dam. The current owner of the Dam, however (SunWater) has decided its risk appetite, which would seem to be consistent with GHD's view of the matter. This presents as a reasonable position for a public body to adopt in connection with a Dam that lies upstream from a residential community.

5.440 Further investigation and careful testing of the RCC lift joints should more reliably characterise their shear strength.

5.441 It is difficult to foresee how the current uncertainty about the long-term behaviour of the upstream membrane can confidently be resolved in the short to medium term. Whether or not it will rupture and leak has a significant bearing on the computed factors of safety with respect to sliding.

5.442 However, further investigation and testing might yet give confidence that the lift joints have sufficient strength to resist all potential loadings with an acceptable factor of safety against sliding, even if the upstream membrane should fail.

5.443 In conclusion, the stability of the Dam with respect to sliding failure is presently uncertain.

Does cohesion resolve the stability problem?

Bedding mix

5.444 Use of bedding mix in LCRCC dams enhances the bond between lift joints. The anticipated benefit of bedding mix in this Dam can be seen from the marked difference between the nominated design values for cohesion on untreated and bedded lift joints. For a lift joint of 'excellent' quality without bedding mix, the designers assumed that the lift joint would achieve cohesion of 400 kPa. If bedding mix was used on a lift joint of that same quality, cohesion was expected to rise to 2,800 kPa.[681]

5.445 Dr Schrader's evidence before the Commission was that bedding mix did little (if anything) to increase the friction angle.[682] However, Mr Dolen considered that a benefit offered by bedding mix was to eliminate porosity because the bedding mix gets squeezed up into any voids during compaction of the upper layer of RCC. That improves shearing resistance.[683]

5.446 As is described above, bedding mix was required to be placed at the upstream face of all lift joints to a minimum width of 600 mm.[684] Mr Herweynen's recollection was that bedding mix was applied over significantly more than 600 mm although that was the width shown in the drawings.[685] In explaining the function of bedding mix, the last RCC QC Report stated:[686]

> *The bedding mix is an important component to improve the bond between layers where it is needed. This increases the lift joint cohesion and the lift joint friction, which are parameters that were taken into account for the design, as they are improved the safety factors increase.*

681 Exhibit 24, **GHD.002.0001**, .0141.
682 **TRA.500.008.0001**, .0031, ln 26-32.
683 **TRA.500.008.0001**, .0034, ln 47 to .0035 ln 4.
684 **DNR.006.0001**, .0017.
685 Exhibit 244, **HER.001.0001**, .0038 [176].
686 Exhibit 38, **SUN.110.003.0001**, .0103.

5.447 An article prepared by Mr Lopez, Mr Griggs, Mr Montalvo, Mr Herweynen and Dr Schrader about RCC construction and quality control for the Dam, described the use of bedding mix in the Dam in the following way:[687]

> *Bedding mix was used to improve the bond between layers of RCC. This improved the shear strength properties and watertightness of the lift joints.*
>
> *Bedding mix was generally placed on the area upstream section of the dam and areas of poor lift joint quality. Bedding mix was placed in sections less than 15 m in front of the RCC progressing edge to avoid over-exposure.*
>
> *…*
>
> *The chosen bedding mix had an MSA of 10 mm and cement content of 300 kg/m^3 with a water cement ratio of 0.80. Superplasticiser, air entrainment agent, and retarder admixtures were included into the mix design. From September 2,004 the cement content in the mix was reduced to 200 kg/m^3, replacing 100 kg/m^3 of cement with equal weight/m^3 of fly ash.*

5.448 Dr Schrader's evidence was that use of bedding mix in the manner adopted for the Dam was common practice for dams around the world:[688]

> *We use a bedding mix where we need more cohesion, more shear resistance, watertightness or tensile strength, and it's a lean mix; you put bedding mix on. That's just the practice around the world. There must be, I don't know, 50 or 100 dams like that.*

5.449 Dr Schrader gave that evidence in the context of discussing the 2013 ANCOLD Guidelines, which require that '*unless there is strong evidence to support bonded lift joints or investigations based on cored samples are undertaken, all concrete lift joints should be considered as unbonded*'.[689] The evidence above was given in response to a suggestion that the 2013 ANCOLD Guidelines require that, if the entirety of an RCC lift is not bonded, the whole lift should be regarded as unbonded. Dr Schrader was of the view that the drafters of the guidelines could not have intended that outcome. According to Dr Schrader, the use of bedding mix at the upstream face of RCC lifts in the Dam is consistent with the design of dams all over the world.[690]

5.450 The evidence of Mr Tarbox was consistent with that view. Mr Tarbox said that there were lots of LCRCC dams where, rather than covering the entire lift surface, bedding mix was applied over some percentage of the upstream portion. Mr Tarbox said that was '*an acceptable and effective technique*'.[691]

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687 Exhibit 75, **PDI.037.0001**, .0008.
688 **TRA.500.008.0001**, .0033, ln 27-31.
689 Exhibit 35, **ACD.001.0001**, .0026.
690 **TRA.500.008.0001**, .0033, ln 16-27.
691 **TRA.500.007.0001**, .0026 ln 15-21.

5.451 Mr Dolen did not seem to share the view expressed by Dr Schrader and Mr Tarbox, at least for cold joints. Instead, he indicated that it was usual for bedding mix to be applied over the full extent of a cold joint.[692]

5.452 The RCC QC Report for August and September 2005 presented the summary below of the bedding mix placed each month from June 2004 until September 2005:[693]

Month	Monthly Placement	Cumulative Volume
June 04	144	144
July 04	109	253
Aug 04	77	330
Sept 04	145	475
Oct 04	176	651
Nov 04	369	1020
Dec 04	553	1573
Jan 05	769	2342
Feb 05	1202	3544
March 05	1355	4899
April 05	1722	6621
May 05	763	7384
June 05	461	7845
July 05	184	8029
Aug 05	317	8346
Sept 05	180	8526

Table 29. Bedding mix placement – Monthly summary

Figure 5.26 – Monthly summary of bedding mix placed from final RCC QC Report

5.453 The report went on to explain, both in narrative form and by graphical representation, the amounts of bedding mix that had been used in different sections of the Dam:[694]

> *Bedding mix / RCC volume ratio during the dam construction ranged from 5.7 % to 1.54 %. High ratios were obtained when the RCC was placed on the rock foundation (June to October 2,004) and once the dam reached the top level at the Main Spillway Section where the area covered by bedding was increased (April & May 2,005). Lower ratios reached correspond to lower level of the dam, where the volume of bedding mix per RCC volume placed in the dam decreased. In the period June & July 2,005 the ratio increased due to placement in the Main Spillway Apron that required covering the previous surface with bedding mix Average bedding mix / RCC volume ratio is equal to 2.14% (See Figure 74).*

692 **TRA.500.009.0001**, .0058 ln 23-27.
693 Exhibit 38, **SUN.110.003.0001**, .0104.
694 Exhibit 38, **SUN.110.003.0001**, .0104 to .0105.

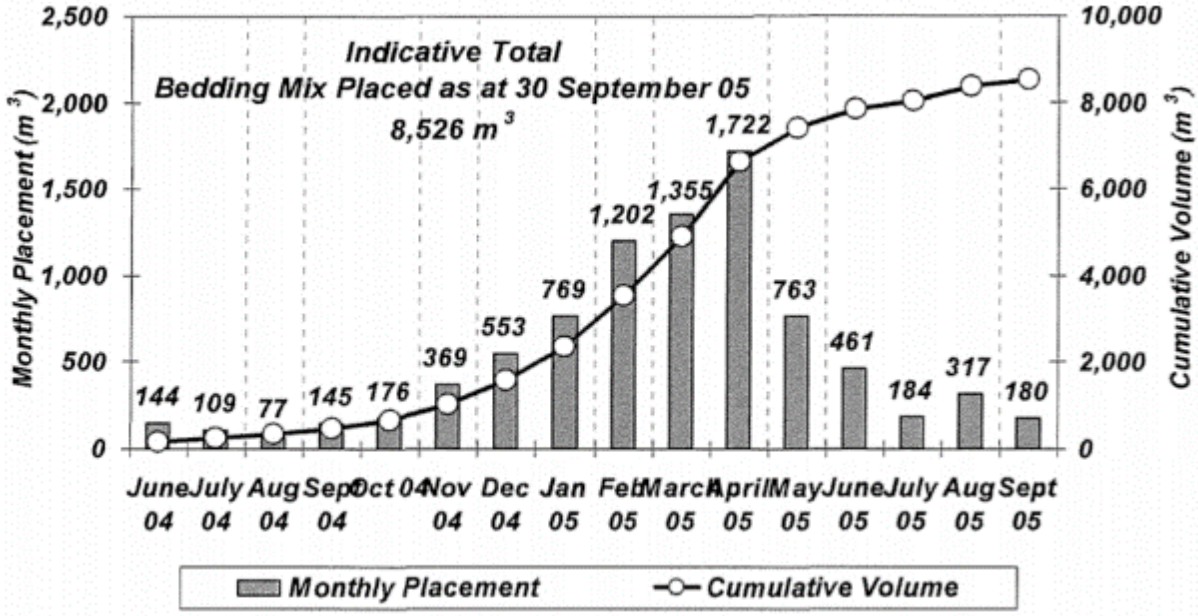

Figure 73. Bedding Mix Placed as at 30 September 2,005

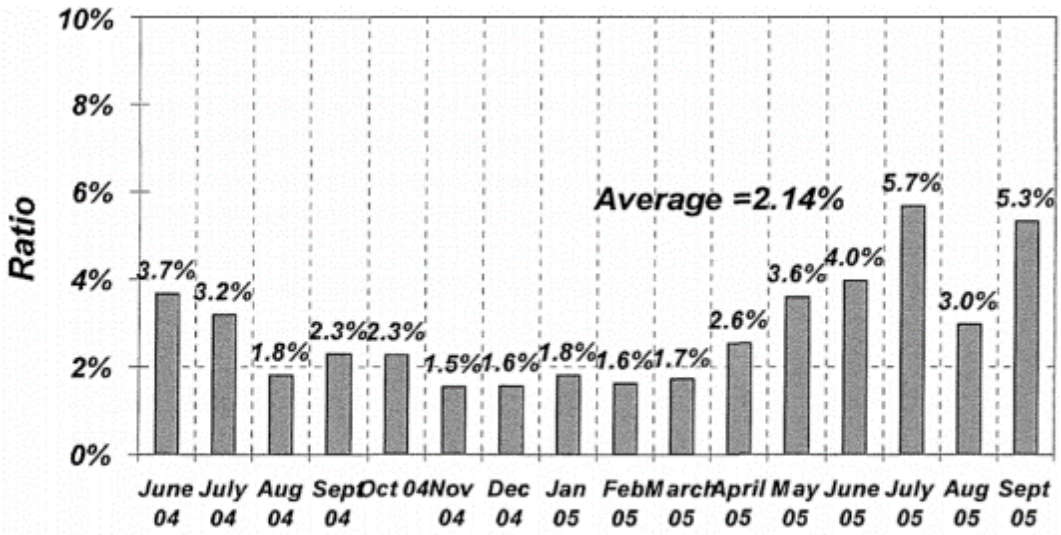

Figure 74. Bedding/RCC volume ratio during dam construction

Figure 5.27 – Charts of bedding mix placed during construction from final RCC QC Report

Surface coverage with bedding mix

5.454 During an interview, Mr Herweynen emphasised the use of bedding mix in the Dam because it '*was always placed in the upstream zone and the bedding mix [had] a significant impact on the shear strength of the project*'.[695] By reference to the graphs directly above, Mr Herweynen pointed to the high RCC production rates from November 2004 to March 2005 when the bedding mix placed was around 1.6% of

695 Exhibit 247, **TRA.510.007.0001**, .0075 ln 44 to .0076 ln 1.

the RCC placed.[696] That provided an average measure of the amount of bedding mix placed on lifts away from the top of the Dam and above the level of the foundation.

5.455 Based on Mr Herweynen's calculations, full coverage of each lift joint by a 25 mm thick layer of bedding mix would correspond to 8% of the volume of RCC, based the ratio of the thickness of the bedding mix layer (nominally 25 mm) to the thickness of the RCC layers (nominally 310 mm). The average of 1.6% placed divided by 8% for complete coverage gives an average of 20% coverage of the lifts with bedding mix.[697] Some or all of that 20% would have been placed on the lift joints at the upstream face.

5.456 As is discussed above, the construction drawings required bedding mix to be placed in a strip 0.6 m wide at the upstream face of the Dam. Near the base of the primary spillway, which is approximately 36.75 m wide,[698] a 0.6 m strip represents only 1.6% of the area of the lift joint. Near the top of the dam this average percentage coverage in a 0.6 m wide strip would reach approximately 5% where the Dam is close to 12 m wide. However, as noted in the final RCC QC Report and by Mr Herweynen, large percentages of bedding mix were used close to the top of the main spillway. It is reasonable to conclude that, on average, the amount of bedding mix applied to these lift joints near the top of the spillway was larger than the minimum requirement of a 0.6 m wide strip at the upstream face.

The location of bedding mix

5.457 While bedding mix was required at the upstream face of every lift joint, it was also used to treat cold joints. The Specification required a Type I cold joint be treated with bedding mix spread over the upstream 25% portion of the lift surface, although that requirement was relaxed to 10% by Dr Schrader. The specified treatment for Type II cold joints was for 30% of the upstream portion of the surface to be treated, although that was relaxed to 15% by Dr Schrader. There was conflicting evidence before the Commission about whether the recommendation to reduce the width of bedding mix on Type I and Type II cold joints made by Dr Schrader in memoranda sent to Mr Herweynen and Mr Hamilton on 2 August 2004 was ever implemented. As is reasoned elsewhere in this report, it is more likely than not that the change was implemented.

5.458 In any event, the requirements for the placement of bedding mix over cold joints, as set out in the Specification and memoranda generated by Dr Schrader, help explain in general terms why and where bedding mix was applied. However, on the evidence, it is not possible to determine with certainty exactly where all bedding mix was applied.

696 Exhibit 247, **TRA.510.007.0001**, .0076 ln 24-28.
697 Exhibit 247, **TRA.510.007.0001**, .0076 ln 30-35.
698 **DNR.006.0001**, 0016.

Bedding mix location not recorded on quality assurance documents

5.459 The location in which bedding mix was placed appears not to have been routinely recorded on quality assurance documentation, although other related details were documented. Cold joints were prevalent in the Dam; '*most of the lift joints were cold joints*'.[699] The incidence of cold joints was recorded on RCC Placement ITPs. An RCC Inspector and an RCC Engineer were both required to sign off on hold point 3:[700]

> *Is a RCC Cold joint treatment required in accordance with Specification S11.10. Refer to checksheet BDA-QA-CHK-0061*

5.460 The Cold Joint Treatment Checklist was the document bearing the 'BDA' reference number quoted above. One of the inspection and sign-off points was: '*Bedding mix spread not thicker than 25mm over the entire surface that is to receive RCC*'.[701] Neither the RCC Placement ITPs nor the Cold Joint Treatment Checklists made provision to record the quantity of bedding mix placed nor the location in which it had been spread.

5.461 The LJQI Scorecard appears to have been revised several times as construction progressed. Early in the project, the form had a section for the area covered with bedding mix to be recorded. There were examples of LJQI Scorecards completed in September 2004 that record the square metre coverage with bedding mix.[702] However, an example on of the LJQI Scorecard dated 16 November 2004 leaves that part of the form blank despite other details indicating that bedding mix had been applied because the lift was a cold joint.[703] By February 2005, the section of the LJQI Scorecard for recording bedding mix coverage had been blacked out completely.[704]

5.462 Bundles of QC documents for RCC placement typically included an RCC Placement ITP and an LJQI Scorecard. Some bundles also include batch supply dockets from Wagners for bedding mix deliveries. The dockets provide some indication of the delivery point by the description provided as a 'delivery address'. However, those details were completed in broad terms that do not align with the lot number or work area of the RCC placement. Some examples include:

a. for RCC placement at EL 66.185 m to EL 66.495 m in the left abutment from chainage 130 m to chainage 175 m,[705] a Wagners delivery docket describes the delivery address as 'L/H ABUTMENT'[706]

699 **TRA.500.006.0001**, .0037 ln 40-41.
700 See, for example, Exhibit 117, **DNR.020.014.4624**, .4624.
701 See, for example, Exhibit 117, **DNR.020.014.4624**, .4626.
702 Exhibit 299, **SUN.114.001.0273**, .0281; **DNR.020.014.5202**, .5204.
703 Exhibit 117, **DNR.020.014.4624**, .4625.
704 Exhibit 310, **SUN.021.006.1405**, .1406.
705 Exhibit 119, **SUN.112.002.0368**, .0368.
706 Exhibit 119, **SUN.112.002.0368**, .0373.

b. for RCC placed at EL 47.895 m between from chainage 490 m to chainage 525 m,[707] a delivery docket describes the delivery address as '*MAIN SPILLWAY*'[708]

c. in respect of RCC placed in the right abutment at EL 53.445 m from chainage 635 m to chainage 655 m,[709] a delivery docket describes the delivery address as '*RCC PAD CRETER CRANE*'.[710]

5.463 Produced to the Commission were copies of other Wagners dockets where the content was blank and could not be read.[711]

5.464 Mr Herweynen gave evidence that the batch supply records of Wagners would identify the volume of bedding mix that was placed at a certain location.[712] However, the evidence does not permit a determination to be made about where bedding mix was placed. The delivery dockets are vague in their description of where bedding mix was delivered. There is no firm link between the dockets and a particular RCC placement area. Even if that link could be established, the delivery dockets do not show where on the lift surface bedding mix was placed. It is not open to assume that all bedding mix was placed at the upstream face, because there is evidence before the Commission that bedding mix was applied locally to other areas, for instance, to repair localised surface segregation.[713]

5.465 The Commission has not been made aware of other records that might indicate the location in which bedding mix was placed. It is not possible to determine definitively where bedding mix is located in the Dam.

Indications from corehole samples

5.466 The top of the corehole taken from the Dam in 2006 was offset from the upstream face by 3.3 m and angled downstream 5º from the axis of the Dam. Dr Schrader was involved in selecting the location of that corehole. Asked whether it was by design that the corehole should miss all the bedding mix, Dr Schrader gave the following response:[714]

> *Yes. And, again, that was certainly what I intended, and you will have to talk to Roberto [Montalvo] about precisely where it was and precisely where was the bedding mix used. But that was by design.*
>
> *As I said, now, I would say absolutely what a mistake. We should have done it in the best of RCC. But I wasn't trying to show the best and say, 'This is typical'; I was trying to get what really was maybe the worst - not "maybe", would be the*

707 Exhibit 276, **SUN.113.005.0202**, .0202.
708 Exhibit 276, **SUN.113.005.0202**, .0206.
709 **SUN.112.003.0186**, .0186.
710 **SUN.112.003.0186**, .0190.
711 Exhibit 310, **SUN.021.006.1405**, .1407.
712 Exhibit 244, **HER.001.0001**, .0041 to .0042 [192].
713 Exhibit 244, **HER.001.0001**, .0041 [191].
714 **TRA.500.010.0001**, .0107 ln 3-14.

worst - and be really confident that upstream everything was going to be fine. Was fine. Is fine.

5.467 This evidence is at odds with the preliminary comments about the coring that explain that:[715]

The location of the hole for core extraction was selected with the aim of obtaining test data for the following placement scenarios that took place during construction:

- *All Conveyor placement (with / without bedding mix)*
- *Placement with trucks or other (with / without bedding mix)*

5.468 However, the preliminary comments seem to indicate, although it is not explicit, that bedding mix was not intercepted. The report says that:[716]

Due to the location of the hole and the actual 'route' it followed through the dam, the following applies:

…

- *Typically, the centre of the dam is technically not as critical as the upstream third of the dam, mainly relying on mass and some friction.*

5.469 Mr Montalvo thought that the core was taken from the middle of the Dam. Bonding was expected on the upstream section where bedding mix was placed but the core was from the middle so the lift joints not being bonded was not a concern. '*There was an expectation that some of them were not going to be [bonded]*'.[717]

5.470 Some of the cores drilled subsequently through the dam were as close as 2.4 m and 2.5 m from the upstream face.[718] An offset of 2.4 m may be within the 20% width of bedding mix at the upstream face of the Dam, particularly lower down in the Dam. However, the Commission was not taken to evidence showing that those coreholes had intercepted bedding mix.

5.471 GHD's Updated Shear Strength Memorandum[719] discusses the 2019 shear strength testing results for 'bonded' joints. The cohesion intercept on the relevant plot of normal and shear stresses,[720] as assessed statistically by GHD and summarised in Table 6.3 of the Memorandum,[721] is well below the design cohesion values for bonded joints. It is in the order of magnitude of the design cohesion value for lift

715 Exhibit 95, **ALC.001.001.1683**, .1683.
716 Exhibit 95, **ALC.001.001.1683**, .1683 to .1684.
717 **TRA.500.006.0001**, .0034 ln 20-46.
718 Exhibit 61, **SUN.009.004.0037**, 0062.
719 Exhibit 61, **SUN.009.004.0037**.
720 Exhibit 61, **SUN.009.004.0037**, 0059.
721 Exhibit 61, **SUN.009.004.0037**, 0060.

joints with no bedding mix. That would tend to indicate that the sections of the corehole subjected to shear testing did not include bedding mix at the joints.

Effect of bedding mix on joint shear strength

5.472 Consider the possibility of shear failure along a lift joint (or joints). Furthermore, ignore for the moment the possibility that if shear failure should occur it could be progressive in nature, as opposed to occurring simultaneously across the entire lift joint. Assume also that the shearing resistance of each joint is given by the Mohr-Coulomb strength criterion that simply states that the mobilised resistance is the sum of frictional and cohesive components of the resistance.

5.473 Given these assumptions, an estimate can be made of the maximum possible shear resistance of the lift joint with bedding mix applied to only a portion of its surface area. This can be done by calculating the contribution to the strength from friction over the entire width of the lift joint and adding to it the component due to the cohesion mobilised only over that section of the lift joint that has had bedding mix applied.

5.474 If progressive failure were to occur, in principle, the shearing resistance of the joint could be somewhat less than the value calculated according to the procedure just outlined. Estimating how much less is not a straightforward matter. Nevertheless, the procedure outlined can be used to provide an upper bound estimate of the strength in shear of a lift joint treated with bedding mix. This is probably best illustrated by a numerical example.

5.475 Consider a planar lift joint that has a proportion x (i.e. 100x%) of its area covered by bedding mix. Also assume that the strength of the bedding mix is cohesive-frictional, with a cohesion intercept of c_b and a friction angle of ϕ. Further, assume that the portion of the lift joint that has no bedding mix (1-x, i.e. 100(1-x)%) has only a frictional component of shear strength, with a friction angle also of ϕ. For the moment numerical values have not been assigned to the bedding cohesion intercept c_b nor the friction angle ϕ.

5.476 The maximum possible shearing resistance that could be mobilized on this lift joint is therefore given by the expression: $xAc_b + N\tan\phi$, where A is the total area of the lift joint and N is the effective normal force acting across the plane of the lift joint. The first term in the expression is the maximum possible shearing resistance that can be mobilised due to the cohesion alone on the section of the lift joint with bedding mix, while the second is the contribution of friction over the entire lift surface.

5.477 If, as occurs often in design, the overall shear strength of this nonhomogeneous lift joint is interpreted or expressed in terms of an average or representative value of cohesion, deemed to apply across the entire surface area of the lift joint, say a cohesion intercept c, as well as a friction angle ϕ, then in this case the shear strength of the entire lift joint would be expressed as: $Ac + N\tan\phi$.

5.478 Equating these two expressions for the maximum shear resistance and simplifying provides the following relatively simple outcome, $c = xc_b$. In other words, in this case

the value of effective cohesion that could be applied to the entire surface area is simply the cohesion of the bedding mix multiplied by the proportion of the overall area that has had bedding mix applied.

5.479 For example, if bedding mix with cohesion of 2,000 kPa (or 2 MPa) was applied over 20% (x = 0.2) of the lift joint area then, provided the strength of the entire surface area is mobilised simultaneously, the effective cohesion of the entire joint is calculated as 0.2*2,000 kPa = 400 kPa.

5.480 As mentioned previously, the calculations presented above rely on the assumption that the maximum shear strength of the interface is mobilised simultaneously at all locations on the lift joint. This situation is uncertain in practice. Theoretically, it would require the sliding block and the material below the interface to be perfectly rigid or at least very, very stiff. RCC is not a perfectly rigid material. It is deformable with a finite value of its deformation modulus. In evidence Dr Schrader suggested it was more like rubber than glass.[722] Indeed, he also proposed the analogy of a lift joint with incomplete coverage of bedding mix being like two pieces of wood glued together, with glue applied only at discrete locations rather than over the entire contact surface.[723]

5.481 It is therefore possible that shear failure along the sliding plane could be progressive, with sliding initiating at or near the stiffer upstream face of the dam where the lift joints have bedding mix applied. In such cases some if not all of the cohesion may be destroyed before a sliding failure surface develops through the full width of the lift joint. Calculating accurately how much of the cohesion is destroyed and how much can still be counted on at the point where the sliding occurs over the full width of the joint is a very challenging task, with the reliability of the outcome of any such calculation being questionable.

5.482 The possibility of progressive failure was also mentioned by GHD in its Stability Memorandum[724] and by Mr Willey in evidence, when he discussed the matter of strain compatibility.[725] However, the appropriateness of considering the concept of progressive failure in the analysis of dam sliding failure was challenged by Mr Herweynen in his evidence.[726]

5.483 It is worth noting that the possibility of progressive failure is referred to in several guidelines in the context of considering potential sliding failure of a dam along the rock mass upon which it is founded. For example, the USBR publication 'Design of Small Dams' defines progressive failure as follows:[727]

722 **TRA.500.010.0001**, .0095 ln 17-29.
723 **TRA.500.009.0001**, .0032 ln 2-31.
724 Exhibit 61, **SUN.009.004.0037**.
725 **TRA.500.003.0001**, .0091 ln 21-30.
726 **TRA.500.013.0001**, .0068 ln 39-44.
727 Exhibit 229, **BOR.001.0001**, .0679.

PROGRESSIVE FAILURE: Failure in which the ultimate shearing resistance is progressively mobilized along the failure surface

5.484 The same publication also contains the following advice on how stability calculations should be carried out if there is a possibility of progressive failure.[728]

> *The total shear resistance against potential sliding along nonhomogeneous foundation planes is the summation of the shear resistance of all the materials along the plane,* ***at compatible shear displacements****.*

5.485 The US Army Corps of Engineers (**USACE**) publication 'Gravity Dam Design - EM 1110-2-2200'[729] also addresses the possibility of progressive failure occurring along a potential sliding surface and how that possibility may be dealt with in the 'limit equilibrium' approach to the analysis of sliding stability:

> *Considerations regarding displacements are excluded from the limit equilibrium approach. The relative rigidity of different foundation materials and the concrete structure may influence the results of the sliding stability analysis. Such complex structure-foundation systems may require a more intensive sliding investigation than a limit-equilibrium approach. The effects of strain compatibility along the assumed failure surface may be approximated in the limit equilibrium approach by selecting the shear strength parameters from in situ or laboratory tests according to the failure strain selected for the stiffest material.*

5.486 A lift joint that has bedding mix applied over only a limited portion of the lift can be regarded as a 'nonhomogeneous' potential sliding plane. Different strength parameters will generally apply to the portion that contains bedding mix and the portion without bedding mix. So when calculating the sliding resistance of such a potential sliding plane due regard should be given to the different displacements at which the different portions of that plane will mobilise their shearing resistance.

5.487 A means of calculating an upper bound on the effective cohesion that may be used in design to represent the entire lift surface has been identified previously. The design adopted a value of cohesion of 2,400 kPa for bedding mix corresponding to lift joints of 'good' quality. Applying the calculation method described previously provides upper bound values of the effective cohesion of 240, 360 and 480 kPa for the entire lift joint for cases where the bedding coverage amounts to 10, 15 and 20% of the total lift area, respectively. These upper bound values can be compared with the design value of 325 kPa identified in Table 5-4 of the Detail Design Report for 'good' quality untreated joints.[730] Whether such values of effective cohesion can be mobilised in practice if progressive failure were to take place remains an open question. According to the Detail Design Report, the upper bound values of effective

728 Exhibit 229, **BOR.001.0001**, .0371 (emphasis added).
729 Exhibit 248, **HER.003.0001**, .0026
730 Exhibit 24, **GHD.002.0001**, .0141.

cohesion identified in this paragraph would all be sufficient to ensure sliding stability of the Dam.[731]

5.488 This question of what value of effective cohesion to assume in a stability analysis would not arise if bedding mix had been applied to the entire surface area of a lift joint. In that case the joint would not be classified as nonhomogeneous. As noted previously, in the evidence provided by Mr Dolen, complete coverage of a lift joint was identified as being the usual requirement in practice for cold joints.[732]

Root cause: uncertainty as to stability

No shear strength testing was carried out on the Dam by the Alliance

5.489 Core samples were taken from a single borehole through the Dam in January and February 2006. The Specification did not require that cores be tested to verify the shear strength of lift joints. The Specification required '*the quality of bedding and RCC between layers*' to be determined by coring and permeability testing.[733]

5.490 Mr Herweynen and Dr Schrader discussed shear strength testing. In an email dated 14 May 2005, Mr Herweynen sought Dr Schrader's advice on the validation coring and whether shear strength testing should be undertaken.[734] Dr Schrader responded:[735]

> *In order to develop shear data that is really representative, you need a number of cores at two different angles, less than the friction angle. So, do not bother with this unless you get say at least five cores at each of two angles drilled at say 42 and 35 degrees from horizontal (quite flat). Even then, this could be statistically not useful unless the coefficient of variation is relatively low (say less than about 15%-20%). …*
>
> *I would do the angled hole form the abutment that we talked about when I was there last time, and* ***probably forget about shear tests****. We will have visual examination of lift joint bonding form the cores.*

5.491 After the due diligence with SunWater in August 2005, there was further discussion within the Alliance about shear strength testing. Dr Schrader, in a memorandum about 'RCC Cores' dated 15 August 2005 to Mr Herweynen, Mr Hamilton, Mr Embery and Mr Montalvo,[736] referred to section 11.16.5 of the Specification and its requirement of '*coring and permeability testing*'.[737] He noted (correctly) that the only testing contemplated there concerned permeability, and not shear strength nor compressive or tensile strength. He suggested, however, that a core be taken and

731 Exhibit 24, **GHD.002.0001**, .0156.
732 **TRA.500.009.0001**, .0058 ln 23-27.
733 Exhibit 21, **DNR.003.8385**, .8477.
734 Exhibit 215, **SCE.023.0001**, .0002.
735 Exhibit 215, **SCE.023.0001**, .0001 (emphasis added).
736 Exhibit 213, **ALC.002.001.0936**, .0937.
737 Exhibit 21, **DNR.003.8385**, .8477.

testing done, including because the Alliance had said to SunWater's due diligence team that it would be forthcoming. He suggested detailed steps for taking and handling the core, and proposed that it be inspected to see if the lifts were bonded.[738]

5.492 The taking of the core proceeded as discussed earlier. After the core had been drilled, Dr Schrader communicated with Mr Montalvo, Mr Lopez and Mr Herweynen by an email in which he said, '*You will not be able to afford to do proper shear tests, so forget about them*'.[739] The email later said, '*Forget all Ann bout shear testing- it would be nice, but we do not have the money or equipment, and it is tricky*'.[740]

5.493 Dr Schrader's advice not to shear test sat against a backdrop where the LCRCC used to build the Dam required consistent and constant good construction practices in RCC placement to ensure that the design parameters were achieved.[741] Dr Schrader accepted that the mix as designed was one that required '*fairly rigorous adherence to good construction practices*'.[742] However, he was aware that such rigorous adherence had not been achieved as is evidenced by his comment in May 2005 that *'the cost of doing the core is MUCH less than the cost of any one of the lifts that we could have taken out (and arguably should have per spec), but we left in place'*.[743]

5.494 Dr Schrader's advice would not have been surprising to the designers. Mr Herweynen's evidence was that there was never any intent to do a validation test on shear strength.[744] Dr Schrader's advice was accepted. No shear strength testing was undertaken on the Dam until 2015, even though that was the only sure way to be satisfied that the design parameters had been achieved and that the Dam had a sufficient margin of sliding stability.[745]

What is usually done to verify shear strength?

5.495 Identifying methods for adopting and verifying shear strength design values was a focus of the hearings when RCC experts testified concurrently.

Views of the RCC experts

5.496 Two of those four experts were of the view that shear strength testing is done frequently to verify shear strength parameters of an RCC dam.

5.497 Dr Rizzo said that practically all projects that he works on have shear strength testing done. Testing is usually carried out during the early stages of construction, including

738 Exhibit 213, **ALC.002.001.0936**.
739 Exhibit 217, **SCE.025.0001**, .0001.
740 Exhibit 217, **SCE.025.0001**, .0002.
741 **TRA.500.007.0001**, .0021 ln 16-25.
742 **TRA.510.006.0001**, .0036 at ln 36 – 42.
743 Exhibit 215, **SCE.023.0001**, .0001.
744 **TRA.500.013.0001**, .0056 ln 21-22.
745 **TRA.500.007.0001**, .0037 ln 41 to .0038 ln 2.

by sampling from a test section.[746] Mr Dolen's experience is that testing is done in major construction either in the laboratory as part of the mix design stage or from a test section before construction. For 'other structures', it is done frequently as post-construction confirmation.[747]

5.498 This is consistent with the evidence of two other RCC experts. Mr Tarbox believes that confirmation testing is engineering good practice. He said:[748]

> *I'm a strong proponent that all dams, when they are completed, should be sampled and tested as a matter of good engineering practice.*
>
> *On the last three dams that I've been associated with, the owners and the regulatory bodies have all agreed to do such investigative programs and they have been not only valuable for the particular owner and that dam, but they are contributing a tremendous amount of good information to the field and construction which is informative of the rest of the community, and I think it's just good practice going forward.*

5.499 Mr Brigden said that RCC mixes should be subjected to testing for shear strength during the trial embankment stage.[749] However, he was not surprised that a corehole was not taken during the trial embankment stage at the Dam because the RCC was too young. Instead, he expected to see coring out of the trial embankment once the RCC had matured or, if it were possible, out of the Dam.[750]

5.500 The view of the two remaining RCC experts – Mr Tatro and Dr Schrader – was that shear strength testing is not typical. Mr Tatro said that shear strength testing is not normally done. The exception might be on large or complex projects where financial savings can be made by narrowing down the properties during the design stage.[751] Dr Schrader said testing was seldom done except on 'larger projects' when it was done some of the time.[752] Dr Schrader recalled that in the early days of RCC technologies, the USBR or USACE would do testing. However, it is now seldom done for smaller projects unless '*it's something special, there's government money involved, there's time available*'.[753]

5.501 Where test data is not available, Dr Schrader, Mr Dolen and Mr Tatro said that shear strength values for the design of a dam are typically based on historical data for

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746 **TRA.500.008.0001**, .0006 ln 1-15.
747 **TRA.500.008.0001**, .0006 ln 19-47.
748 **TRA.500.007.0001**, .0038 ln 15-26.
749 Exhibit 48, **TRA.510.025.0001**, .0025 ln 20-27.
750 Exhibit 48, **TRA.510.025.0001**, .0026 ln 8-33.
751 **TRA.500.008.0001**, .0005 ln 26-42.
752 **TRA.500.008.0001**, .0007 ln 4-15.
753 **TRA.500.008.0001**, .0007 ln 17-30.

comparable mixes.[754] In the absence of test data, Dr Rizzo's approach is to adopt a friction angle of 45º. He does not rely on cohesion.[755]

5.502 All four experts agreed that shear strength testing should be done, as a matter of engineering good practice, if there was a reason to doubt that the design values had been achieved. Examples of when doubts might be raised included where:

a. a quarry exposed fairly poor quality aggregate where good quality rock had been anticipated[756]

b. the assumed values for cohesion and friction were very high[757]

c. there was a question about the quality of the lift joints.[758]

5.503 When asked whether an LCRCC mix made any difference to testing, Dr Schrader said:[759]

> *To be fair about it all, I think there should be more of a tendency to be a bit more cautious with the leaner mixes and certainly a lot more cautious in how they're tested and how they're handled. If you make a mix that's very high strength, very small aggregate, you can make a relatively small sample, handle it, and it's not very delicate. But the lean mixes are more delicate.*

5.504 Mr Dolen said that he would require confirmation testing for an LCRCC dam for assurance that the RCC was fully compacted throughout the lifts. Because workability of LCRCC is low compared to HCRCC, there is more potential for problems in achieving full compaction throughout the layer, which testing would confirm.[760]

5.505 The cementitious content made no difference according to Mr Tatro[761] and Dr Rizzo.[762] Dr Rizzo's practice was to confirm with testing, preferably from the dam, that the design parameters had been achieved.[763]

5.506 Three of the four experts said that LCRCC mixes were less amenable to confirmation testing than HCRCC mixes. Dr Schrader said that it was easier to have confidence in cores taken from HCRCC than from LCRCC.[764] Mr Tatro agreed that it was more difficult reliably to test LCRCC. He explained that because the strengths are low, the

754 **TRA.500.008.0001**, .0007 ln 37-47 (Dr Schrader), .0008 ln 10-22 (Mr Dolen); .0008 ln 34-36 (Mr Tatro).
755 **TRA.500.008.0001**, .0008 ln 27-30.
756 **TRA.500.008.0001**, .0011 ln 5-21 (Dr Schrader).
757 **TRA.500.008.0001**, .0008 ln 44 to .0009 ln 6, .0009 ln 31-39 (Mr Tatro).
758 **TRA.500.008.0001**, .0010 ln 21-24 (Dr Rizzo), .0010 ln 29-41 (Mr Dolen agreeing).
759 **TRA.500.008.0001**, .0011 ln 39-45.
760 **TRA.500.008.0001**, .0012 ln 2-20.
761 **TRA.500.008.0001**, .0013 ln 25-31.
762 **TRA.500.008.0001**, .0012 ln 24-25.
763 **TRA.500.008.0001**, .0012 ln 38-46.
764 **TRA.500.008.0001**, .0015 ln 13-17.

processes of drilling, preparing the samples, and testing require more care to ensure that the results are not influenced.[765] Dr Rizzo said it was too difficult to get a valid core sample of LCRCC to test. While Dr Rizzo was not prepared to say that testing of a core of LCRCC could not be done, his view was that the most indicative results come from testing block samples of LCRCC.[766] None of the experts said that coring of LCRCC was not possible.

5.507 Mr Dolen's experience was that if LCRCC was well compacted throughout, it was possible to drill cores and obtain samples for testing. Problems with porosity made it more difficult to obtain core samples.[767] Mr Dolen had earlier acknowledged that LCRCC is more susceptible to porosity because it is less workable.[768]

5.508 The RCC experts were divided as to whether shear strength testing should be done as a matter of course. Dr Schrader and Mr Tatro said confirmation testing was seldom done and only required where there were questions about whether the design values had been achieved. However, four RCC experts believed that shear strength testing should be done to confirm design values. Also contrary to the views of Dr Schrader and Mr Tatro are several of the design guidelines for RCC that were in effect at the time the Dam was designed. They recommend that shear testing be conducted.

Guidelines applicable during design and construction of the Dam

5.509 When the Dam was designed, there was industry guidance either to conduct verification testing of the RCC to ensure that the design parameters were met or else to adopt conservative design assumptions. The Alliance did neither.

1991 ANCOLD Guidelines

5.510 ANCOLD's 1991 *Guidelines on Design Criteria for Concrete Gravity Dams* (**1991 ANCOLD Guidelines**) were in effect when the Dam was designed. They referred to testing the bonds between RCC layers. The introduction said:[769]

> *Brief comments are made on special criteria for existing dams and roller compacted concrete (RCC) dams. It is suggested that the designer may use his judgment in relaxing some of the criteria for existing dams where the design assumptions are supported by observation and measurement. In the case of RCC dams, the greater difficulty in obtaining bond between layers is recognised so the suggested criteria are more conservative. However, provision is made for relaxation in the light of tests on bond between the RCC layers.*

5.511 The 1991 ANCOLD Guidelines went on to say, importantly, that '*tension and sliding strengths at a joint within the concrete* ***should be assessed from tests***'.[770] In the absence of test data, the Guidelines provided the following baseline assumptions:[771]

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765 **TRA.500.008.0001**, .0013 ln 39-47.
766 **TRA.500.008.0001**, .0014 ln 4-10.
767 **TRA.500.008.0001**, .0014 ln 19-27.
768 **TRA.500.008.0001**, .0012 ln 2-20.
769 Exhibit 33, **ACD.003.0001**, .0006.

For roller compacted concrete (RCC):

- *ultimate tensile strength = 0 (unless shown otherwise by tests)*
- *peak effective cohesion = 0.02 f'_c MPa (**unless shown otherwise by tests**)*
- *peak effective coefficient of friction = 1.0*

It should be noted that reasonable tensile and cohesive strengths are more easily obtained by the use of an high paste content in RCC than a lean variety.

5.512 In recommending 2% of compressive strength for the peak effective cohesion, no distinction was drawn between a treated and untreated lift joint. As is observed elsewhere, the cohesion assumed in the design of the Dam was higher than that baseline of 0.02 f'_c MPa set out in the 1991 ANCOLD Guidelines; however, that relaxation from the conservative assumption was not justified by test results indicating that the higher value could be achieved with the LCRCC mix for this Dam.

USACE RCC Manual

5.513 One of the industry standards to which Dr Schrader referred as supportive of the design parameters he advised the Alliance about was the USACE RCC Manual.[772] The Manual spoke of shear strength testing at the design stage, at the trial embankment stage and post-construction.

5.514 Under the heading 'Shear strength', the USACE RCC Manual provided:[773]

> *(2) Lift joint shear strength (from cores). The shear strength at the lift joints is generally the critical value for design. … McLean and Pierce (1988) found that use of Ø = 45 deg for preliminary design was generally conservative, while use of c = 0.1 f'_c was unconservative, due partly to the natural variation of all strength properties. For unbedded lift joints, c / f'_c has varied from 0.03 to 0.06. For bedded lift joints, c / f'_c has varied from 0.09 to 0.15. Friction angle for bedded and unbedded lift joints has been essentially unchanged. … A preliminary design value of c = 0.05 f'_c is recommended for lift joint surfaces that are to receive a mortar bedding; otherwise, a value of 0 should be assumed. A value of Ø = 45 deg can be assumed for preliminary design or for small projects, for both parent and lift joint shear strength. Design values should also take into account the*

770 Exhibit 33, **ACD.003.0001**, .0014 (emphasis added).

771 Exhibit 33, **ACD.003.0001**, .0014 (emphasis added).

772 US Army Corps of Engineers, *Roller-Compacted Concrete: EM 1110-2-2006* (2000), accessed on 12 April 2020 <www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-2006.pdf> (**USACE RCC Manual**).

773 US Army Corps of Engineers, *Roller-Compacted Concrete: EM 1110-2-2006* (2000), accessed on 12 April 2020 <www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-2006.pdf>, p4-6 – 4-7 (emphasis added).

> *expected percentage of the joint which will be adequately bonded, as indicated by the testing of cores from test sections and later from the completed structure.* ***Assumed values must be verified for final design by tests performed on samples prepared in the lab and on cores taken from test fills.***

5.515 Some of this detail was repeated in section 5.2 headed 'Special Structural Design Requirements for RCC Gravity Dams':[774]

> *c. Minimum sliding factors of safety for RCC gravity dams. … [B]ecause of the uncertainties and variability of cohesive strength at RCC lift joint surfaces, the selection of cohesive strengths used in sliding analyses must be made carefully. A preliminary cohesion design value of 5 percent of the compressive strength is recommended for lift joint surfaces that are to receive a bedding mortar; otherwise, a value of 0 should be assumed. The angle of internal friction can vary from 40 to 60 deg. A value of 45 deg may be assumed for preliminary design studies.* ***Assumed values must be verified by tests performed on samples prepared during laboratory design of RCC mixtures and on cores taken from design stage test sections. These tests must demonstrate that the shear resistance of a typical lift joint meets or exceeds the design requirements.*** *Some minor increases in shear resistance can be achieved by sloping lift surfaces down from downstream to upstream. Requiring inclined lift surfaces is not recommended if the primary goal is to improve shear resistance.*

5.516 The USACE RCC Manual is noteworthy for two reasons. First, it required that samples of the RCC be subjected to verification testing. The results of those tests were required to demonstrate that the shear resistance of a typical lift joint met or exceeded the design values. Secondly, the Manual set out the shear strength parameters that could be assumed in preliminary design studies, before verification tests had been undertaken. The approach is similar to the 1991 ANCOLD Guidelines because conservative bases for design values were recommended unless testing proved that more robust assumptions were appropriate.

5.517 Dr Schrader's recommended design value for friction of 45º was consistent with the USACE RCC Manual. However, that manual recommended preliminary design values for cohesion of:

a. 5% compressive strength for a bedded lift joint, i.e. 700 kPa (0.05 x 14 MPa) compared to Dr Schrader's recommended probable value of 2,800 kPa for an excellent lift joint (which was reduced by the Alliance to 2,400 kPa for a good quality lift joint)

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774 US Army Corps of Engineers, *Roller-Compacted Concrete: EM 1110-2-2006* (2000), accessed on 17 April 2020 <www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-2006.pdf>, p5-2 (emphasis added).

b. 0 kPa for an unbedded lift joint, compared to Dr Schrader's recommended design value of 400 kPa for an excellent quality lift joint (reduced to 325 kPa by the Alliance for a good quality lift joint).

5.518 By comparison to the assumed values recommended in the USACE RCC Manual, Dr Schrader's design values for cohesion were quite high.

5.519 The USACE RCC Manual also provided the following in respect of a post-construction drilling program:[775]

> *Samples of RCC can be obtained from coring in order to determine the in situ properties. This provides the best evidence of concrete performance by providing samples for strength and density determination, for viewing the density matrix from top to bottom of the lifts, and for identifying lift joint bond or lack of bond.* ***The primary purpose for obtaining intact lift joints is to determine the performance of shear and tensile strength properties in relation to those used for design.*** *Generally, coring is performed upon completion of the RCC structure. It can also be performed during planned cold joints such as during the planned gallery construction. Skid-mounted or truck-mounted hydraulic coring rigs have successfully obtained intact RCC and foundation cores. Conventional core barrels with a split inner barrel, about 1.5 m (5 ft) in length and 155 mm (6 in.) in diameter, are commonly used for RCC core sampling. Some core breakage occurs where weak lift joints shear during coring. Experienced and careful drillers typically have greater core recovery with intact lift joints. The use of a polymer drilling fluid has also improved recovery of lift joints.*

5.520 Unlike the mandatory language in which testing at the design stage was described, the Manual stated that post-construction testing *could* be done to compare the achieved shear strength properties to those used in design.

ACI Report

5.521 In effect at the time the Dam was designed was the '*Roller-Compacted Mass Concrete: ACI 207.5R-99*' reported by American Concrete Institute Committee 207 (**ACI Report**). That report recognised that the design strength of an RCC dam '*is usually not determined by the compressive stresses in the structure, but is more dependent on the required tensile strength, shear strength, and durability*'.[776] The

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775 US Army Corps of Engineers, *Roller-Compacted Concrete: EM 1110-2-2006* (2000), accessed on 17 April 2020 <www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-2006.pdf>, p7-10 (emphasis added).

776 American Concrete Institute, *Roller-Compacted Mass Concrete: ACI 207.5R-99*, ACI Committee 207 (1999), accessed on 12 April 2020 <http://dl.mycivil.ir/dozanani/ACI/ACI%20207.5R-99%20Roller-Compacted%20Mass%20Concrete_MyCivil.ir.pdf>, p8 (2.3.2).

ACI Report provided guidance about the starting values for initial planning and design purposes:[777]

> *As with any dam design, the designer of RCC structures should be confident that design assumptions are realistically achievable with the construction conditions anticipated and the materials available. …* ***For initial planning and design purposes, a value of cohesion of 5 percent of the design compressive strength with a coefficient of friction of 1.0 (corresponding to a ϕ angle of 45 deg) is generally used****.*

5.522 On the basis of an untreated lift joint, and assuming a compressive strength of the RCC of 14 MPa, the cohesion design values for the Dam of 325 kPa for a good joint and 400 kPa for an excellent joint were within the guidance (i.e. less than 700 kPa = 0.05 x 14,000 kPa). However the values for a lift joint treated with bedding mix (2,400 kPa for a good joint and 2,800 kPa for excellent) were well beyond the recommended limit. However, it is not apparent whether the guidance in the ACI Report was for treated or untreated lifts.

5.523 The ACI Report went on to explain that design values could be determined in several ways, where all the ways were a type of test that might be conducted on the RCC mix to be used. The ACI Report proposed that design values be determined by testing:[778]

> *Design values for tensile and shear strength parameters at lift joints can be determined in several ways. Drilled cores can be removed from RCC test placements and tested in shear and direct tension. Individual specimens can be laboratory fabricated and similarly tested if the mixture is of a consistency and the aggregate is of a size that permits representative individual samples to be fabricated. At a number of RCC projects, joint shear tests have been performed on a series of large blocks of the total RCC mixture cut from test placements compacted with walk behind rollers. Various joint maturities and surface conditions of the actual mixture for the project are evaluated and used to confirm or modify the design and construction controls. In-situ direct shear tests have been performed at various confining loads on blocks cut from field test placements made with full production equipment and field personnel.*

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777 American Concrete Institute, *Roller-Compacted Mass Concrete: ACI 207.5R-99*, ACI Committee 207 (1999), accessed on 12 April 2020 <http://dl.mycivil.ir/dozanani/ACI/ACI%20207.5R-99%20Roller-Compacted%20Mass%20Concrete_MyCivil.ir.pdf>, p20 (4.3.2) (emphasis added).

778 American Concrete Institute, *Roller-Compacted Mass Concrete: ACI 207.5R-99*, ACI Committee 207 (1999), accessed on 12 April 2020 <http://dl.mycivil.ir/dozanani/ACI/ACI%20207.5R-99%20Roller-Compacted%20Mass%20Concrete_MyCivil.ir.pdf>, p40 (4.3.3).

5.524 The ACI Report stated that confirmation testing should be done for high dams and for those dams (like the Dam) where joint shear strength was critical to stability and safety:[779]

> *High dams, and those* ***where joint shear strength is critical to stability and safety, should have design assumptions for joint shear strength confirmed with shear tests of the RCC to be used, the conditions to be encountered, and the construction controls that will be enforced****. Initial design assumptions can be based on extrapolation from tests, evaluations, and successful design assumptions from previous projects.*

2003 ICOLD Bulletin 126

5.525 The 2003 ICOLD Bulletin 126 – *RCC Dams, State of the Art and Case Histories*[780] – said of selecting values to be used in final designs and verification testing:[781]

> *Given the layered form of construction of RCC, the strength of lift joints and the potential for sliding on lift joints must be considered carefully. With high cementitious content RCC, good cohesion is achievable, but low-cementitious RCC and RCCs that segregate can have low cohesion …* ***Actual values used in final designs should be based on tests of the materials to be used or careful extrapolation from tests on RCC mixtures from other projects with similar aggregates, cementitious material contents, and aggregate gradings. As with any dam design, the Designer of RCC structures must be sure that design assumptions are realistically achievable with the construction conditions anticipated and the materials available.***
>
> *…*
>
> ***For final design, values for tensile and shear strength parameters at lift joints can be determined in several ways.*** *Some examples (in order of preference) are :*
>
> 1. *In-situ direct shear tests can be conducted at various confining loads on blocks cut into full-scale trials made with full production equipment and site personnel.*
> 2. *Drilled cores can be removed from RCC full-scale trials and tested in shear and direct tension.*
> 3. *Joint shear tests can be performed on a series of large blocks of the total RCC mixture cut from test placements compacted with small, walk-behind rollers under laboratory conditions. Various joint maturities and surface*

779 American Concrete Institute, *Roller-Compacted Mass Concrete: ACI 207.5R-99*, ACI Committee 207 (1999), accessed on 12 April 2020 <http://dl.mycivil.ir/dozanani/ACI/ACI%20207.5R-99%20Roller-Compacted%20Mass%20Concrete_MyCivil.ir.pdf>, p32 (5.6.1) (emphasis added).

780 Exhibit 68, **ICO.001.0001**.

781 Exhibit 68, **ICO.001.0001**, .0061, .0063 (emphasis added).

conditions of the actual mixture for the project can be evaluated and used to confirm or modify the design and construction criteria.

4. *Individual specimens can be manufactured and tested in the laboratory. Nevertheless for these tests to be valid the mixture should be of a consistency, and the aggregate of a size, that permits representative individual samples to be made. It is very difficult to simulate joint conditions in the laboratory.*

For high and medium-height RCC dams, full-scale trials are strongly recommended. These trials must be designed specifically for a particular project.
…

5.526 The 2003 ICOLD Bulletin saw full scale trials as an essential part of the quality control program, recommending that a full-scale trial section be constructed before placing RCC in the dam proper on all projects except the 'smallest dams'.[782] It was said that '*verification of the full-scale trial may entail obtaining concrete cores by diamond drilling for verification of concrete and joint properties (density, strength and modulus). The holes so drilled in the test fill can be tested for permeability and joint quality. Larger cross-sections of the test fill can be cut with excavation equipment or by sawing. The blocks of RCC can be tested for in-situ lift joint strength*'.[783]

5.527 The 2003 ICOLD Bulletin included a table showing the '*Coefficient of Variation*'[784] of various tests that were associated with perceived level of quality control for an RCC dam. Mr Lopez included a copy of the table in the RCC QC Reports.[785] The types of tests against which a coefficient of variation could be read included shear strength tests for unjointed, jointed and unbonded lift joints.[786]

5.528 Although the 2003 ICOLD Bulletin did not give guidance on design values, it did discuss the results of testing on cores taken from RCC dams. The following table and discussion appears at page 213:[787]

782 Exhibit 68, **ICO.001.0001**, .0195.
783 Exhibit 68, **ICO.001.0001**, .0197.
784 The coefficient of variation is a normalized measure of the expected spread or dispersion of a variable. It is formally defined as the standard deviation divided by the mean of a set of values and is often expressed as a percentage.
785 Exhibit 38, **SUN.110.003.0001**, .0056.
786 Exhibit 68, **ICO.001.0001**, .0207.
787 Exhibit 68, **ICO.001.0001**, .0213.

Table 11
Range of in-situ performance of RCC dams from the testing of cores

	Low-cementitious	RCD	Medium-cementitious	High-cementitious
Compressive strength (MPa)				
Nº	7	24	21	31
Mean	11.6	17.3	15.2	20.7
Range (1)	5 to 15	12 to 25	10 to 25	15 to 30
Direct tensile strength across joints (MPa)				
Nº	3	1	1	5
Mean	0.35	-	-	1.35
Range (1)	0 to 0.7	0.8 to 1.8 (2)	0.3 to 1.0 (2)	0.8 to 1.8
Cohesion at joints (MPa)				
Nº	3	5	8	8
Mean	0.70	2.40	0.90	1.90
Range (1)	0 to 1.5	1.5 to 4.0	0.5 to 1.8	1.0 to 4.0
Permeability (m/s)				
Nº	4	7	15	20
Range (1)	10^{-4} to 10^{-5}	10^{-8} to 10^{-9}	10^{-5} to 10^{-8}	10^{-7} to 10^{-12}

Notes : 1. Extreme values are not included in these ranges
2. Interpolated from other results

It is also apparent that the performance at the joints increases significantly as the cementitious content increases, for example with low-cementitious RCC dams the average cohesion is approximately 6 % of the compressive strength, while with high-cementitious content RCC dams it is over 9 %. In RCD dams, where very great care is taken with the joint treatment, the cohesion is over 13 % of the average compressive strength.

Portland Cement Association Design Guidelines

5.529 In November 2003, the 'Fourth International Symposium on Roller Compacted Concrete (RCC) Dams' was held in Spain. One of the papers delivered was by R A Kline, the principal author of the Portland Cement Association's '*Guidelines for the Design and Construction of RCC Lift Joints*' (**Portland Cement Guidelines**). Those Guidelines were yet to be published. The paper provided a synopsis of their content.[788] The paper was published in the proceedings of the conference.[789]

788 **PDI.045.0001**.

789 Spanish National Committed on Large Dams (SPANCOLD), Spanish Institute of Cement and its Applications (IECA), Chinese National Committee on Large Dams (CHINCOLD), *Proceedings of the Fourth International Symposium on Roller Compacted Concrete (RCC) Dams, 17-19 November 2003, Madrid, Spain: Roller Compacted Concrete Dams* (2006) 2nd print, accessed on 18 April 2020 <https://play.google.com/books/reader?id=tJdYDwAAQBAJ&pg=GBS.PP5> at p 898.

5.530 The paper gave the following explanation about what the PCA Guidelines would provide by way of testing during an RCC project:[790]

> ***2.1 Testing program guidelines***
>
> ***For high RCC gravity dams or gravity dams in which shear and/or tensile strength of parent RCC and lift joints are critical design factors, values used for design should be confirmed by a testing program. For smaller projects where a testing program is cost prohibitive, a more conservative design approach may be acceptable to use as a substitute.*** *Testing programs can be performed either during design or in the early stage of construction, keeping in mind that sufficient time is usually needed to obtain testing results at later ages (i.e. 90-day to 365-day). If a testing program is considered necessary, it should be developed, to the extent practical, to simulate anticipated construction materials and conditions. Simulating a range of possible or worst-case conditions is also likely to be useful. With specific regard to lift joint and parent RCC strength properties, this would include such items as possible or actual materials sources; RCC and bedding mix proportions; placing, spreading, and compaction methods and equipment; varying lift joint exposure times, treatments, and cleaning methods; and varying daily and seasonal weather conditions.* ***Following completion of construction, a second testing program involving core samples taken from the completed dam may be necessary and prudent to verify that actual parent and lift joint strength properties meet or exceed values used for design****.*

5.531 The paper said that shear strength testing could be done by laboratory-cast cylinders, drilled cores or sawn blocks from a test section.[791]

5.532 While the paper did not recommended design values, it did say that a friction only assessment of stability should be carried out:[792]

> *Assuming that each lift joint will be fully bonded throughout the entire structure is realistically not valid. Therefore,* ***the stability analysis should include a parametric study to evaluate various percentages of the total lift area having no cohesion in combination with a minimum internal friction angle when evaluating shear strength requirements.***

2005 USBR Guidelines

5.533 In 2005, the USBR published the first edition of guidelines called 'Roller-Compacted Concrete - Design and Construction Considerations for Hydraulic Structures' (**2005 USBR Guidelines**). It provided the following about shear testing:[793]

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790 **PDI.045.0001**, .0010 (emphasis added).

791 **PDI.045.0001**, .0012-.0013.

792 **PDI.045.0001**, .0009 (emphasis added).

793 USBR, *Design and Construction Considerations for Hydraulic Structures Roller-Compacted Concrete*, 1nd ed (2005) 20 [4.3] (emphasis in original).

> *Because it is generally necessary to maintain true 'cohesion' for meeting required factors of safety, the following discussion is directed at the strength properties of* ***bonded*** *lift lines and the* ***percentage*** *of any horizontal lift surface bonded. The* ***percentage*** *of a lift surface bonded is normally determined by coring through multiple lifts of concrete and examining individual joints. The coring program may be designed to examine multiple lifts from a few locations or a few lifts from many locations, depending on the intent of the test program, thickness of the placement, drilling equipment used, and accessibility of the site. Bonded and disbonded lift lines are identified and counted. Lift lines that are mechanically broken by the coring operation are not considered 'disbonded'. Determining the percentage of bonded lift lines requires the examination of drilled cores to be performed carefully eliminate those defects caused by the drilling process.*

5.534 The method for determining shear strength at RCC lift lines was then described. Further in the publication, a means of verifying bonding on lift joints was described:[794]

> ***Bond on lift joints is generally verified with core drilling and testing of concrete from RCC test sections or the actual RCC placements.*** *Core drilling cannot be done on RCC until the concrete obtains a compressive strength of about 1,000 lb/in^2 [Approximately 7 MPa]. Since the concrete continues to gain strengths, bond on lift joints also continues to improve. A quality assurance program over a year after construction of an RCC structure may assist in determining the overall performance of the bonding on lift joints.*

5.535 Values of cohesion and internal friction 'may be determined' by actual tests of the concrete to be used in the dam.[795] For preliminary design purposes, the USBR recommended a friction angle of 40º and apparent cohesion of 50 lb/in^2 (which converts to approximately 345 kPa) over the entire surface area.[796]

Dr Schrader's article

5.536 In 1999, Dr Schrader warned against using published data based on conditions at one project as an '*absolute basis for what will occur at another project*'.[797] His 1999 article about the LJQI recommended that absolute values for a project be determined by testing:[798]

> *When site-specific tests cannot be conducted, an approximate lift joint shear strength can be developed using information from other projects in combination*

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794 USBR, *Design and Construction Considerations for Hydraulic Structures Roller-Compacted Concrete*, 1nd ed (2005) 39 [5.11] (emphasis added).

795 USBR, *Design and Construction Considerations for Hydraulic Structures Roller-Compacted Concrete*, 1nd ed (2005) 45 [6.3].

796 USBR, *Design and Construction Considerations for Hydraulic Structures Roller-Compacted Concrete*, 1nd ed (2005) 46 [6.3].

797 Exhibit 124, **PDI.040.0001**, .0004.

798 Exhibit 124, **PDI.040.0001**, .0004 - .0005.

with knowledge of the mixture, aggregates, and other materials proposed for the new project, but absolute values should come from testing of the specific mixture and conditions in question.

When should testing have been done?

Standards and engineering practice justified shear strength testing

5.537 Four witnesses with expertise in RCC consider that confirmation testing for shear strength is typically conducted:

a. Dr Rizzo: '*I like to have a high confidence level in the design parameters that are used in the design, particularly the shear strength parameters, particularly the phi angle. So as a confirmatory matter, I would like to know from laboratory tests of samples extracted from the dam, preferably from the dam, and tested with shear apparatus, that my design is consistent with the parameters that I originally instructed and that those parameters were in fact achieved during construction.*'[799]

b. Mr Dolen: '*It has been a practice on most of the structures that I have been associated with, or major structures, that we would do a post-construction assessment to assure that the design values have been achieved.*'[800]

c. Mr Tarbox: '*[A]ll dams, when they are completed, should be sampled and tested as a matter of good engineering practice*'.[801]

d. Mr Brigden, when asked whether there might be a shear strength test in the trial mix stage: '*Yes, in my opinion, to have the suite of values that you're using for your design to check with your mix, then put it to test in the trial bank - so shear strength, [compressive] strength, tensile strength*'.[802]

5.538 When the Dam was designed several industry guidelines strongly encouraged confirmation testing for shear strength:

a. The 1991 ANCOLD Guidelines said that shear strengths at RCC lift joints should be assessed through testing.[803] Conservative design assumptions could be relaxed if justified by testing.[804]

b. The USACE RCC Manual said that assumed design values must be verified by testing of samples prepared in the laboratory and on cores taken from test

799 **TRA.500.008.0001**, .0012 ln 38-46.
800 **TRA.500.008.0001**, .0010 ln 30-34.
801 **TRA.500.007.0001**, .0038 ln 15-17.
802 Exhibit 48, **TRA.510.025.0001**, .0025 ln 20-27.
803 Exhibit 33, **ACD.003.0001**, .0014 (emphasis added).
804 Exhibit 33, **ACD.003.0001**, .0006, .0015.

sections.[805] That testing must demonstrate that the shear resistance of a typical lift joint exceeded the design requirements.[806]

c. The ACI Report said that dams where joint shear strength was critical to stability and safety should have design assumptions for joint shear strength confirmed with shear tests.[807]

5.539 Other guidelines, while not requiring shear strength testing, recommended that it be done:

a. The 2003 ICOLD Bulletin said that design values *should* be based on testing of the materials to be used or by correlation to RCC mixes from other projects with similar features. The Bulletin did say though that a designer *must* be sure that the design assumptions were reliably achievable.[808]

b. The Portland Cement Guidelines added that for gravity dams in which shear strengths of parent RCC and lift joints are critical design factors, design values *should* be confirmed by testing. The Guidelines went on to say that if testing was cost prohibitive for a smaller project, a more conservative design approach might be acceptable.

c. The 2005 USBR Guidelines said that bonds on lift joints were 'generally verified' with core drilling from a test section or the dam proper.[809]

5.540 In addition to that industry guidance supporting confirmation testing, Dr Schrader's 1999 article warned against using correlation to historical data as an 'absolute basis' for values on a project. Those values should come from site specific testing.[810]

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805 US Army Corps of Engineers, *Roller-Compacted Concrete: EM 1110-2-2006* (2000), accessed on 12 April 2020 <www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-2006.pdf>, p4-6 – 4-7.

806 US Army Corps of Engineers, *Roller-Compacted Concrete: EM 1110-2-2006* (2000), accessed on 17 April 2020 <www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-2006.pdf>, 5-2.

807 American Concrete Institute, *Roller-Compacted Mass Concrete: ACI 207.5R-99*, ACI Committee 207 (1999), accessed on 12 April 2020 <http://dl.mycivil.ir/dozanani/ACI/ACI%20207.5R-99%20Roller-Compacted%20Mass%20Concrete_MyCivil.ir.pdf>, p32 (5.6.1) .

808 Exhibit 68, **ICO.001.0001**, .0063.

809 USBR, *Design and Construction Considerations for Hydraulic Structures Roller-Compacted Concrete*, 1nd ed (2005) 39 [5.11].

810 Exhibit 124, **PDI.040.0001**, .0004-.0005.

5.541 Hydro Tasmania submitted that even had Mr Herweynen decided to conduct shear strength testing, '*it would not have been practically possible to obtain reliable shear strength tests or to halt the construction schedule of the Dam for a year before the tests results were obtained*'.[811] The submission apparently misunderstands the process of confirmation testing, which might be carried out on a test section once the RCC matures sufficiently or after construction has finished. So much is clear from the 2005 USBR Guidelines that say a quality assurance program over a year after construction may assist in determining the performance of lift joints. Confirmation testing does not delay construction. Its purpose is not to derive updated design values before a dam is built. Rather, confirmation testing verifies whether the Dam as built has achieved the original design values. If not, remedial works can be undertaken. But without those test results, there is no assurance that the design values have been achieved.

5.542 LCRCC is less amenable to testing to confirm that the design shear strength has been achieved. However, testing can be done using cores or larger shear blocks. In any event, that difficulty was inherent in the choice to use LCRCC. Indeed, because LCRCC is more susceptible to segregation than HCRCC and has less cementitious content to promote bonding across lift joints, confirmation testing was all the more important.

Cohesion values were not conservative

5.543 All four RCC experts said that post-construction testing should be done if there was reason to doubt that the design values had been achieved. Mr Tatro said that such doubt might arise if the design parameters were 'extraordinarily high'.[812] Mr Tatro said:[813]

> *[I]f shear performance is not extraordinary,* ***very low cohesions, low to no cohesions****, reasonable phi angles, confirmation testing isn't done, and that's not a bad thing.*

5.544 That view is consistent with the approach taken by the 1991 ANCOLD Guidelines and the Portland Cement Guidelines, which was to adopt a more conservative design approach if testing was not to be done (including because of financial limitations on a small project). While the Alliance's design values for friction were consistent with industry baseline assumptions, the values for cohesion were not conservative – they were not very low to low[814] – when compared with benchmarks that were available when the Dam was designed.

5.545 Based on Dr Schrader's advice, the friction angles that were adopted in the design for a 'good' untreated lift joint was approximately 40.5º. The angle for a good treated lift joint was 42º. The range of 40º to 45º was typically suggested as a conservative

811 **HYT.008.0001.0054**, .0054.
812 **TRA.500.008.0001**, .0009 ln 31-39.
813 **TRA.500.008.0001**, .0009 ln 3-6 (emphasis added).
814 **TRA.500.008.0001**, .0009 ln 3-6.

baseline assumption in industry guidance.[815] But the cohesion values warrant separate consideration.

5.546 The design cohesion value for a 'good' lift joint was specified as 325 kPa without bedding mix treatment and 2,400 kPa with bedding mix.[816] In November 2004, engineers involved in the SunWater due diligence remarked that the adopted cohesion values were 'particularly high'.[817] That comment was made with reference to the 1991 ANCOLD Guidelines, which said that, without testing, the peak effective cohesion should be 2% of compressive strength. The Alliance used a design value for compressive strength of 14 MPa. Therefore, on the ANCOLD guidance, a conservative cohesion value was 280 kPa (2% x 14 MPa). There was no testing to support the Alliance's higher cohesion values.

5.547 Later in 2014, the first TRP considered that the cohesion value for an untreated lift joint was reasonable. However:[818]

> *The increase in the 'cohesion' component assumed by BWA for a surface with bedding mix does seem too high. On the lift joint strength plots in EPRI, 1992, which represent the results of testing on generally 150 mm diameter specimen from US dams taken over a large number of years, the 45º/2 400 kPa strength parameters would plot well above the lower bound zone for the normal effective stress range for a dam like Paradise dam.*

5.548 Indications from the guidelines at the time were also that the cohesion values were high. The table below summarises the conservative design values recommended by a range of guidelines available when the Dam was designed. The 2005 USBR Guidelines were published after the Dam was designed but before the corehole was taken in 2006. So that guidance was available when Mr Herweynen corresponded with Dr Schrader about whether to conduct shear strength testing on the retrieved core.

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815 Exhibit 33, **ACD.003.0001**, .0014; US Army Corps of Engineers, *Roller-Compacted Concrete: EM 1110-2-2006* (2000), accessed on 12 April 2020 <www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-2006.pdf>, p4-6 – 4-7, 5-2; American Concrete Institute, *Roller-Compacted Mass Concrete: ACI 207.5R-99*, ACI Committee 207 (1999), accessed on 12 April 2020 <http://dl.mycivil.ir/dozanani/ACI/ACI%20207.5R-99%20Roller-Compacted%20Mass%20Concrete_MyCivil.ir.pdf>, p20 (4.3.2); USBR, *Design and Construction Considerations for Hydraulic Structures Roller-Compacted Concrete*, 1nd ed (2005) 46 [6.3(b)].

816 Exhibit 24, **GHD.002.0001**, .0140, .0141.

817 Exhibit 54, **SUN.016.014.1266**, .1270.

818 Exhibit 9, **IGE.019.0001**, .0034.

Guidance	Untreated lift joint (c.f. 325 kPa[819])	Treated lift joint (c.f. 2400 kPa[820])
1991 ANCOLD Guidelines[821]	0.02 f'_c = 280 kPa	
USACE RCC Manual[822]	0 kPa	0.05 f'_c = 700 kPa
ACI Report[823]	0.05 f'_c = 700 kPa	
2005 USBR Guidelines[824]	345 kPa	

5.549 In recommending design values for cohesion, no distinction was made between a treated and an untreated lift joint in the 1991 ANCOLD Guidelines, the ACI Report[825] or 2005 USBR Guidelines.[826] Where the guidelines do not distinguish between a treated and an untreated lift joint, it is assumed in the table above that the guidance is for an untreated lift joint.

5.550 The 2003 ICOLD Bulletin is not listed in the table above. In discussing historical data from RCC cores, the Bulletin said that cohesion was typically 0.06 f'_c (840 kPa).[827] However, those historical results were unlikely to have been intended as a guide to conservative values appropriate for designing a dam. That is because other publications recommended conservative design values that were below published test results:

a. The USACE RCC Manual cited an article that reported test results for cohesion as varying from 0.03 to 0.06 f'_c for unbedded lift joints. However, the Manual went on to recommend a design value of 0 kPa.[828]

819 Exhibit 24, **GHD.002.0001**, .0140 - .0141.
820 Exhibit 24, **GHD.002.0001**, .0140 - .0141.
821 Exhibit 33, **ACD.003.0001**, .0014.
822 US Army Corps of Engineers, *Roller-Compacted Concrete: EM 1110-2-2006* (2000), accessed on 17 April 2020 <www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-2006.pdf>, 4-7, 5-2.
823 American Concrete Institute, *Roller-Compacted Mass Concrete: ACI 207.5R-99*, ACI Committee 207 (1999), accessed on 12 April 2020 <http://dl.mycivil.ir/dozanani/ACI/ACI%20207.5R-99%20Roller-Compacted%20Mass%20Concrete_MyCivil.ir.pdf>, p20 (4.3.2).
824 USBR, *Design and Construction Considerations for Hydraulic Structures Roller-Compacted Concrete*, 1nd ed (2005) 46 [6.3(b)].
825 American Concrete Institute, *Roller-Compacted Mass Concrete: ACI 207.5R-99*, ACI Committee 207 (1999), accessed on 12 April 2020 <http://dl.mycivil.ir/dozanani/ACI/ACI%20207.5R-99%20Roller-Compacted%20Mass%20Concrete_MyCivil.ir.pdf>, p20 (4.3.2).
826 USBR, *Design and Construction Considerations for Hydraulic Structures Roller-Compacted Concrete*, 1nd ed (2005) 46 [6.3].
827 Exhibit 68, **ICO.001.0001**, .0213.
828 US Army Corps of Engineers, *Roller-Compacted Concrete: EM 1110-2-2006* (2000), accessed on 12 April 2020

b. Similarly, the ACI Report recommended cohesion of 0.05 f'_c for design purposes[829] despite presenting RCC test data where the unconfined shear strength of an *unjointed* section of RCC varied from 16% to 39% of its compressive strength.[830] Dr Schrader relied on the reported test results for unjointed RCC (rather than the 5% guidance for cohesion at a lift joint) in saying that the design value that he recommended for cohesion was conservative.[831]

5.551 The 2003 ICOLD Bulletin is, therefore, not taken to have recommended a design figure for lift joint cohesion simply by reporting on historical test results.

5.552 As the table above demonstrates, the cohesion value adopted for untreated lift joints exceeded two of the guidelines. In the USACE RCC Manual, the recommended conservative approach was a friction only assessment of stability. That aligns with the approach of Dr Rizzo. He relies on friction (and does not have regard to cohesion) when designing an RCC dam.[832] Mr Dolen's view was that it was '*impractical*' to calculate cohesion derived from the upstream portion of lift joints treated with bedding mix if the remainder of the lift joint was unbonded.[833] Dr Schrader disagreed saying that the calculation was done '*all the time*'.[834] Mr Tatro was also familiar with that approach being used,[835] as was Mr Tarbox.[836]

5.553 In relation to the design value for a treated lift joint, limited guidance was provided in the industry standards at the time. The recommendation in the USACE RCC Manual of 700 kPa was exceeded (by a wide margin) by the design value that the Alliance adopted. The first TRP also considered that the design value was outside the bounds of data presented in EPRI 1992. The cohesion value for a treated lift joint in the Dam was very high by available benchmarks.

<www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-2006.pdf>, p4-6 – 4-7 (emphasis added).

829 American Concrete Institute, *Roller-Compacted Mass Concrete: ACI 207.5R-99*, ACI Committee 207 (1999), accessed on 12 April 2020 <http://dl.mycivil.ir/dozanani/ACI/ACI%20207.5R-99%20Roller-Compacted%20Mass%20Concrete_MyCivil.ir.pdf>, p20 (4.3.2).

830 American Concrete Institute, *Roller-Compacted Mass Concrete: ACI 207.5R-99*, ACI Committee 207 (1999), accessed on 12 April 2020 <http://dl.mycivil.ir/dozanani/ACI/ACI%20207.5R-99%20Roller-Compacted%20Mass%20Concrete_MyCivil.ir.pdf>, p15 (3.2.3).

831 Exhibit 109, **SCE.019.0001**, .0002.

832 **TRA.500.008.0001**, .0022, ln 40-42.

833 **TRA.500.008.0001**, .0035 ln 6-13.

834 **TRA.500.008.0001**, .0032 ln 18-19.

835 **TRA.500.008.0001**, .0036 ln 3-11.

836 **TRA.500.007.0001**, .0026 ln 23 to .0027 ln 21.

5.554 It is also worth noting comparisons that were available to the design team from the two stage 2 designs. The Alliance was looking at more conservative values at that stage:[837]

a. 100 kPa for a lift joint with no bedding mix (c.f. 325 kPa Detail Design)

b. 800 kPa for a lift with bedding mix (c.f. 2,400 kPa Detail Design).

5.555 Influenced by Dr Schrader's advice, the designers adopted Detail Design values substantially more than those stage 2 values.

5.556 The stage 2 design of the Thiess and URS consortium was another available comparator. Mr Griggs reviewed that design and drew a direct comparison between the shear strength values adopted. The result was a memorandum dated 7 November 2003[838] that Mr Griggs sent by email to Mr Herweynen, Mr Neumaier, Mr Brett and Mr Johnson.[839] It included the following table comparing the assumed RCC material properties of the two designs:[840]

Table 4 – Comparison of Material Properties

Material Parameter	*Team 1*	*Thiess/URS*	*Discussion of Issues*	*Action*
Foundation shear strength	c'=200 kPa and ϕ'=45° (Goodnight beds)	c'=350 kPa and ϕ'=55° (Goodnight beds – primary spillway) c'=220 kPa and ϕ'=50° (Goodnight beds – secondary spillway)	An increase in foundation design strength would allow us to further optimise the dam section, while still maintaining acceptable factors of safety.	The foundation design shear strength will be reviewed.
RCC lifts - shear strength	c'=100 kPa and ϕ'=45° (no bedding mix) c'=800 kPa and ϕ'=45° (with bedding mix)	c'=500 kPa and ϕ'=45°	The RCC properties are dependent on the mix design. The mixes are different and therefore not directly comparable. The mix design spec. produced by Ernie Schrader has detailed tables of expected material properties.	Undertake sensitivity analysis using parameters from mix design specification.

5.557 Mr Griggs's memorandum went on to say:[841]

> *The range of shear strength parameters given in the mix design specification depends on the age, quality of the joint and whether bedding mix is used. The*

837 Exhibit 88, **DNR.005.4886**, .5025.
838 Exhibit 88, **DNR.005.4886**, .5022.
839 **DNR.020.019.2555**, .2555.
840 Exhibit 88, **DNR.005.4886**, .5025.
841 Exhibit 88, **DNR.005.4886**, .5024.

table below provides the values that could be expected for a 65 kg/m^3 cement content mix - the range is from poor lift joint quality to excellent lift joint quality.

Table 5 – Range of Shear Strength Parameters from Team 1 Stage 2 Mix Design

Age	*No Bedding Mix*		*Bedding Mix*	
	Friction angle, ϕ'	*Cohesion, c'*	*Friction angle, ϕ'*	*Cohesion, c'*
7	24° - 36°	50 - 200 kPa	26° - 38°	280 – 1120 kPa
28	29° - 38°	125 - 300 kPa	31° - 40°	700 – 1680 kPa
90	31° - 40°	200 - 350 kPa	33° - 42°	1120 – 1960 kPa
180	33° - 41°	250 - 400 kPa	36° - 44°	1400 – 2240 kPa
365	36° - 45°	275 – 400 kPa	38° - 45°	1540 – 2240 kPa

5.558 The origin of the 'Team 1' shear strength values in table 4 is not clear. The values do not neatly align with anything in Table 5, although they are within the ranges for early age RCC.

5.559 As Table 4 above shows, the Thiess and URS consortium assumed effective cohesion of 500 kPa for lift joints. No distinction was made between treated and untreated joints,[842] presumably because the design used an HCRCC mix with 210 kg/m^3 of cementitious material[843] so that treatment with bedding mix at lift joints was not needed to promote bonding.

5.560 HCRCC is able to achieve more intimate bonding across lifts than LCRCC. HCRCC has excess paste in the mix which rises to the surface of the lift when compacted by the vibrating roller.[844] The 2003 ICOLD Bulletin said that '*Low-cementitious RCC has been found to have rather lower in-situ performance in terms of cohesion and direct tensile strength when compared with high-cementitious content concretes*'.[845] Mr Dolen's evidence was that:[846]

> *Low percentage of bonded lift joints and lower than assumed sliding friction resistance, and low sliding resistance due to voids in LCRCC dams have been documented as early as 1988 (Drahushak-Crow, 1988).*

5.561 The published literature at the time indicated that HCRCC would achieve greater cohesion across a lift joint than LCRCC. Dr Schrader's 1999 article about the LJQI said that, '*all other things being constant, increasing the cementitious content of RCC increases lift joint quality and strength*'.[847] That article also included the chart at Figure 5.16 above.[848]

5.562 Design cohesion would be expected to be higher for HCRCC than for LCRCC. By comparison to the Thiess and URS figure of 500 kPa, the Hydro Tasmania design initially assumed 20% of that for an untreated lift and 160% for a treated lift. By the

842 Exhibit 88, **DNR.005.4886**, .5024.
843 Exhibit 81, **DNR.007.1087**, .1111.
844 **TRA.500.002.0001**, .0005 ln 16-25.
845 Exhibit 68, **ICO.001.0001**, .0049.
846 Exhibit 104, **GHD.006.0001**, .0022.
847 Exhibit 124, **PDI.040.0001**, .0004 (emphasis added).
848 Exhibit 124, **PDI.040.0001**, .0012.

detail design report, the Alliance assumed 65% of the Thiess and URS figure for an untreated joint and 520% for a treated joint. That is despite the RCC having less than a third of the cementitious content of the Thiess and URS mix. These percentages are high in light of the comparative mixes.

5.563 It seems that rather than causing the designers concern that their cohesion values might be high, Mr Griggs said that the different properties could not be compared because the material properties were dependent on the mix design. That rationale contrasts with the approach taken by the Alliance to derive its design parameters, which relied on Dr Schrader's advice about the properties, based as it was on his database of historical RCC properties elsewhere.

5.564 The cohesion design values adopted by the Alliance were not conservative because:

a. these values more than tripled from the stage 2 design to the Detail Design

b. the value for a treated lift substantially exceeded the figure adopted by the Thiess and URS consortium for its HCRCC mix while the Hydro Tasmania value for an untreated lift joint was not substantially lower

c. the cohesion value for an untreated lift joint exceeded guidance from ANCOLD and the USACE (although was less than the values in the ACI Report and the 2005 USBR Guidelines, but those two guidelines did not distinguish between a treated and an untreated joint)

d. the cohesion value for a treated lift joint far exceeded all guidance

e. the design cohesion for a treated lift was high compared to the data in EPRI 1992.

Design values required confirmation testing

5.565 As the cohesion design parameters were so high relative to benchmarks in contemporary guidelines, the design team had reason to be cautious before deciding that the cohesion values were achievable. Mr Herweynen knew of no method to verify independently the design parameters advised by Dr Schrader.[849] That inability to interrogate Dr Schrader's advice should have been the impetus to require confirmation testing. Mr Herweynen said that he felt differently now about not having done shear testing and would change his mind about that in future.[850]

5.566 Dr Schrader repeatedly stated during the Dam's design and construction that the Dam achieved stability essentially on friction alone. That is, cohesion was essentially not needed. Far from being an answer to the criticism that the cohesion values were not conservative, indeed the opposite holds true. The Dam's stability relied on cohesion.[851] The 'worst case' scenario upon which the designers' sensitivity analysis

849 Exhibit 244, **HER.001.0001**, .0033 [157].
850 **TRA.500.013.0001**, .0057 ln 18-21.
851 **TRA.500.013.0001**, .0014 ln 17-37.

was based assumed cohesion of 250 kPa over the entire lift surface.[852] While cohesion may not have been needed in much of the Dam for stability, it was needed in critical zones of the Dam.[853]

5.567 For that reason, it was all the more important that the designer be certain that the assumed values could be achieved by the particular RCC and bedding mixes used to construct the Dam. That is what the prevailing industry guidance of the time provided. As Mr Griggs had stated, RCC mixes are different and therefore not directly comparable.

5.568 With accessible and reasonable comparators available to them, the design team should have recognised that shear strength testing was needed to ensure that the high cohesion values could be achieved by this RCC mix and the bedding mix proposed. As is Dr Rizzo's practice, a designer should have a '*high confidence level in the design parameters that are used in the design, particularly the shear strength parameters*'.[854]

5.569 The failure of the Alliance to conduct confirmatory shear strength testing is a root cause of the uncertainty that now attends the Dam's stability.

Would peer review have led to changes?

5.570 The peer review of the design did not extend to a review of the stability assessment, which was unfortunate.

5.571 It is probable that a proper peer review of the stability assessment would have identified that the cohesion values were relatively high and recommended that the values be reduced or else shear strength testing be conducted at the mix design or post-construction stages, or both.

5.572 The Alliance might not have acted on such advice about shear testing. When Mr Brett disagreed with Dr Schrader about the RCC mix, the Alliance preferred the advice of Dr Schrader.[855] However, a peer review was to be:

> *[C]arried out by suitably qualified and independent persons, to ensure that the following requirements [were] met*
>
> *(i) All regulatory requirements*
>
> *(ii) Safety considerations*
>
> *(iii) Functional and operational requirements*
>
> *…*

852 Exhibit 24, **GHD.002.0001**, .0156.
853 See, for example, **TRA.500.013.0001**, .0032 ln 5-9; **TRA.500.014.0001**, .0085 ln 14-18.
854 **TRA.500.008.0001**, .0012 ln 38-46.
855 Exhibit 266, **DNR.020.019.1037**, .1038.

5.573 So a proper peer review process required that the reviewer's advice be seriously considered. If, as is likely, the peer reviewer would have reported that the design values for cohesion were too high, there is a good chance that the designers would have implemented changes to address that concern.

5.574 That the Dam's stability assessment was not subjected to a proper peer review is a root cause of the current uncertainty about the Dam's stability.

Questions about the quality of lift joints

5.575 Dr Rizzo said that shear strength testing should be carried out if questions about the quality of the lifts joints raised doubts about whether the design values had been achieved. Mr Dolen agreed.[856]

5.576 For the reasons that follow, there were questions about the quality that had been attained in placing RCC in the Dam. These questions concern the RCC itself and the lift joints contained therein.

5.577 First, quality problems were identified during construction and it is not possible to find that all those problems were remedied. Where issues were raised in the construction memoranda that Mr Lopez and Mr Montalvo prepared, the issues were generally remedied. If they were not, the issues were localised and not sufficient to raise fundamental concerns with quality. However, there were only 80 construction memoranda. The principal tool for detecting quality issues with lift joints was the LJQI. The LJQI did not facilitate a reliable assessment of quality because of the previously identified defects with the system, which are summarised as follows.

5.578 Foremost among those doubts is the manner in which points were added to the LJQI score if bedding mix was applied. The usual response to a low LJQI score was to apply bedding mix and then increase the score with addition points. Adding points in that way did not accord with the Specification and was in addition to the in-built addition of 6 points for treating a cold joint with bedding mix (within the 'maturity' category). That practice masked the quality issues to which the LJQI was designed to draw a designer's attention. By increasing the score for bedding mix outside the specified application of the LJQI, the design team's attention was not drawn to any quality issues that were being identified. The data were affected by this approach.

5.579 The LJQI is assessed by inspecting the top surface of a layer but not the bottom of the layer immediately above, which if badly segregated with no paste, would be a weakness in the structure.[857] This is a serious shortcoming. Bonding between lifts depends on the quality of both the top of the preceding lift and of the lower part of the following lift. It is that interface where bonding should occur. For that reason, the 'Lift **Joint** Quality Index' might be more appropriately referred to as a 'Lift **Surface** Quality Index'.[858] It is limited to an assessment of the quality of the lift surface. While

856 **TRA.500.008.0001**, .0010 ln 21-24 (Dr Rizzo), .0010 ln 29-41 (Mr Dolen).

857 **TRA.500.002.0001**, .0029 ln 42 – .0030 ln 3; Exhibit 104, **GHD.006.0001**, .0009 [29], .0009-.0010 [32], .0013 [48].

858 **TRA.500.009.0001**, .0045 ln 6.

indications of the joint quality might be drawn from that of the surface, the LJQI is unable to assess the bottom of the overlying layer. The LJQI provides an assessment of only one half of the interface.

5.580 The evaluations inherent in the LJQI were subjective. Different inspectors could look at the same lift and arrive at a different score. Training was the means used to smooth those inconsistencies[859] but, even with training, the LJQI remained a '*measure of trying to calibrate people's eyes*'.[860] Assessing the LJQI suffered from the weakness that it was only as good as the judgements upon which the scoring was based.[861] The RCC Inspectors were responsible for completing the LJQI Scorecards. While the forms were designed to be countersigned by a delegate of the Alliance, the forms were not endorsed by the RCC Engineers at the time of RCC placement in many instances. Countersignatures of the RCC Engineers were not made for substantial periods after the relevant RCC had been placed and well after an opportunity to remedy any issues had been lost.

5.581 Because the qualitative assessments of the lift joints resulted in a numerical outcome, the scores (and statistics and charts based on them) took on the appearance of having a quantitative and scientific basis. That was not the case. LJQI evaluations were subjective and were only as good as the visual perception of the person making the evaluation.

5.582 Although the LJQI as a quality assurance tool was better than no framework for inspections, it was a complicated system. It is doubtful that the RCC Inspectors, even with best efforts, could meaningfully have made all the evaluations required by the RCC Placement ITPs and the LJQI Scorecards. There were thousands of sign off points for the RCC Inspectors across the course of the project. The problem of the difficulties faced by an inspecting engineer were recognised in an article written by Mr Forbes published in July 2003:[862]

> *In any RCC dam, occasional sub-standard work on lift joints will be inevitable due to high placement rates, the need for night work, and the urgency to prepare lift surfaces ahead of the rapidly advancing next RCC lift. The inspecting engineer will simply not be able to ensure that every lift joint is properly constructed, without inherent weaknesses.*

5.583 Even with the structured inspection regime which the LJQI provided, there could be no guarantee that the requisite lift joint quality would be achieved. Nor that the inspection regime itself would detect every issue because of the sheer amount of RCC placed rapidly across the project and, at times, simultaneously in multiple locations.

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859 **TRA.500.010.0001**, .0073 ln 4-26.
860 Exhibit 48, **TRA.510.025.0001**, .0033 ln 21-22.
861 **TRA.510.023.0001**, .0027 ln 35-40.
862 Exhibit 307, **SME.001.0001**, .0001- .0002.

5.584 The LJQI was the primary basis upon which lift joint quality was assessed. Both because of the LJQI's structural problems and because of the way the Alliance applied it on site, it did not provide reliable assurance of quality. In the result, questions arise about the quality attained in placing RCC in the Dam.

5.585 In addition to the construction memoranda and the LJQI Scorecards, other ways of raising quality issues were the RCC Placement ITPs, NCRs and discussions on site.[863] Regardless of how issues were raised, the application of bedding mix came to be used as the standard remedy. That was not a solution to all problems. For instance, when applied to an area of a lift that had not been properly cleaned, it was questionable whether bedding mix would improve the bond.

5.586 There were other problems that bedding mix would not seem to redress. To assess the surface flatness category on the LJQI, the RCC Inspectors observed how much of the surface had been compacted with the drum of the vibratory rollers. Low scores reflected insufficient contact of the roller drum. The degree of contact is associated with the degree of compaction that could have been achieved. If a roller did not compact part of the surface at all, compaction is unlikely to have been effective. If an LJQI score was low by virtue of poor surface flatness, that was an indication of possible segregation of the RCC lift. That would have ramifications for the sliding resistance of the lift joint below. Adding bedding mix to top of the RCC layer would not assist with that particular problem.

5.587 Bedding mix was not the cure-all that the Alliance seems to have assumed. Even if it were, it is not possible to determine definitively where bedding mix is located in the Dam. While an early version of the LJQI Scorecard form had a section to record the area covered with bedding mix, that section was blacked out in a later version. The batch supply dockets for bedding mix that accompany some RCC Placement ITPs do not always assist. Some are blank. The dockets describe the delivery point in general terms that do not align with the area of RCC being placed. Even if that link could be established, the delivery dockets do not show where on the lift surface bedding mix was placed.

5.588 The quality assurance and quality control system as a whole does not permit a finding that remedial measures were taken to address all problems during construction. This is consistent with Dr Schrader's suggestion towards the end of the project that lifts had been left *in situ* when they arguably should have been removed in accordance with the Specification.[864]

5.589 Secondly, satisfaction that quality issues did not arise or that any such issues were rectified was all the more important because an LCRCC mix was used. That choice meant that '*fairly rigorous adherence to good construction practices*' was required.[865] Mr Tarbox said that RCC construction methodologies have to be '*applied religiously*

863 **TRA.500.009.0001**, .0080 ln 34-38.
864 Exhibit 215, **SCE.023.0001**, .0001.
865 **TRA.510.006.0001**, .0036 at ln 36 – 42.

and continuously'.[866] '*Any time that these practices are not followed to the letter, you can end up with a flawed or weakened surface within your dam, which represents a potential instability*'.[867] Consistent with Mr Tarbox's view was the 2005 USBR Guideline that said:[868]

> *Any deviation from the approved construction materials or procedure can affect the dam's overall structural stability. For example, underbatching the cement during placement of a single lift of RCC, or failure to properly prepare a single lift joint, may limit the entire dam's sliding stability. Unfortunately, the influence of a single lift joint on sliding stability is greatest near the base of the dam, placed very early in the construction.*

5.590 The need to strictly adhere to prescribed methodologies was amplified by the characteristics of the particular LCRCC mix used here. A cement content of only 63 kg/m^3 was at the very lean end of the LCRCC spectrum. Mr Dolen said that of 906 dams listed in a published database,[869] only 12 had a cementitious content less than 65 kg/m^3 and none less than 60 kg/m^3.[870] By its nature, LCRCC is less able to achieve bonding across lift joints than HCRCC because of the lack of paste in the mix. With such low cement content as was used for the Dam, the ability to achieve bonding across lift joints was even further limited.

5.591 The mix also had a high workability requirement. The Specification referred to a '*modified*' VeBe time of about 20 to 30 seconds at a temperature of 20ºC. Even at that temperature, Mr Dolen thought the specification was high for a modern RCC dam.

5.592 Workability issues with the RCC were exacerbated by high temperatures which corresponded with its peak production and placement. The Alliance had initially proposed not placing RCC at all during summer of 2004/05. In its stage 1 proposal, the design and construction program incorporated a break in RCC placement from the start of December 2004 to the end of March 2005.[871] That break aligns with strategies to manage RCC constructability risks that were identified at stage 2. One such risk was that RCC placement may need to stop until temperatures fall. A preliminary management strategy was to '*[p]lace exclusively in winter*'.[872] This did not happen. Peak placement (including in the primary spillway) occurred in the warmer and wetter months. The Project Manager said at the time that RCC placement was

866 **TRA.500.007.0001**, .0021 ln 16-18.
867 **TRA.500.007.0001**, .0021 ln 22-25.
868 USBR, *Design and Construction Considerations for Hydraulic Structures Roller-Compacted Concrete*, 1nd ed (2005) 46 [6.3(a)].
869 Database of all RCC dams throughout the world maintained by Malcolm Dunstan & Associates, <http://www.rccdams.co.uk/>.
870 Exhibit 104, **GHD.006.0001**, .0010.
871 Exhibit 251, **HYT.510.004.0001**, .0126.
872 Exhibit 251, **HYT.510.004.0001**, .0081.

less efficient because it was conducted during the wet season, which caused extra work and delays.[873]

5.593 The combination of using a very low cementitious content mix that was difficult to work with and using that mix to build the primary spillway of a Dam during a Queensland summer gives rise to questions about the quality that could have been achieved.

5.594 The third indication that there were quality issues with lift joints is the fact that the cold joints were prevalent. Mr Lopez recalled that almost all the joints were cold joints.[874] Mr Montalvo agreed.[875] His view was that cold joints were not a problem. It was only bad construction practice to have an untreated cold joint.[876] There are two problems with that assertion. One is the present inability to verify that all cold joints were treated with the required amount of bedding mix. The second is the question mark over how much bedding mix was required to achieve stability. There is no evidence that a reasoned mathematical analysis was the basis for either the original specified amount or the relaxed requirement for cold joint treatment.

5.595 Mr Herweynen said that the schedule had not envisaged so many cold joints.[877] The Dam was not designed on the basis of that quality of lift joint. The worst case scenario for the stability assessment was a lift near the base of the primary spillway with 'poor' quality, which still assumed cohesion of 250 kPa without bedding mix.

5.596 Cold joints were distinct from poor quality lift joints. The Specification dealt with them separately. Methods to remedy a lift where the LJQI score was equal to or lower than -3 were provided in section 11.10.1 of the Specification. The treatment of cold joints was specified in sections 11.10.3 and 11.10.4. In the result, a poor lift joint did not necessarily require remediation while a cold joint did. Cold joints are less able than hot or warm joints to achieve bonding because they develop when the RCC is exposed for longer than the initial set time. The '*more yielding the surface of the receiving layer at the moment of compaction of the layer above, the better the interpenetration and bonding between successive layers*'.[878] There was no evidence that the Dam's stability was reassessed once the designer became aware that it was being constructed with many more cold joints than had been contemplated.

5.597 The fact that cold joints were all but a daily occurrence on site indicates quality problems. The workforce was not able to keep up with the pace required to lay RCC on hot or warm joints, which are better able to promote bonding between consecutive lifts.

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873 **SUN.018.019.6859**, .6882.
874 **TRA.500.011.0001**, .0007 ln 3-17.
875 **TRA.500.006.0001,** .0037 ln 41-42.
876 **TRA.500.006.0001,** .0038 ln 19-21.
877 **TRA.500.031.0001**, .0018 ln 29-33.
878 Exhibit 69, **ICO.002.0001**, .0093.

5.598 For the foregoing reasons, there were questions about the quality of the lift joints and whether those joints, therefore, had achieved the design values for shear strength. This forms a further basis upon which confirmation shear strength testing should have been pursued.

The use made of the LJQI in constructing the Dam

5.599 An article written by Mr Lopez, Mr Griggs, Mr Montalvo and Mr Herweynen and Dr Schrader in 2005[879] (referred to in Chapter 4), and the evidence, show that the LJQI was used on site to:

a. provide a framework for inspections of lift joints by the RCC Inspectors

b. estimate whether the design parameters had been achieved.

5.600 Dr Schrader asserted that the second use was not permitted. It is not known whether that was his advice to the Alliance when the Dam was being designed. While the design strength values Dr Schrader recommended to the Alliance derived from his database (and not from the LJQI), it was the LJQI that allowed the quality assurance program to link back to those values. This is consistent with Dr Schrader's 1999 article that said a positive LJQI score would assure the designer that design values had been achieved. That is the purpose to which the LJQI was put on site, as can be seen from the final RCC QC Report[880] and the SunWater due diligence presentation in August 2005.[881]

5.601 It was submitted that the LJQI was and could be used by Mr Herweynen as part of the following three stage process for being satisfied that the design intent had been met:[882]

a. Mr Herweynen, as the designer, made an assumption of fact about what lift joint quality would be achieved on site.

b. The LJQI was used to document what quality was being obtained on the lift joints.

c. Mr Herweynen then considered the records maintained during construction (including the LJQI records) to form a view about whether the assumed lift joint quality had been achieved.

5.602 Mr Herweynen assumed that the quality that would be achieved would be 'good'. Dr Schrader advised Mr Herweynen what the friction and cohesion for an 'excellent' and

879 Exhibit 75, **PDI.037.0001**, .0009.
880 Exhibit 38, **SUN.110.003.0001**, .0101.
881 Exhibit 39, **ALC.001.001.1874**, .1882.
882 **TRA.500.009.0001**, .0055 ln 23-39; **TRA.500.010.0001**, .0072 ln 33 to .0073 ln 12.

a 'poor' lift joint should be. Those values were averaged to arrive at design values for a 'good' quality lift joint.[883]

5.603 Mr Herweynen says that he formed a view about whether the quality of lift joints as constructed was good, in the relevant sense, by considering the construction records. However, the perceptions of an onsite inspector that the quality was 'good' might mean something completely different to Mr Herweynen's idea of what was 'good' without something more. That something more was provided by the LJQI, which gave meaning to what 'good' quality was. Using the LJQI meant that the inspector's evaluation of 'good' quality aligned with what Mr Herweynen had intended when he assumed 'good' quality at the outset. That allowed Mr Herweynen to view the construction records and be satisfied that the design values had been achieved. The LJQI system of evaluation was therefore linked with his starting assumption of 'good' quality and the corresponding design values.

5.604 The position of Hydro Tasmania and Mr Herweynen, therefore, is properly understood to be that the LJQI was an appropriate means of verifying that shear strength values had been achieved. Dr Schrader's 1999 article said that '*absolute values should come from testing of the specific mixture and conditions in question*'.[884] It also said that '*Each project should be examined carefully and preferably tested to determine the shear capacity of its joints*'.[885] Of the data upon which the LJQI system was developed, only 12 of the test results were from an RCC mix with a cementitious content similar to the Dam.[886] Mr Dolen considered that the 1999 article did not provide sufficient information for low cementitious mixes.[887]

5.605 It is accepted that the LJQI was a useful quality assurance tool. Inspection as RCC is placed is a fundamental requirement in RCC construction and many of the problems an inspector looks for are matters of judgement. Mr Brigden said that in RCC construction:[888]

> *[T]he very first test is the eye, because by the time that you have got a density result or a moisture result, you've probably placed another hundred to 500 or to a thousand cubic metres, and so it's critical that you have people as observers on construction, day and night, all the time, with very, very keen eyesight.*

5.606 It is not suggested that the LJQI should not have been used for quality control at all. Indeed that is the context of its mention in the 2013 ANCOLD Guidelines where it is referred to as a guide for evaluating the acceptability of a lift joint.[889]

5.607 The problem was that the LJQI was elevated above an inspection framework because its results were used to estimate whether the RCC had fulfilled the design

883 **TRA.500.014.0001**, .0083 ln 9-33.
884 Exhibit 124, **PDI.040.0001**, .0005.
885 Exhibit 124, **PDI.040.0001**, .0004.
886 Exhibit 104, **GHD.006.0001**, .0024.
887 **TRA.500.009.0001**, .0040 ln 40-47.
888 Exhibit 48, **TRA.510.025.0001**, .0033 ln 24-29.
889 Exhibit 35, **ACD.001.0001**, .0047.

parameters. While Hydro Tasmania submitted that the LJQI was but one part of a larger quality assurance program, no other aspect of that program facilitated correlation between qualitative assessments and design values. It was the LJQI that gave meaning to the rating of a lift joint as 'excellent, 'good' or 'poor'.

5.608 In the final RCC QC Report, data on LJQI scores across the project were presented.[890] That summary was immediately followed by tables showing the estimated cohesion and friction angle achieved during construction. Based on that information, the report concluded that '*It can be seen that all construction estimates exceed the design basis*'.[891] Mr Herweynen relied on the information in the RCC QC Reports to certify the Dam as safe for impoundment and at practical completion.

5.609 Estimating shear strength values achieved on site, based heavily on LJQI evaluations, was adopted in favour of verification by testing. The effect of the approach was to choose a qualitative means of verifying the design parameters had been met over more orthodox quantitative means. That was not an appropriate substitution to have made. Dr Schrader and Mr Dolen agreed that the LJQI could not be used in substitution of shear strength testing.[892] As is earlier discussed, the LJQI suffered from a number of flaws, both in its design by Dr Schrader and its application during construction of the Dam. Those problems combined to cast serious doubt over the reliability of the LJQI scores and, therefore, any correlation between those scores and the design values.

5.610 The Alliance seems to have followed the guidance in Dr Schrader's 1999 article about how the LJQI could be used on site. The graphs and discussion in that article explain how the LJQI could be used by a designer during construction. The scores achieved, it was said, would show whether the quality achieved on site was as good as the designer had intended at the time that design values were settled on.[893] If not, the graphs in the article could be used to estimate how much of an impact a shortfall (or exceedance) in quality had as a percentage reduction (or increase) on the design value.

5.611 As is discussed above, the cohesion design figures were not conservative. Therefore, shear strength testing was required to confirm that the design values had been achieved on site. If the design values were always unachievable, no assessment of quality could ever provide assurance that the lifts were in fact sufficiently resistant to sliding. That could only be confirmed by quantitative assessment.

5.612 The LJQI was used to estimate whether shear strength parameters had been achieved. It was not appropriate that such an approach should be favoured over orthodox quantitative confirmation of those parameters.

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890 Exhibit 38, **SUN.110.003.0001**, .0101.
891 Exhibit 38, **SUN.110.003.0001**, .0101.
892 **TRA.500.009.0001**, .0059 ln 6-21.
893 Exhibit 124, **PDI.040.0001**, .0025.

The choice to construct with LCRCC

5.613 A root cause of the present uncertainty about instability was the choice to use an LCRCC mix. LCRCC is less 'forgiving' than HCRCC. It is more difficult to work with, which means it is harder to achieve quality on site. However, quality is all the more important for LCRCC because the material is less amenable to bonding across lift joints due to the lower paste content. Added to this is the observation that LCRCC (especially when of poor quality or if segregated) is difficult to reliably core for shear strength testing. As Dr Schrader said, '*it is extremely difficult to get good cores when crossing lift joints in lean mixes'*.[894] Shear block testing was said to be a more reliable way to test LCRCC[895] but it is expensive.[896] It also tricky so the sampling and testing need to be done by people with sufficient experience to ensure reliable results are obtained.[897]

5.614 These problems are all interlinked. They make it difficult to attain the level of quality needed to achieve the design values, and also make it difficult to verify that those values have been achieved once the dam is constructed.

5.615 However, advancement in LCRCC technologies has led to the adoption of the sloped layer method of construction. That approach leaves far less of the underlying layer of RCC exposed for inspection and preparation before the next lift is placed. The number of horizontal joints is reduced by around 90 percent, with the same percentage saving in inspection, cleaning, and bedding mix requirements (depending on the design basis of the dam).[898]

5.616 The decision to construct the Dam with LCRCC is a root cause of the uncertainty that now attends its stability.

Possible root causes if the Dam is found to be unstable

5.617 On the state of the sampling and shear strength testing undertaken to date, there remain doubts about whether this Dam is stable for some loading conditions. Experts were asked, based on the information currently known to them, whether the Dam as it stands is stable.[899] The experts said:

a. Dr Schrader: '*[I]n my opinion and with my knowledge of it, I think it's most probably, almost certainly, stable, but it hasn't been demonstrated by test results that it is*'.[900] He later said that he was '*99 per cent sure … that there isn't a problem*' with the Dam's stability[901]

894 Exhibit 223, **SCE.031.0001**.
895 **TRA.500.008.0001**, .0014 ln 4-10, .0013 ln 40-41.
896 Exhibit 127, **TRA.510.006.0001**, .0040 ln 26-27.
897 Exhibit 127, **TRA.510.006.0001**, .0040 ln 41 to .0041 ln 2.
898 **SME.001.0001**, .0004.
899 **TRA.500.008.0001**, .0082 ln 42-46.
900 **TRA.500.008.0001**, .0083 ln 2-4.
901 **TRA.500.016.0001**, .0099 ln 28-36.

b. Mr Willey: '*It's presented in the stability memorandum, and that indicates that for particularly large and extreme flood events, there's a factor of safety less than 1*'[902]

c. Mr Dolen: '*I would agree with Mr Willey*'[903]

d. Dr Rizzo: '*I have no objection to what Mr Willey said, except that I am confident that any deficiency in stability could be resolved with some form of remediation*'[904]

e. Mr Tatro: '*I've got to qualify this a little bit. If you're asking me if these are all the test results we're going to get and no further testing is going to be done, what do we do, my first reaction is we're shooting ourselves in the foot and I guess we're forced to move toward some kind of remediation. I just hate believing we have*'[905]

5.618 According to the assumptions made by GHD and the deterministic analyses it conducted, and with reference to the current 2013 ANCOLD Guidelines, the Dam has unacceptable computed values of the factor of safety with respect to shear sliding failure. The Dam, in its current configuration and subject to Unusual and Extreme loading, is structurally unsafe with respect to shear sliding based on the GHD approach to that question. However, the deterministic factor of safety approach is only a guide to safety, and not a guarantee, either of safety or failure. Its reliability is only as good as its inputs and assumptions. Of primary importance in that regard are the shear strength parameters used.

5.619 GHD has adopted the values for cohesion and internal friction from its Updated Shear Strength Memorandum. The figures are based on intrusive sampling and core testing carried out to date on RCC from the Dam. However, the expert evidence was that more testing should be done. Mr Dolen was conscious that more sampling and testing might delay work to mitigate risks to the downstream population. However, he did not categorically reject the suggestion that more testing should be carried out. The three other RCC experts and Mr Willey supported further testing.

5.620 Given the uncertainty that surrounds the Dam's stability, further testing of the RCC lift joints needs to be carried out more accurately and reliably to characterise the shear strength of the lift joints in the Dam. That will allow the stability question to be reappraised. Until further confidence can be had in the results of testing, the stability of the Dam with respect to sliding failure remains uncertain.

5.621 It is, therefore, not possible to make determinative findings about what the root causes of the Dam's instability are when a finding is not yet possible on the question whether the Dam is in fact unstable. However, if that circumstance were to come to

902 **TRA.500.008.0001**, .0083 ln 10-12.
903 **TRA.500.008.0001**, .0083 ln 14.
904 **TRA.500.008.0001**, .0083 ln 18-20.
905 **TRA.500.008.0001**, .0083 ln 24-30.

pass, the following are offered as possible explanations of why the friction angle might be as low as 38º, as testing to date has indicated.

Unique properties of the RCC mix

5.622 A hypothesis suggested by Dr Schrader to explain a friction angle as low as 38º was that there was something unique about the aggregate used in the RCC mix.

5.623 That aggregate was sourced from site: basalt mostly from the diversion channel supplemented by the 'Goodnight beds' from the dam foundation.[906] During the mix design phase, testing was performed on those aggregates to assess their potential for expansive breakdown. No measurable breakdown was detected in the aggregate.[907]

5.624 There is no evidence that there was anything peculiar about the aggregate. The testing results set out in GHD's Updated Shear Strength Memorandum would be the only basis for drawing that inference. When there are other possible explanations, on the current evidence, this hypothesis does not seem likely.

Inability of this RCC to achieve design values

5.625 Aside from whether there was something unique about the aggregate in the RCC mix, the Alliance never verified that the particular LCRCC mix used could achieve the assumed design parameters for shear strength. This raises the question whether it was ever capable of doing so.

5.626 The design values were based on Dr Schrader's advice. His recommended value for friction is in line with conservative industry guidance at the time. However, the cohesion value for an untreated lift joint was high against some industry standards while the value for a treated lift joint far exceeded industry guidance. The high values adopted also contrast with the approach of Dr Rizzo who does not rely on cohesion in stability calculations at all. Mr Foster said it was better to use residual strengths (i.e. no cohesion) in assessing the stability of an LCRCC dam because of the higher probability that there are really low-strength joints compared to a CVC dam.[908] Mr Dolen also suggested that such an assessment should be done in light of documented problems with unbonded lifts and lower than assumed shear strength values in LCRCC dams.[909]

5.627 The Alliance did not adopt a conservative design approach. The industry guidelines indicated that if conservative values were not adopted, the values should be verified by testing. That was never done.

5.628 In circumstances where cohesion was critical to the sliding stability of the Dam and the design values adopted were not conservative, they should have been verified by

906 Exhibit 75, **PDI.037.0001**, .0003.
907 Exhibit 156, **DNR.007.2295**, .2525.
908 **TRA.500.004.0001**, .0046 ln 30-33.
909 Exhibit 104, **GHD.006.0001**, .0022.

testing the RCC as constructed. If, because of the fragility of this LCRCC, cohesion could not be verified by coring and shear strength testing, the cohesion values assumed should have been lower to ensure that they could be achieved. Another possible approach would have been to provide additional mass by flattening the downstream slope of the Dam.

5.629 Dr Schrader, in his 1999 article, stated that '*if a lower cementitious content mix is used for a design which specifies a wider or thicker dam, the shear stress requirements decrease because of the increased mass and area of the lift joint, so less unit, shear strength is needed*'.[910] The Dam has a downstream slope of 0.64 horizontal to 1.0 vertical. Given that the RCC from which the Dam was built had '*one of the lowest cementitious contents on record*', the relatively steep slope was noteworthy for Mr Dolen. He was aware of other LCRCC dams with a downstream slope 0.8 horizontal to 1 vertical. Mr Dolen said:[911]

> *I have one last comment regarding just the overall stability related to the dam, and that is to recognise that the downstream slope is steeper than I would have expected for this type of structure. I would have expected a 0.8 to 1 slope, and that would have greatly affected the stability analysis here. I don't know if it would have been sufficient, but I was surprised by the relatively steep slope.*

5.630 Confirmation testing could not have changed the achieved shear strength parameters. However, if they were shown not to attain the design values, remedial work could have been promptly undertaken to address the shortcoming.

5.631 If the Dam is found to be unstable, possible explanations are that the design did not adopt a conservative approach or incorporate confirmation testing for shear strength parameters.

Poor construction

5.632 As is discussed above, quality control problems were identified during construction. The documents do not permit a finding that all those problems were remedied. This is important because the LCRCC used in the Dam was extremely lean. That made it less workable and harder for the contractor to achieve good quality. It also meant that reliance was greater on pristine standards of construction and quality, which were not always achieved.

5.633 The second TRP has identified poor construction practices as the likely cause of the Dam's instability, should the latter finding ultimately be made. That conclusion was based on a review of the construction memoranda discussed earlier in this Chapter. While the issues raised on those memoranda are not considered as fundamental, as was expressed in TRP Report No. 2, they show that quality issues did arise during construction.

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910 Exhibit 124, **PDI.040.0001**, .0004.

911 **TRA.500.008.0001**, .0086 ln 7-14.

5.634 It is possible that lower than anticipated friction angle and cohesion values could be attributable to construction methodologies that, while perhaps not 'poor', did not reach the heights of exactitude required when building a dam in the Queensland summer with very lean RCC.

Recommendations

1

The materials used to construct a dam and the dam as-built should be subjected to inspection and physical testing to confirm the values adopted for critical design parameters. It is preferable that those responsible for the dam's design and construction organise and oversee such testing.

2

The Commission encourages consideration by the Regulator of mandating the independent technical review of referable dam projects.

3

The panel or body established to conduct the independent technical review should have the authority to co-opt others with appropriate expertise to conduct peer review of matters beyond the collective expertise of the panel members or where obtaining additional views is considered advisable.

4

Matters for review should include but may not be limited to regulatory, safety and operational requirements, the principal components of the dam and its critical design parameters.

5

The Regulator should consider how best to ensure the independence of the persons chosen to conduct peer reviews and whether guidelines to assist and direct those in peer reviewing dam projects would be useful.

Chapter 6 – Downstream protection

Introduction

6.1 A purpose of downstream protection at a dam is to ensure that the dam will not fail during spillway discharges. Without adequate protection of that kind – whether that be a constructed energy dissipator such as an 'apron' or an erosion-resistant riverbed – the water concentrated by the dam structure may cause 'scour'. Scour is erosion caused by forces of water. Depending on its severity and location, erosion or scour risks undermining the dam structure unless there is adequate downstream protection.

6.2 As outlined in Chapter 2, in 2013, scour occurred immediately downstream of Paradise Dam's (**the Dam**) apron at the toe of the primary spillway. Damage was caused to the apron itself. These problems are 'structural and stability issues', within the meaning of sub-paragraph 3(a) of the Terms of Reference.[1] The issues were described in these terms in the Key Issues framed early in the Inquiry:

> *1.2 The adequacy of downstream protection immediately below the Dam, principally:*
>
> *a. the adequacy of the primary spillway apron's dimensions;*
>
> *b. the capacity of the materials from which the primary spillway apron was constructed (and the way in which it was constructed) to resist the erosive force of water.*

6.3 This Chapter concerns the root causes of inadequate protection downstream of the Dam's primary spillway. It discusses how the primary spillway apron came to be designed and built, and how this relates to the structural and stability issues identified above. The apron's adequacy, in terms of design and construction, is relevant. So too is the effect of an earlier flood in December 2010 and January 2011. It was suggested that damage sustained during that flood may have contributed to the damage sustained in 2013.

6.4 The design of appropriate downstream protection measures – for present purposes an 'apron' – engages geotechnical and hydraulic engineering disciplines. It is necessary to examine how those disciplines influenced the design of the apron.

6.5 This Chapter deals with damage to the apron during flood events in 2010/11 and 2013 (**the 2011 event** and **the 2013 event** respectively); what engineering good practice requires of a designer in deciding on downstream protection; as well as geotechnical and hydraulic engineering input into apron design.

[1] Exhibit 1, **PDI.024.0001**, .0001 [3].

Key Issues

6.6 The key downstream protection issues mainly concern the apron below the primary spillway, the adequacy of its dimensions, and that it was constructed using Roller Compacted Concrete (**RCC**). Key Issue 3.2 raised:

> *3.2 In terms of the downstream protection:*
>
> *a. the adequacy of the dimensions, structure and quality of construction of the apron downstream of the primary spillway;*
>
> *b. whether the primary spillway apron was constructed of sufficiently strong material to withstand the erosive forces of water and abrasion;*
>
> *c. the design process and the accuracy and adequacy of the hydraulic modelling, including as to the energy dissipation effects that tailwater would offer, and whether the complexity of anticipated flood flows had been properly accounted for in the apron's design;*
>
> *d. the appropriateness and sufficiency of geological investigations prior to and during construction of the Dam and the availability of them to the Dam's designers;*
>
> *e. the effect of damage from flooding in 2010/11 and how it may have influenced (if it did influence):*
>
> *i. the damage sustained immediately downstream of the Dam in 2013;*
>
> *ii. the hydraulic jump.*

6.7 The scouring immediately downstream of the primary spillway in 2013 was considered in engineering and technical studies that identified the apron's dimensions as contributing to the scour.

6.8 Eric Lesleighter, an hydraulics engineer with more than 50 years' experience,[2] considers that the apron was of insufficient width, observed from the toe of the Dam looking downstream.[3] Andreas Neumaier, the Design Manager, accepted that the 'hydraulic jump' (a means of dissipating energy[4]) was not contained in an apron just 20 m wide.[5]

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2 Exhibit 238, **LEE.001.0001**, .0001.

3 Mr Lesleighter said that the '*apron was too short*'. He also believes that the view that the '*apron contained the hydraulic jump*' was 'fallacious': **TRA.500.012.0001**, .0046 ln 7-16. Mr Lesleighter and Mr Griggs agreed that the appropriate description is 'width': Exhibit 238, **TRA.510.019.0001**, .0011 ln 3-11; Exhibit 288, **TRA.510.009.0001**, .0039 ln 11-14.

4 See paragraphs 6.37-6.42 below.

5 **TRA.500.015.0001**, .0002 ln 28-32. He added: '*The tailwater is always higher than what is required to contain the hydraulic jump, but the hydraulic jump itself would be extending beyond*

6.9 There is disagreement about the nature and content of advice from geotechnical engineers concerning erodibility of the natural riverbed downstream of the primary spillway apron.

6.10 The extent of the damage to the primary spillway apron itself in 2011 and 2013 prompted concerns that the durability of the Roller Compacted Concrete (**RCC**) with which it was constructed might have substantially contributed to the problem. Christopher Dann, a dam designer, said that using RCC for the apron was '*undesirable*'.[6] The first Technical Review Panel (**TRP**) comprised of hydraulic and geotechnical engineers and a dam design expert considered that the '*prime problem*' with the basin floor was that it '*was not constructed in conventional heavily reinforced concrete, but rather in low cementitious content RCC*'.[7] Other witnesses considered that much of the same damage would have been sustained regardless of whether RCC or Conventional Concrete (**CVC**) was used.

6.11 The design of the end sill has also been criticised. Its failure was said to have affected the scour.

6.12 Those opinions are analysed below.[8]

Main characteristics of the Dam

6.13 The Dam has a 315 m long stepped primary spillway with an ogee crest level at EL 67.6 m and a 485 m long secondary spillway on the right bank of the Dam.[9] The gradient of the primary spillway was 1V:0.64H. The Detail Design Report stated that *'[t]he construction of an RCC dam lends itself to a stepped downstream face'*.[10] It added that the spillway 'steps' would provide 'some' energy dissipation, especially in smaller floods.[11] The steps were covered in CVC.

6.14 The apron at the toe of the primary spillway was 20 m wide with a 1 m high sill at the downstream end.[12] It was constructed with a 45 m section at EL 37.5 m which sloped

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the 20 metres in some cases' (.0002, ln 32-35). This is consistent with the URS Independent Technical Review in 2014: '*the hydraulic jump would extend beyond the dissipator structure*': Exhibit 237, **SWA.512.001.0578**, .0587.

6 **TRA.500.012.0001**, .0076 ln 24-28.

7 Exhibit 7, **IGE.017.0001**, .0023.

8 The studies relevant to downstream protection are the TRP (Paradise Dam – Phases 2 and 3 of Remedial Works Program) Reports No. 1 (October 2013) and 4 (15 December 2015) Exhibit 7, **IGE.017.0001** and Exhibit 10, **IGE.020.0001**, respectively; the Draft Inspection Report of the Dam Safety Regulator in April 2013, **DNR.012.9331**; SunWater's Dam Safety Review, Revised Report, in 2016 Exhibit 42, **DNR.002.3132**; and the TRP (Paradise Dam Spillway Improvement Project) Reports No. 1 (29 May 2019) and 2 (23 September 2019) Exhibit 11, **SUN.009.003.0613** and Exhibit 12, **IGE.051.0001**, respectively. See also the Review of Dam Safety Management Actions report by NSW Water Solutions, 22 August 2013, which set out some inadequacies (Exhibit 5, **DNR.002.8498**, especially .8522-3). They were identified in engineering and technical studies within the dates specified in the Terms of Reference.

9 Exhibit 24, **GHD.002.0001**, .0059.

10 Exhibit 24, **GHD.002.0001**, .0132.

11 Exhibit 24, **GHD.002.0001**, .0132.

12 Exhibit 24, **GHD.002.0001**, .0188.

down toward the right abutment to a 165 m wide section at EL 30.845 m.[13] The result was an 'asymmetry' built into the apron (shown in the image below).[14]

*Figure 6.1 – The Dam 'as constructed' in 2005. The asymmetry created by the upwards slope on the left hand end of the apron is visible, depicted by the red arrow. (Exhibit 237, **SWA.512.001.0578**, .0596)*

6.15 Two layers of RCC (620 mm thick in total) were considered '*adequate*' for the floor of the primary spillway apron.[15] A corrective layer of dental concrete underlay the two.[16] A higher cement content was used for the upper layer of the apron (150 kg/m^3)[17] than for the RCC in the mass of the Dam wall.[18] Angled drain holes were drilled through the apron into the rock below to reduce apron uplift pressures.[19] The end sill was constructed of CVC.[20]

6.16 The Dam was designed to pass the Probable Maximum Precipitation Design Flood (**PMPDF**) (or the 'design flood') which represented a 1 in 30,000 Annual Exceedance Probability (**AEP**) event. Such a flood was estimated to have a peak discharge of

13 Exhibit 24, **GHD.002.0001**, .0136; Exhibit 237, **SWA.512.001.0578**, .0617.

14 **SWA.512.001.0578**, .0596.

15 Exhibit 24, **GHD.002.0001**, .0188.

16 Exhibit 237, **SWA.512.001.0578**, .0637.

17 **DNR.002.5621**, .5698. In an email dated 4 April 2005, Dr Schrader wrote to Mr Montalvo and others stating, '*I have always intended that the 150 [kg/m^3] mix would be in the secondary spillway also … As with the primary apron, it really only needs to be the top lift …*': **DNR.011.1433**.

18 Exhibit 24, **GHD.002.0001,** .0188. See also **DNR.005.3763**, .3767 confirming RCC mix for the last layer of the aprons.

19 **DNR.001.0036**, .0048.

20 **DNR.020.012.3848**, .3947.

93,457 m^3/s.[21] The discharges expected at the Dam are very large – within the top 10 dam discharges in the world.[22]

6.17 The Probable Maximum Flood (**PMF**) was estimated to have a discharge of 104,451 m^3/s.

Table 3-8: Flood Exceedence Probabilities

AEP (1:Y)	PEAK INFLOW (M^3/S)	PEAK DISCHARGE (M^3/S)	PEAK LEVEL EL (M)
50	12,808	12,446	74.87
100	15,326	14,869	75.71
200	17,799	17,247	76.48
500	22,033	21,297	77.72
1,000	25,477	24,769	78.61
2,000	29,202	28,490	79.41
5,000	37,650	36,739	80.94
10,000	50,328	49,212	82.86
20,000	71,738	70,531	85.43
30,000	94,861	93,415	87.69
PMF	106,863	104,451	88.67

*Figure 6.2 – Table of expected discharges at the Dam as set out in the Detail Design Report. (Exhibit 24, **GHD.002.0001**, .0049)*

Damage to the primary spillway apron from the 2011 event and its likely mechanisms

6.18 A flood with a peak discharge with an AEP of approximately 1 in 25 occurred during the 2011 event (a peak discharge of approximately 8,770 m^3/s).[23] This was the first time the Dam had spilled.[24] It continued to spill until September 2012, except for three days in November 2011 and two days in January 2012.[25] This made inspection of the primary spillway apron virtually impossible and the remedying of any damage impracticable until the Dam stopped spilling.

21 Exhibit 24, **GHD.002.0001**, .0059.

22 **TRA.500.011.0001,** .0039 ln 8. The Dam was also described as '*almost record breaking in terms of its discharge capacity*': **TRA.500.012.0001,** .0071 ln 27-28.

23 Exhibit 230, **DNR.006.3156**, .3168.

24 **DNR.001.0036**, .0072.

25 **DNR.001.0036**, .0072.

*Figure 6.3 – Primary spillway in operation during the 2011 flood event. (Exhibit 230, **DNR.006.3156**, .3211)*

6.19 A damage inspection was carried out in September 2012.[26] The end sill and left 'training wall'[27] were rounded by abrasion. Steel reinforcing within the apron was exposed.[28] Other parts of the apron suffered abrasion.[29] The downstream face of the stepped primary spillway was damaged. Shotcrete placed on the left bank above the training wall had been scoured back and rock was lodged between the left training wall and the shotcrete face.[30] There was no evidence of undermining of the rock foundation on the downstream edge of the apron end sill.[31]

6.20 The damage was caused by the '*tumbling of rock, probably over a wide size range, in the turbulent flow*'.[32] This was the 'ball-mill' effect.[33] According to Mr Lesleighter, the 'asymmetry' in the apron created different flow conditions over the two ends of the apron. His November 2012 report stated:[34]

> *On the right half of the basin length the flow seems to have been deflected up over the end sill, then plunge back into the tailwater. Along the other half, the action is quite different, and seems to have entrained rock in such a way as to batter the end sill. The fact that the flow regime is different from one end to the other is enough to cause rock to be brought back into the basin and then be*

26 Exhibit 230, **DNR.006.3156**, .3164.
27 Training walls to the sides of the primary spillway guide the flow into the apron.
28 **DNR.001.0036**, .0074.
29 Exhibit 5, **DNR.002.8498**, .8520; Exhibit 230, **DNR.006.3156**, .3175-6.
30 Exhibit 230, **DNR.006.3156**, .3169.
31 Exhibit 230, **DNR.006.3156**, .3176.
32 Exhibit 230, **DNR.006.3156**, .3241.
33 See paragraph 6.276 below.
34 **DNR.006.3156**, .3241.

tumbled around violently. The flow has been violent enough to cause the abrasion of the end sill as well as in the floor of the basin near the end sill….

Given the asymmetry from the right side to the left side of the spillway, and the variable basin floor level, the asymmetric flow pattern will always be there in future floods.

6.21 Mr Neumaier made the point that, although the 2011 event may not have been large, it was, '*an exceptional event in the sense that it lasted for several months*'.[35]

*Figure 6.4 – Abrasion damage to the apron floor and the end sill following the 2011 event. (Exhibit 5, **DNR.002.8498,** .8520)*

Damage to the primary spillway apron in 2013 and its likely mechanisms

6.22 A larger flood[36] occurred in January 2013 as a result of heavy rain following ex-tropical cyclone Oswald.[37] The peak discharge was estimated to be 17,000 m^3/s on 28 January 2013.[38] There was a second, smaller peak on 3 March 2013.[39] Although that flood was a significant event, the peak flow of the 2013 event was about 16% of the PMF's flow.[40]

35 **TRA.500.015.0001**, .0019 ln 3-11. See also Exhibit 247, **TRA.510.007.0001**, .0083-4.
36 An estimated AEP 1 in 170 event: **DNR.001.0036**, .0072.
37 Exhibit 237, **SWA.512.001.0578**, .0602.
38 Exhibit 237, **SWA.512.001.0578**, .0602. See also **DNR.001.0036**, .0075.
39 **HYT.008.0001**, .0012.
40 **DNR.001.0036**, .0075.

*Figure 6.5 – Flow over the primary spillway on 29 January 2013. (Exhibit 7, **IGE.017.0001**, .0060)*

6.23 The 2013 event caused almost complete removal of the vertical end sill from the apron (apart from a 7 m length[41]) and an extensive scour of the riverbed at the downstream edge of the apron.[42] About half of the top RCC layer of the apron was damaged – an area of about 1,100 m^2 on the left end of the apron.[43] In this area, the RCC '*either cracked, broke up and/or washed out*'.[44] The separation of the RCC layers occurred at the plane of the reinforcing steel. The destruction of the RCC above that reinforcing steel was said, in a report by NSW Water Solutions, to have occurred by pounding from the high energy discharge jet flowing over the primary spillway.[45] Some of the lower layer had also scoured away, exposing part of the underlying rock (the Goodnight Beds) to turbulent flow.[46] Anchor bars that fixed the end sill to the apron snapped.[47]

41 **DNR.001.0036**, .0090.
42 Exhibit 7, **IGE.017.0001**, .0057; Exhibit 5, **DNR.002.8498**, .8526.
43 **DNR.020.012.3848**, .3947; **DNR.001.0036**, .0080.
44 **DNR.001.5574**, .5673.
45 Exhibit 5, **DNR.002.8498**, .0570.
46 **DNR.001.0036**, .0080.
47 **DNR.020.012.3848**, .3948.

Figure 6.6 – Damage to the apron following the 2013 event.
*The end sill on the left hand side of the apron has detached. (**DNR.001.0036**, .0092)*

6.24 Significant scouring occurred immediately downstream of the primary spillway apron. A large scour hole developed on the left side. It was about 13 m deep.[48] That hole and the related scour developed along a fault zone in the rock.[49] At the right end, another scour hole was about 3.5 m deep and 25 m long.[50] A geological feature was exposed comprising two significant faults underlying the primary spillway foundation and apron.[51]

Figure 6.7 – Scour hole immediately downstream at the right side of the apron.
*The arrows show the estimated region of the fault zone. (**DNR.001.0036**, .0093)*

48 Exhibit 7, **IGE.017.0001**, .0057.
49 **DNR.001.0036**, .0081. The fault zones are described in more detail at paragraph 6.98 below.
50 **DNR.020.012.5265**, .5272.
51 Exhibit 13, **SUN.009.002.0001**, .0005 to .0006.

Figure 6.8 – Scour hole immediately downstream at the left side of the apron.
*(Exhibit 7, **IGE.017.0001**, .0059)*

6.25 The extent of the apron damage during a flood of the 2013 magnitude was not expected.[52] Richard Herweynen said that one would '*not usually expect*' such damage.[53] Mr Neumaier said similarly, '*you wouldn't normally expect that sort of damage that we have experienced for a flood which is less than a 1 in 200 year event*'.[54] A report by URS (an 'Independent Technical Review') in 2014 (**2014 URS Review**), concluded:[55]

> *The reported damage to the primary spillway following the 2010 and 2013 spillway discharge events would not be expected for a structure designed and constructed to modern design standards.*

6.26 The Dam was designed with the intention of withstanding events much greater than these. That there was a continuous spill in 2011 followed by the 2013 event may have meant the resulting damage was, in Mr Lesleighter's view, '*a little bit more severe, but … only marginal*'.[56]

6.27 According to a report by SunWater Limited (**SunWater**), the damage to the apron and the areas immediately downstream of it created risks that were '*above the limit of tolerability*'.[57] John Young, a geotechnical engineer who was a member of the 2019 TRP,[58] described the scour as '*very serious*', adding, '*[i]f the flood in 2013 had*

52 Exhibit 27, **IGE.076.0001**, .0007.
53 **TRA.500.013.0001**, .0079 ln 32-33.
54 **TRA.500.015.0001**, .0019 ln 1-3.
55 Exhibit 237, **SWA.512.001.0578**, .0586.
56 **TRA.500.012.0001**, .0018 ln 18-30.
57 **DNR.001.0036**, .0091.
58 Exhibit 76, **YOJ.001.001.0001**, .0002 [6].

persisted for a significantly longer period of time, erosion could have worked its way to the dam and started to undermine the rock under the dam'.[59]

Remedial work

6.28 Following the 2013 event, remedial work was undertaken to ensure '*short-term protection*'.[60] It proceeded in phases. Phase 1 provided '*immediate protection*' to the damaged apron concrete and the downstream riverbed scour holes.[61] The scour holes in the riverbed were backfilled with concrete.[62] That was completed by June 2013.[63] In Phase 2, the end sill was reinstated and a heavily reinforced and anchored 'stepped scour protection wall' was placed on the upstream face of the scours in the riverbed.[64] Extensive repairs of the apron floor at each end of the primary spillway were carried out.[65] That phase was completed towards the end of 2013.[66] Phases 3A and 3B involved a dam safety review and comprehensive risk assessment which were completed in 2016.[67]

6.29 A later phase (Phase 4A) involved capping the entire dissipator apron slab with a conventional reinforced concrete slab anchored to the rock.[68] The 'cap' was 600 mm thick.[69] It was placed on top of the existing RCC apron.[70] The slabs comprising the reinforced concrete layer incorporated instrumentation to provide warning if the slabs were to fail during a flood event.[71]

Downstream protection: general considerations

The primary spillway at the Dam

6.30 Spillways allow the release of surplus or flood water. A 'primary' spillway releases water before other spillways (such as 'secondary' or 'auxiliary' spillways) operate.

6.31 The Dam has an 'uncontrolled overflow spillway'.[72] Such dams, according to URS, are typically designed with the following major components:[73]

59 Exhibit 76, **YOJ.001.001.0001**, .0004 [14].
60 **DNR.020.012.5265**, .5270.
61 **DNR.001.5574**, .5674.
62 **DNR.020.012.5265**, .5270.
63 Exhibit 234, **IGE.033.0001**, .0005.
64 **DNR.001.5574**, .5674.
65 Exhibit 7, **IGE.017.0001**, .0021.
66 Exhibit 234, **IGE.033.0001**, .0005; cf. **DNR.001.0152**, .0156.
67 Exhibit 234, **IGE.033.0001**, .0005.
68 **DNR.001.5574**, .5674.
69 **DNR.001.5574**, .5674.
70 **DNR.001.0152**, .0165.
71 **DNR.001.0152**, .0168.
72 Exhibit 237, **SWA.512.001.0578**, .0617.
73 Exhibit 237, **SWA.512.001.0578**, .0617. An image of the Dam's primary spillway is at Figure 6.1.

- *Control structure – a component that regulates the outflows from the reservoir. For the Paradise Dam primary spillway, the control structure is a 315 m wide ogee shaped concrete crest at EL 67.6 m.*

- *Spillway chute – the spillway chute contains the spillway discharge from the control structure and guides the flow towards the dissipator structure. For the Paradise Dam primary spillway, the RCC steps and the concrete lined walls guide the flow (jet) into the dissipator structure at the downstream end.*

- *Dissipator – the purpose of the dissipator structure is to reduce the kinetic energy (i.e. high velocity flow) that is generated from flow over the spillway crest in the form of an incoming water jet, and return the water to the river/creek.*

6.32 During flows over the spillway, particularly at large discharges, the dissipator surface experiences significant transient pressures (i.e. the forces going into the apron[74]) and the surface must be adequate to resist the abrasive forces expected.[75] The Detail Design Report stated that reinforced RCC was '*less expensive*' than reinforced CVC,[76] and was the '*favoured material, if adequate erosion resistance could be demonstrated*'.[77]

Functional requirement of the primary spillway dissipator at the Dam

6.33 Where surplus storage water and flood flows may be released, it is necessary to consider the potential for erosion downstream and the protection measures required.[78] The functional requirements of the apron (the primary spillway dissipator) were:[79]

- *Protection to the foundation immediately downstream of the Dam to prevent erosion of the toe of the spillway*

- *Adequate energy dissipation during low flows. During medium to high flows the high tailwater will provide energy dissipation and erosion protection to the toe of the spillway.*

6.34 Erosion of the riverbed may be caused by water plunging over the spillway. In *Hydraulics of Spillways and Energy Dissipators*, Khatsuria writes:[80]

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74 **TRA.500.012.0001**, .0022, ln 40-46.

75 According to URS, '*The RCC apron was designed to support the incoming water jet and provide a non-erodible surface for the hydraulic jump to form*': Exhibit 237, **SWA.512.001.0578**, .0617.

76 Exhibit 24, **GHD.002.0001**, .0186.

77 Exhibit 24, **GHD.002.0001**, .0186.

78 See, e.g., Exhibit 81, **DNR.007.1087**, .1193 (URS's stage 2 design proposal).

79 Exhibit 24, **GHD.002.0001**, .0186.

80 Rajnikant Khatsuria, *Hydraulics of Spillways and Energy Dissipators* (2005) 371.

Dissipation of the kinetic energy generated at the base of a spillway is essential for bringing the flow into the downstream river to the normal – almost pre-dam – condition in as short of a distance as possible. This is necessary, not only to protect the riverbed and banks from erosion, but also to ensure that the dam itself and adjoining structures like powerhouse, canal, etc. are not undermined by the high velocity turbulent flow.

6.35 The undermining of a dam may be caused by scour.

6.36 This section briefly considers the means by which the energy contained in a body of flowing water may be reduced or controlled, particularly in the context of erosion and scour. It considers the mechanisms which dam designers use downstream of the dam to protect its structure.

Energy dissipation by means of a 'hydraulic jump'

6.37 To reduce its erosive effect, the energy within water spilling at high velocity requires 'dissipation'. 'Hydraulic jump' is the term used to describe the mechanism within which the spilling water goes from high velocity to a much lower velocity.[81] It is, therefore, '*a means of dissipating a lot of energy … so that it flows out at a velocity which the downstream area can contain or cope with*'.[82] Mr Lesleighter said that, typically, a hydraulic jump '*has a certain length where you can say it has achieved its energy dissipation sufficiently*'.[83]

6.38 The following diagram in Mr Lesleighter's July 2013 report depicts a hydraulic jump:[84]

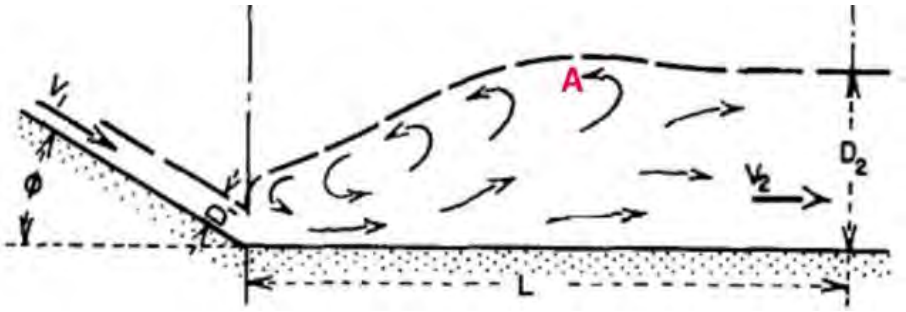

*Figure 6.9 – Hydraulic jump profile (USBR Eng. Mono.25). (Exhibit 7, **IGE.017.0001**, .0049).*

6.39 The full length of the hydraulic jump is shown by 'L' in the diagram above.[85] 'D_2' depicts the height of the tailwater. 'A' shows the location of the 'upwelling flow' within the hydraulic jump.

81 Exhibit 238, **TRA.510.019.0001**, .0012 ln 26-32. Mr Lesleighter explained that, in technical hydraulic terms, this process converts the water from a 'supercritical' to a 'subcritical' flow.
82 Exhibit 238, **TRA.510.019.0001**, .0013 ln 10-13.
83 Exhibit 238, **TRA.510.019.0001**, .0013 ln 32-34.
84 Exhibit 7, **IGE.017.0001**, .0049. The diagram is from the US Bureau of Reclamation's Monograph 25: Exhibit 235, **PDI.064.0001**.
85 Exhibit 7, **IGE.017.0001**, .0049.

6.40 The hydraulic jump can be altered by the structure designed to dissipate the energy: for example, an end sill or 'baffle blocks' (concrete block-like appurtenances on the floor of the apron). They may assist in the formation of the hydraulic jump.[86]

6.41 The level of the tailwater is commonly a factor influencing how the hydraulic jump will affect the surface downstream of the spillway. Tailwater assessments are typically conducted by hydrologists. In the case of the Dam, a flood studies team produced the initial tailwater assessments.[87] They showed that the tailwater was '*unusually high*'.[88] Russell Paton, a civil engineer who worked for SunWater and was part of the team that produced the Preliminary Design for the Dam, said that the tailwater itself could '*greatly benefit*' energy dissipation.[89]

6.42 According to Mr Lesleighter, however, reliance on high tailwater to dissipate energy can be a 'trap' for designers.[90] That is because a 'drowned' or 'submerged' hydraulic jump may form.[91] Where the tailwater is very high, the discharging jet of water may flow over the spillway, plunge to significant depths in the tailwater and continue on downstream.[92] This is an unfavourable hydraulic condition[93] which may result in the hydraulic jump continuing far beyond the area designed to control it. That the jump is submerged does not mean the plunging water loses all the energy that might cause riverbed erosion.

Protecting against erosion and scour

6.43 The undermining of the dam and its structures may be caused by scour where the rock is eroded or abraded by the forces of the water. Mr Paton explained that when considering an energy dissipation structure, '*you want to dissipate the energy such that it doesn't scour downstream of the structure and potentially undermine the dam structure itself*'.[94]

6.44 That is not to say that all erosion can be prevented. Some erosion downstream of the apron is not unusual when the spillway flows, particularly in large flood events.

6.45 Dr Steven Pells, an engineer with experience in hydraulics, hydrology and groundwater studies, considers that the proper approach is to allow for some amount of erosion. A dam designed to experience no erosion in a PMF is an '*over-design*'.[95] According to Malcolm Barker, an engineer with a specialisation in the risk analysis of

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86 **TRA.500.006.0001**, .0014 ln 2-3. Dr Maleki agreed that an end sill may help to reduce the length of the stilling basin: **TRA.500.011.0001**, .0033 ln 14-17.
87 **TRA.500.006.0001**, .0006 ln 16-17.
88 **TRA.500.006.0001**, .0004, ln 19-20.
89 **TRA.500.006.0001**, .0005 ln 19-23.
90 Exhibit 238, **TRA.510.019.0001**, .0008 ln 34-40 and **TRA.500.012.0001**, .0011 ln 36-41.
91 **TRA.500.006.0001**, .0005 ln 29-32.
92 **TRA.500.012.0001,** .0011 ln 43-45. This was also identified in the April 2003 SunWater 2D hydraulic model study: Exhibit 236, **SUN.018.026.5653**, .5657.
93 **TRA.500.006.0001**, .0005 ln 32-33.
94 **TRA.500.006.0001**, .0005 ln 16-23.
95 Exhibit 72, **STP.001.0001**, .0002 [10].

dams,[96] '*[t]he apron should, however, dissipate the energy of the spillway flow which will lessen the erosion downstream*'.[97] Mr Lesleighter agreed that it was not uncommon to experience some erosion in a flood. But, he said, whatever happens, that erosion ought not be such as to fail the dam or any major facility within it.[98]

6.46 Mr Herweynen said that reliance on the riverbed to withstand the erosive force of some of the hydraulic jump '*is always the balance that happens in a dissipator structure*'.[99]

The apron as a mechanism for energy dissipation

6.47 Dr Pells explained that an apron is a section of concrete lining downstream of a spillway which offers some protection against erosion of the underlying natural materials.[100]

6.48 The Dam's primary spillway apron was 20 m wide.[101] It terminated with an end sill.[102] The stated intention was that the apron and end sill (operating as a 'stilling basin') would create a 'roller' effect in the downstream tailwater.[103]

6.49 According to the 2014 URS Review:[104]

> *The purpose of the end sill was to direct the submerged jet on the apron up into the downstream flow and off the river/creek bed to reduce the potential for downstream erosion. Without the end sill, the submerged jet would carry along the river bed.*

6.50 Erodibility of the riverbed immediately downstream of the apron is an important aspect of the design of downstream protection measures.

Guidelines relating to spillways and downstream protection

Background

6.51 The design of spillways and downstream protection and the investigations that inform design decisions are the subject of guidelines. Industry practice is also relevant.

6.52 The Queensland Dam Safety Management Guidelines 2002 (**the Guidelines**) mention two types of investigations when developing a dam:[105] geological and

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96 **BAM.004.0001**, .0002. Mr Barker peer reviewed the Dam's 2009 risk assessment.

97 **BAM.004.0001**, .0006 [22]. Mr Lesleighter stated in his interview: '*the stability of the dam is going to depend on whether there is a scour taking place. That's part of it. It's not only the quality of the RCC, but if you have a big hole downstream, the dam stability is affected*': Exhibit 238, **TRA.510.019.0001**, .0042 ln 22-26.

98 **TRA.500.012.0001**, .0013 ln 46 to .0014 ln 3.

99 **TRA.500.013.0001**, .0071-2 (see in particular .0072 ln 1-3).

100 Exhibit 72, **STP.001.0001**, .0010 [40].

101 See paragraph 6.14.

102 Exhibit 24, **GHD.002.0001**, .0007.

103 Exhibit 24, **GHD.002.0001**, .0032.

104 Exhibit 237, **SWA.512.001.0578**, .0617.**SWA.512.001.0578**, .0617.

105 Exhibit 17, **PA-18**, .0014.

geotechnical investigations; and hydrological investigations.[106] In relation to the former, the Guidelines note that they are typically carried out in stages from broad scoping to detailed investigations. Each stage '*should be thoroughly planned to ensure that all matters, which may affect dam safety, are identified, investigated and appropriately resolved* ***by the designer***'.[107]

6.53 The Guidelines add that '*[i]nvestigations should not be limited to the dam site alone*'[108] but do not refer specifically to the area downstream.

6.54 Hydrological investigations include '*assessing the consequences of potential failure of the dam*' and '*determining the spillway design standard*'.[109] All such work, '*including documentation of mathematical models*',[110] should be presented in a comprehensive report.

6.55 The dam designer is required to document design and construction, including:[111]

- *… design parameters adopted and assumptions made (and their basis)*
- *methods of analyses*
- *results of analyses and investigations (numerical and physical)*
- *hydraulic model testing of final spillway arrangements …*

6.56 Further, according to the Guidelines, the issues that ought be considered when preparing a design report or a safety review report include a '*[s]ummary of assumptions and methods adopted for the design of energy dissipaters for spillways and outlets*'.[112]

6.57 Industry practice for setting design criteria for the dissipator, according to URS, includes dissipator basin type, design event (or discharge), and assessment of potential risks for lower probability discharge events (or discharge).[113] There is no Australian Standard for spillway dissipator design.[114]

US Bureau of Reclamation guidance

6.58 An important guide to the design of dissipators is the US Bureau of Reclamation's (**USBR's**) monograph *Hydraulic Design of Stilling Basins and Energy Dissipators* (8th ed, May 1984) (**the USBR Monograph**).[115] Mr Lesleighter described the document

106 Exhibit 17, **PA-18**, .0015.
107 Exhibit 17, **PA-18**, .0015 (emphasis added).
108 Exhibit 17, **PA-18**, .0015.
109 Exhibit 17, **PA-18**, .0015.
110 Exhibit 17, **PA-18**, .0015.
111 Exhibit 17, **PA-18**, .0020.
112 Exhibit 17, **PA-18**, .0064.
113 Exhibit 237, **SWA.512.001.0578**, .0618.
114 **TRA.500.012.0001**, .0071 ln 47 to .0072 ln 2.
115 **TRA.500.012.0001**, .0014 ln 43-47. The monograph is Exhibit 235, **PDI.064.0001**.

as '*almost like a Bible*'.[116] Mr Neumaier said that the USBR Monograph would be in the repertoire of '*every dam engineer, every hydraulics engineer*'.[117] Dr Shayan Maleki, an hydraulics engineer with GHD, called the USBR Monograph a '*good first pass*'.[118] Mr Lesleighter regards that characterisation as an understatement, saying that the monograph was a '*first port of call*',[119] and that it is:[120]

> *a very reliable pass, a way to evaluate a stilling basin, if it's of this sort of configuration. Then you can do refinements. Refinements come along with maybe detailed hydraulic modelling. But [the USBR Monograph is] not just a first pass.*

6.59 The USBR Monograph, although not a standard, sets out descriptions for 'standard type' dissipators.[121] The dissipator apron at the Dam does not conform to a standard type but it is analogous to a 'Type II' dissipator,[122] except that the Dam's end sill is not dentated and does not slope.[123] Further, unlike the standard Type II dissipator, no 'chute blocks'[124] are incorporated at the upstream end of the apron.

6.60 Insofar as a designer chooses to depart from the USBR Monograph's standard types, that decision, Mr Lesleighter suggests, would need to be justified by way of a 3D physical model: '*a good, well-designed, well-specified, well-managed and well-controlled physical model, and all of those ingredients*'.[125] Such a model also needs to deal with 'scale effects'.

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116 Exhibit 238, **TRA.510.019.0001**, .0012 ln 31-32.

117 Exhibit 302, **TRA.510.021.0001**, .0031 ln 14-16.

118 **TRA.500.011.0001**, .0044 ln 6.

119 **TRA.500.012.0001**, .0016 ln 10.

120 **TRA.500.012.0001**, .0015 ln 27-32. Mr Dann agreed the USBR Monograph should be used as a 'starting point' before a designer turns to detailed physical models: **TRA.500.012.0001**, .0080 ln 15.

121 Exhibit 7, **IGE.017.0001**, .0060.

122 The USBR Monograph describes a type II dissipator (or 'stilling basin') as containing '*chute blocks at the upstream end a dentated sill near the downstream end. No baffle piers are used …*': Exhibit 235, **PDI.064.0001**, .0034.

123 **TRA.500.012.0001**, .0034 ln 32-35: Mr Lesleighter said, '*A type II dissipator has a dentated end sill, an end sill which is sloped in the direction of flow, and also half of it is vertical. That's a dentated end sill. In this case here we have something analogous to a type II, but the end sill has been changed, so it's not standard in that sense that it's just a vertical end sill.*'

124 These '*tend to corrugate the jet, lifting a portion of it from the floor to create a greater number of energy dissipating eddies, resulting in a shorter length of jump than would be possible without them. These blocks also reduce the tendency of the jump to sweep off the apron at tail water elevations below conjugate depths*': see the USBR Monograph, Exhibit 235, **PDI.064.0001**, .0035.

125 **TRA.500.012.0001,** .0037 ln 27-30.

Engineering judgement

6.61 Dam designers are confronted with uncertainties in relation to hydraulics and geology – the two disciplines relevant to designing an apron.[126] A degree of judgement is exercised. This judgement, however, must be based on '*proper study*'[127] and a '*well-designed physical hydraulic model, with all of the effects of scaling taken into consideration*'.[128] Further, according to Mr Lesleighter, 'conservatism' is normally built into the design of the apron '*depending on the things that you don't know, such as geology*'.[129]

6.62 Mr Herweynen explained it in these terms:[130]

> *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).*

6.63 However, a dam should not be 'over-designed', to use Dr Pells's characterisation.[131] Designers expect to experience some erosion during flood events.[132] Although that is so, Mr Lesleighter said that:[133]

> *… normally you would have a dissipator which is of sufficient length not only to achieve your energy dissipation to a sufficient degree but also, if it does erode beyond that structure, it's far enough away from the dam that it's not going to threaten the dam. Even if the spillway started to undermine for some reason, it's going to take many floods before that finds its way back to the dam, and it gives you a chance to do maintenance. You've got a balancing act going on there.*

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126 **TRA.500.012.0001**, .0009 ln 33.
127 **TRA.500.012.0001**, .0010 ln 23-24.
128 **TRA.500.012.0001**, .0010 ln 31-33.
129 Exhibit 238, **TRA.510.019.0001**, .0010 ln 24-25.
130 Exhibit 245, **HER.002.0001**, .0004 [15].
131 Exhibit 72, **STP.001.0001**, .0002 [10].
132 **TRA.500.012.0001,** .0013 ln 43-46.
133 **TRA.500.012.0001**, .0014 ln 3-12. GHD states: '*Adopting an unlined stilling basin, or a stilling basin with a limited extent of lining, to dissipate the energy of the spillway discharge is a common design method in practice to reduce the cost. Unlined stilling basins are often sited in rock which is judged to be resistant to erosion at least for at least high frequency floods*': Exhibit 228, **GHD.041.0001**, .0066. GHD also noted that the '*geology and hydraulic of the flow in Paradise Dam is quite complex*': Exhibit 228, **GHD.041.0001**, .0071.

The design process and compliance with guidelines and industry practice

Interaction between geotechnical and hydraulic engineers

6.64 In terms of the apron, issues that confront dam designers include the potential for erosion downstream of the spillway. Designers also take account of the expected hydraulic forces to determine what protection mechanisms are required.

6.65 These issues involve consideration of whether the expected hydraulic forces were to be wholly contained in the apron or whether it was necessary to rely upon rock in the riverbed to resist erosion. This brings into focus the interaction between engineering disciplines: first, hydraulics, and secondly, geotechnical considerations.[134]

6.66 Thus, geotechnical and hydraulic engineering expertise (the latter relying in some part on hydrological expertise) come together in this assessment. Witnesses accepted that there needs to be an effective 'interface' between these two disciplines. Mr Dann proposed that the design manager should play that role.[135] Mr Herweynen accepted that it is important to have '*the correct interface between [the] hydraulic engineer and [the] geotechnical engineer so far as the apron is concerned*'.[136]

6.67 In answer to a question by Senior Counsel Assisting the Commission, Mr Herweynen explained the role of the dam designer in terms of that 'interface':[137]

> *Q. It was for you as the designer, in this case, to - I'm going to use the word 'interface' because someone else used it - interface with those two disciplines to make the judgment about where the apron needs to be and where it doesn't need to be?*
>
> *A. Yes, that's the case, but, as I said, in this case it was quite unique in terms of the reporting structure with Andreas Neumaier [the design manager] who was heavily involved in that particular area.*

6.68 Mr Herweynen was involved in the 'interfacing' between the hydraulic and geotechnical engineers.[138] However, both the hydraulic and geotechnical engineering disciplines reported to Mr Neumaier rather than Mr Herweynen.[139] Golder Associates (**Golder**) was retained by SMEC Australia Pty Ltd (**SMEC**), Mr Neumaier's employer, and he was responsible for the formal communication with Golder.[140] Mr Neumaier '*approved for issue*' the Dam's Hydraulic Model Study Report (appendix D to the Detail Design Report)[141] and reviewed the Hydraulic Design section of the Detail

134 **TRA.500.005.0001**, .0039 ln 30-37.
135 **TRA.500.012.0001**, .0061 ln 23-25.
136 **TRA.500.013.0001**, .0072 ln 26-33.
137 **TRA.500.013.0001**, .0073 ln 6-14.
138 **TRA.500.013.0001**, .0073 ln 6-14.
139 **TRA.500.013.0001**, ,0072 ln 31-35.
140 **TRA.500.013.0001**, .0073 ln 21-22.
141 Exhibit 24, **GHD.002.0001**, .0554.

Design Report.[142] In the Design Management Plan, a '*Design Team and Responsibility Matrix*' listed Mr Herweynen as responsible for the 'Geotech' and 'Dam' aspects.[143] Mr Neumaier was listed as responsible for all areas, including those.[144]

6.69 The dam designer, however, does not carry out the erodibility assessment. That, it was said, is the task of the geotechnical engineer[145] with input from hydraulic engineers.[146] As Dam designer, Mr Herweynen was to obtain, evaluate and appropriately use the information provided by the hydraulic and geotechnical engineers.

'Documenting' the design of the dissipator

6.70 The exercise of the Dam designer's judgement was to be documented, as required by the Guidelines. That would also accord with engineering good practice. URS stated that '*[i]ndustry practice for designing spillway structures (crest, chute, and dissipator) includes outlining the criteria for which the structure is designed to achieve*'.[147]

6.71 In the 2014 URS Review, the authors concluded that '*[n]o specific design criteria have been set in Section 4.2.1.8 or Section 5.7.6 of the Design Report for the design of the spillway dissipator, only performance criteria*'.[148] The criticism is warranted.

6.72 Section 4.2.1.8 of the Detail Design Report provided:[149]

> ***4.2.1.8 Spillway Apron***
>
> *A 20m wide spillway apron was proposed in SunWater's Preliminary Design (SunWater, 2003a) and adopted for the final design after the hydraulic model studies showed that it performed satisfactorily. A hydraulic jump occurred on the apron for all flood events tested.*
>
> *The apron invert is at EL 30.845 along the majority of its length. At the left-hand end it was initially set at EL 34.5 but this was later raised to a maximum of EL 37.5 depending on the quality of the rock found under the apron and on the excavation ledge immediately downstream. Details of the apron are shown of Drawings BDA-D-C-011 and BDA-D-C-013.*
>
> *A 1.0m high end sill is provided at the downstream end of the apron. In the hydraulic model study, however, testing of the possibility of incorporating the*

142 Exhibit 24, **GHD.002.0001**, .0056.
143 Exhibit 255, **HYT.514.006.0293**, .0305.
144 Exhibit 255, **HYT.514.006.0293**, .0305.
145 **TRA.500.013.0001,** .0072 ln 44 to .0073 ln 1.
146 **TRA.500.005.0001**, .0040 ln 27-44. Dr Maleki suggested that the particular task of combining the disciplines of hydraulic and geotechnical engineering to conduct a scour assessment would have to be done by a person specialised in scour: **TRA.500.011.0001**, .0017 ln 7-19.
147 Exhibit 237, **SWA.512.001.0578**, .0617.
148 Exhibit 237, **SWA.512.001.0578**, .0618.
149 Exhibit 24, **GHD.002.0001**, .0066.

downstream cofferdam into the stilling basin at the base of the primary spillway was conducted. Alternative end sill heights of 2.1m and 5.1m were tested but both proved to be an unsatisfactory arrangement due to the very high jet produced downstream from the end sill due to a 1H:1.5V sloping upstream face on the cofferdam.

6.73 Section 5.7.6 of the Detail Design Report includes:[150]

5.7.6 Dissipator Apron

5.7.6.1 Primary Spillway Apron

- *The functional requirements of the primary spillway apron are as follows:*
- *Protection to foundation immediately downstream of the dam to prevent erosion of the toe of the spillway*

Adequate energy dissipation during low flows. During medium to high flows the high tailwater will provide energy dissipation and erosion protection to the toe of the spillway

Two options have been considered for the dissipator apron design as follows, namely:

Option 1: Reinforced conventional concrete

Option 2: Reinforced RCC

…

6.74 The 2014 URS Review summarised the conclusion regarding the documentation of the dissipator structure as follows:[151]

1. ***The hydraulic design of the dissipator structure is not well documented in the Detailed Design Report*** *and there is no evidence of independent technical reviews being carried out on either the spillway design or the physical model study. This is a concern given the large PMPDF spillway discharges for this project.*

6.75 This criticism was raised with Mr Neumaier. He did not deny that the structure was '*not well documented*'.[152] The following exchange took place with Mr Herweynen in evidence:[153]

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150 Exhibit 24, **GHD.002.0001**, .0186.

151 Exhibit 237, **SWA.512.001.0578**, .0586 (emphasis added). The absence of independent technical review is dealt with at paragraphs 6.370-6.379 below.

152 **TRA.500.015.0001**, .0018 ln 31-47.

153 **TRA.500.013.0001,** .0076 ln 37 to .0077 ln 26 (emphasis added).

Q. I want to suggest to you that the detail design report doesn't document well your proposed dissipator structure; do you agree with that?

A. Now that I look at it, ***because it is one of those things that comes between the boundary of many different disciplines, and if I look at it, yes, there's portions in I think three different sections****. Is there somewhere which combines it all? I would say,* ***yes, that would be a lack****.*

Q. Because it comes at the boundary of disciplines, it's important that there be clarity from the designer about it, so that both the hydraulic engineer and the geotechnical engineer can work out whether, given both of the inputs, it's adequate to deal with, on the one hand, the energy and, on the other hand, the erodibility?

A. Yes, I understand that, and I'm not taking that I'm shifting things, but I would like to once again put on the record that that interface between those two was very strongly around the design manager and within Andreas Neumaier, and that same role that he played on Paradise I played on Wyaralong Dam, and, yes, there's a very different approach to that. There's lots of reasons for that. It's not just the difference of people. Things had changed. More emphasis was placed on that particular failure mechanism by the time we did Wyaralong Dam.

Q. Yes, but at what point do you, as the immediate reporter to Mr Neumaier, say to yourself, and now, "Well, in my position as principal designer, I should draw attention to that interface and ensure that the design manager attends to it"?

A. … we had plenty of discussions around the apron, and, yes, I agree with you that maybe we could have done more in terms of trying to have that dialogue and that interface better…

6.76 Mr Herweynen's evidence was in effect that: there was a 'lack' in documenting the dissipator structure; managing the 'interface' between the disciplines at the Dam was Mr Neumaier's responsibility; and more could have been done to engage those two disciplines.

6.77 Those things, however, did not materially affect the design choices for the apron and end sill.

Geological and geotechnical investigations

Introduction

6.78 Dam designers must manage geological and geotechnical investigations in selecting downstream protection measures such as an apron. Another input is hydraulic engineering.[154] The output – the '*judgment about where the apron needs to be*'[155] – should take account of advice from the two disciplines.

Early geotechnical investigations

6.79 Geotechnical investigations for preliminary design of the proposed Burnett River Dam began in late 2002. Conducted by the Queensland Department of Main Roads (**DMR**),[156] the investigations focused on foundations, sources of quarried rock for rock fill, as well as sources of embankment fill and concrete aggregates.[157] Logs of the rock cores recovered during the drilling program were later given to the tenderers.[158] Further investigations were planned.[159]

6.80 The early geological investigations informed SunWater's Preliminary Design Report.[160] The report stated that '*[a]dequate information*' had been collected to '*enable a geological model of the site to be constructed sufficient for preliminary design*'.[161]

6.81 During stage 2 of the tender process, each tenderer was '*given the opportunity to request additional boreholes*'.[162] A '*follow-up drilling program*' was conducted by DMR in which '*a selected number of the boreholes requested by the Alliance partners [tenderers] were drilled*'.[163]

6.82 In relation to the primary spillway dissipator, SunWater's preliminary design referred to the estimated tailwater and stated, '*[c]onsidering this, and the **reasonable erosion resistance of the riverbed**, a simple horizontal apron was considered sufficient for energy dissipation*'.[164]

6.83 The notion that the riverbed had a 'reasonable' erosion resistance found expression in the evidence. According to Mr Herweynen, the Alliance approached the stage 2

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154 **TRA.500.005.0001**, .0039 ln 30-37.

155 See paragraph 6.67.

156 Geological investigations were first carried out in 1966 by the Snowy Mountains Hydro-Electric Authority: **DNR.002.0001**, .0013. Low level reconnaissance was carried out in 2002 searching for construction materials: Exhibit 96, **DNR.003.7930**, .7942.

157 Exhibit 96, **DNR.003.7930**, .7942.

158 Exhibit 83, **MAR.001.0001**, .0002 [9].

159 The further investigations were to commence in around February 2003: Exhibit 96, **DNR.003.7930**, .7944.

160 Exhibit 96, **DNR.003.7930**, .7940 to .7942. A separate geotechnical report was attached to that document.

161 Exhibit 96, **DNR.003.7930**, .7942.

162 Exhibit 83, **MAR.001.0001**, .0002 [10].

163 Exhibit 83, **MAR.001.0001**, .0002 [10].

164 Exhibit 96, **DNR.003.7930**, .7971 (emphasis added). The source of this impression is not given.

tender process in the belief that *'everything was good downstream of the primary spillway'*.[165] Golder, he said, was asked to conduct an erodibility assessment and '*[t]here was never a strong indication that there was an issue downstream of the primary spillway*'.[166] The URS consortium approached its tender with a similar outlook.[167]

6.84 Before turning to the work that Golder undertook and the advice it gave, consideration is given to how erodibility is assessed.

Assessment of erodibility

6.85 Protection measures required downstream of a spillway are, according to Michael Marley, a former geotechnical engineer who was Golder's Project Manager for the Dam,[168] a function of two considerations: the condition of the rock and the hydraulic forces that are exerted on that rock.[169]

6.86 In assessing the susceptibility of rock to erosion and scour, dam designers used to rely heavily on precedents.[170] From the 1960s, however, more detailed erosion and scour studies were undertaken.

6.87 The most widely adopted method for assessing the erodibility of rock is that published by Dr George Annandale in 1996, commonly referred to the 'Annandale method'[171] or the 'Erodibility Index Method'.[172] According to Dr Pells, in Australia and the United States, '*the Annandale method is used almost ubiquitously on dams*'.[173] Mr Marley described the 1996 paper as one of the authoritative sources on erodibility.[174]

6.88 The Annandale method uses a rock mass index, which is a numerical way of characterising a rock mass as an engineering (as opposed to geological) material. Dr Annandale uses '*stream power dissipation*' to represent hydraulic loading and to combine that with the rock mass index.[175] Stream power dissipation is a means of characterising the erosive power of the flow of the water. The Annandale method is thus a semi-empirical technique that is used to quantify the threshold stream power

165 **TRA.500.013.0001,** .0074 ln 14-16.
166 **TRA.500.013.0001,** .0074 ln 19-25.
167 **TRA.500.012.0001**, .0082 ln 32-39: Mr Dann said, '*… before construction, there was a view that the materials in the bed of the river were founding on fresh rock, high-strength rock, reasonably resistant to erosion. That was the premise behind our design. I'm pretty sure it was the premise behind the other team's design as well. That's what was … inferred at the time based on the geotechnical investigation that had been done to date*'.
168 **MAR.001.0001**, .0001-2 [1] and [5].
169 **TRA.500.005.0001**, .0037 ln 43-46.
170 Exhibit 72, **STP.001.0001**, .0003 [11].
171 Exhibit 72, **STP.001.0001**, .0004 [15].
172 Exhibit 237, **SWA.512.001.0578**, .0790.
173 Exhibit 72, **STP.001.0001**, .0004 [15].
174 **TRA.500.005.0001**, .0038 ln 1-2.
175 Exhibit 72, **STP.001.0001**, .0003 [13].

of earth materials: that is, the power that is required to initiate scour of earth materials, such as rock.[176]

6.89 To apply the Annandale method, it is necessary to know the conditions of the rock in the area to be assessed as well as the hydraulic forces likely to impact that rock. The exercise requires hydraulic and geotechnical engineering expertise.[177]

6.90 More recently, other methods of predicting scour and erosion have been proposed. Dr Erik Bollaert researched whether high frequency pressure fluctuations in the water flow might unravel rock masses. His method was applied in relation to the Dam in 2016.[178]

6.91 The assessment of erodibility is not an exact science. Mr Herweynen observed that '*using the same raw data two geotechnical engineers could determine two different Erodibility Index*'.[179] Dr Pells explained that Dr Annandale had '*interpreted a single threshold for the onset of erosion*',[180] adding that '*different researchers reported different interpreted "thresholds" despite using largely the same data set for cases of rock erosion'*.[181] He said that this '*highlights some of the uncertainty and limitations in this methodology'*.[182]

6.92 GHD wrote:[183]

> *Erosion is a complex three phase (gas-liquid-solid) interactive problem, governed by a multitude of hydraulic, hydrodynamic and geotechnical phenomena that are strongly dependent on both time and space.*

6.93 Dr Maleki said that scour is '*probabilistic … not definitive*' and '*geology also can be called a probabilistic science*'.[184] Mr Lesleighter agreed that there '*will always be uncertainties, not only in the geology, although that's probably one of the primary ones'*.[185]

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176 Exhibit 237, **SWA.512.001.0578**, .0790.

177 **TRA.500.013.0001,** .0072 ln 44 to .0073 ln 1. See also Mr Herweynen's interview with the Commission: '*if you use the Annandale method or if you use the revised Annandale method, you have two elements of it - the erosivity of the rock and the energy within the flow*': Exhibit 247, **TRA.510.007.0001**, .0097 ln 38-40.

178 **DNR.002.0001,** from .1069.

179 Exhibit 245, **HER.002.0001**, .0004 [18].

180 Exhibit 72, **STP.001.0001**, .0004 [15].

181 Exhibit 72, **STP.001.0001**, .0004 [15].

182 Exhibit 72, **STP.001.0001**, .0004 [15].

183 Exhibit 228, **GHD.041.0001**, .0066.

184 **TRA.500.011.0001**, .0041 ln 15-18.

185 **TRA.500.012.0001**, .0009 ln 33-35.

Characteristics of rock materials

6.94 The condition of rock can be assessed by seismic refraction surveys in association with boreholes drilled into the rock.[186] On the erosion resistance of rocks, Mr Marley said:[187]

> *… the erosion resistance is not only to do with the strength of the rock; it's to do with the size of the blocks of rock that can be plucked out, if you like, and that is determined by the closeness of the jointing of the rock, the condition of the joints of the rock and whether or not they have been weathered to any degree*.

6.95 Weathering was explained by Mr Young.[188] Generally, the rock nearest the surface, when exposed to the elements for many hundreds or thousands of years, 'weathers'; that is, the chemical composition of the rock changes and it softens.[189] Over long periods of time, the rock will break down to soil. Fresh rock is the hardest rock. The degree of weathering affects the strength of the rock: Mr Young explained that '*it could be just a fraction of the strength of fresh rock or as much as maybe half, but rarely more than half if it is moderately weathered*'.[190]

6.96 There are different grades of weathering. Slightly weathered rock is characterised by a little discolouration on fractures, but it is '*almost as good as fresh rock*'.[191] Moderately weathered rock exhibits some '*serious decomposition of the rock fabric*'.[192] Such rock would be expected to be significantly weaker than fresh rock.[193] However, at the Dam site there was also a category of 'extremely' weathered rock, which is almost soil-like.[194]

6.97 A solid piece of rock is generally stronger than a slightly jointed rock, which is in turn stronger than sheared, or brecciated, rock.[195] A 'melange' is an area of poor quality rock that no longer has the properties of fresh rock:[196] it is a mixture of chaotically placed or arranged pieces of rock.[197] Melange may include areas of 'breccias' or 'shear zones'.[198] Dykes (intrusions into the surrounding bedrock, which are usually of limited width[199]) and faults also affect geological composition of rock.

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186 **TRA.500.005.0001,** .0036 ln 5-9.
187 **TRA.500.005.0001,** .0038 ln 41-47.
188 Exhibit 76, **YOJ.001.001.0001**, .0009 [29].
189 **TRA.500.005.0001,** .0005 ln 4-12.
190 **TRA.500.005.0001,** .0005 ln 16-19.
191 **TRA.500.005.0001,** .0005 ln 22-24.
192 **TRA.500.005.0001,** .0005 ln 24-25.
193 **TRA.500.005.0001,** .0005 ln 25-27.
194 **TRA.500.005.0001,** .0005 ln 27-29.
195 **TRA.500.005.0001,** .0006 ln 21-26.
196 Exhibit 76, **YOJ.001.001.0001**, .0005 [15].
197 Exhibit 76, **YOJ.001.001.0001**, .0005 [15].
198 **TRA.500.005.0001,** .0014 ln 7-10.
199 Exhibit 83, **MAR.001.0001**, .0020 [40(f)].

Faults

6.98 Mr Young reported on two fault zones identified after the 2013 event: the Paradise Fault and the Apron Fault zone.[200] The Paradise Fault runs '*right across the left side of the dam*' for about 400 m; the Apron Fault zone runs parallel to the Dam axis '*along the rock just downstream of the dam, mostly beneath the concrete apron*'.[201] The Apron Fault structures consist of two meandering 'thrust' faults that are oriented parallel to the axis of the primary spillway. These faults are not straight or curved lines; they are zones.[202] They exist below the Dam itself, apron and downstream areas.[203]

6.99 According to Mr Young, the erosion in 2013 '*started along the zone of the Paradise Fault and then got much worse when it reached the intersection of the two faults where the zone of poor quality rock was much wider*'.[204] Most of that area of rock at the intersection of the two faults was a melange.[205] Although the melange at the intersection of the fault zones may not have been visible during design, in Mr Young's view, it should have become evident during construction.[206]

6.100 It is not clear whether those fault zones were identified by Golder.[207] Mr Marley said that:[208]

> *A series of faults, joints and dykes was identified during mapping at time of construction (some at and near the locations of features now described as 'Paradise' and 'Apron' faults) resulting in areas of broken rock and soil, some of which we described as "brecciated" rather than Melange as described by John Young. This was a focus of foundation preparation in which our field geologist and engineer specified significant excavation and replacement with dental concrete under the Dam and apron.*

6.101 Mr Young noted that both faults '*were probably covered by concrete at the time of the 2013 flood*'.[209] He considered that the area of the Dam where concrete was placed to have been '*very well mapped*'.[210] It is possible that action was taken in response to features identified by Golder which may have corresponded to the Paradise and Apron Fault zones.

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200 Exhibit 76, **YOJ.001.001.0001**, .0004 [14].
201 Exhibit 76, **YOJ.001.001.0001**, .0004 [14].
202 Exhibit 76, **YOJ.001.001.0001**, .0004 [14].
203 Exhibit 13, **SUN.009.002.0001**, .0006.
204 Exhibit 76, **YOJ.001.001.0001**, .0005 [17]. Mr Young added, at .0009 [27], that '*[n]one of the geological information available to date means that the dam wall should never have been placed where it is*'.
205 Exhibit 76, **YOJ.001.001.0001**, .0005 [15].
206 **TRA.500.005.0001,** .0021 ln 35-42.
207 **TRA.500.005.0001,** .0036 ln 39 to .0037 ln 6; Exhibit 83, **MAR.001.0001**, .0025 [41(a)].
208 Exhibit 83, **MAR.001.0001**, .0025 [41(a)(i)].
209 Exhibit 76, **YOJ.001.001.0001**, .0005 [17].
210 Exhibit 76, **YOJ.001.001.0001**, .0010 [32].

6.102 Aside from their relevance to how the scour and erosion may have developed in 2013, it is not suggested that the faults mattered. The key issue is the erodibility of the area immediately downstream of the primary spillway apron. That requires examination of Golder's work.

Golder's geotechnical work

Scope and reporting structure

6.103 Golder was engaged to perform the geological and geotechnical mapping and studies for the Alliance from October 2003.[211] The extent to which Golder advised on conditions *downstream* of the primary spillway apron – in particular, where the significant scour occurred in 2013 – is controversial.

6.104 Golder's retainer did not mention the area downstream of the apron.[212] The Scope of Works was set out in Schedule 2 to Golder's engagement letter:[213]

> *b) Provide specialist geotechnical inputs into the design of:*
> - *Collection and collation of data and production of Geotechnical Model*
> - *Foundation design*
> - *Foundation grouting / seepage control*
> - *Support of temporary and permanent excavations*
> - *Basalt treatment*
> - *Foundation design for Rotec Conveyor system*
>
> *c) Provide specialist geotechnical input during dam construction into:*
> - *Mapping dam foundation*
> - *Advice on foundation treatment / levels*
> - *Field monitoring of excavation*
> - *Advice on excavation support support requirements*
> - *Advice on grouting / seepage control*
>
> *d) Provide Mike Marley for inputs into Alliance Management Team …*

6.105 Mr Marley and David Starr were identified as '*inputs to Alliance Management Team and Specialist Advice*'.[214] Mr Marley was the Project Manager of the Golder input to the Dam project[215] but was not often on site.[216] That responsibility fell to others.[217]

6.106 In February 2004, the scope of the retainer was extended to the '*ponded area*' of the Dam in order to consider potential areas of seepage loss, bank instability and erosion prone areas.[218]

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211 Golder undertook some work for the Hydro Tasmania consortium during the tender process. That work involved reviewing bore logs and other investigation data that had been developed by SunWater: **TRA.500.005.0001**, .0026 ln 11-24.

212 Exhibit 82, **GOL.002.0001**, .0018.

213 Exhibit 82, **GOL.002.0001**, .0018.

214 Exhibit 82, **GOL.002.0001**, .0019.

215 Exhibit 83, **MAR.001.0001**, .0002 [5].

216 **TRA.500.005.0001**, .0028 ln 39 to .0029 ln 6.

217 **TRA.500.005.0001**, .0029 ln 8-13.

6.107 Golder reported to Mr Neumaier.[219] Occasionally, Golder dealt directly with Mr Herweynen.[220] Mr Herweynen described this arrangement as '*unique in terms of the reporting structure*'.[221]

6.108 The results of Golder's work were shared with the Alliance partners[222] and that work was peer reviewed.[223]

Work performed

6.109 Golder received documentation setting out earlier geotechnical studies. This included URS's design proposal and SunWater's geotechnical studies. Early in the engagement, Mr Marley recommended that the DMR cores be 're-logged'. They gave insufficient detail satisfactorily to interpret the required parameters for determining the strength and deformation characteristics of the foundation materials.[224]

6.110 Golder developed an '*engineering geological model*' for the Dam site using VULCAN software.[225] The inputs included 59 cored boreholes, 47 open (air track) boreholes, 26 test pits, and 23 seismic refraction survey lines. Golder also prepared 137 Dam Foundation Inspection (**DFI**) reports covering the Dam's footprint and apron.[226] Each DFI contained a plan view of the area inspected by the geotechnical engineers (a 'mapping sheet') upon which the material strength and weathering characteristics of the rock were recorded, along with the location, type and properties of discontinuities.[227]

6.111 Golder issued a draft Geotechnical Design Report in March 2004.[228] Both parts of the geotechnical sections of the Detail Design Report were prepared by Brett Collins of

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218 Exhibit 233, **DNR.005.3464**, .3575. See also revised proposal at **DNR.005.5217**, .5329.

219 **TRA.500.013.0001,** .0072 ln 31-34.

220 Exhibit 83, **MAR.001.0001**, .0003 [14].

221 **TRA.500.013.0001**, .0073 ln 11-14.

222 Exhibit 83, **MAR.001.0001**, .0004 [14]

223 **SUN.245.003.0001** and Exhibit 232, **DNR.010.0929**, .0945. Patrick MacGregor, whom Hydro Tasmania described as a '*leading geologist*' (**HYT.008.0001**, .0160 [583]) peer reviewed 'engineering geology'. His report is dated 22 January 2004 and considered four 'issues': foundation strength; foundation permeability; foundation stability; and construction materials. It stated: '*The purpose of the review has been to check that the key project objectives are being met in the area of Engineering Geology and that issues relevant to the modification of the Stage 2 Design and subsequent detailed design are being considered. At the time of the review some geotechnical studies were in progress. Preliminary results of these studies were presented during the Peer Review meetings and have been incorporated in these comments. …The site investigations have located several narrow (<2 m wide) dolerite dykes intruded through the Goodnight Beds. Although the valley orientation appears to be controlled by geological structure no evidence of major faulting in the valley floor has been located*': **DNR.010.0929**, .0945-6.

224 Exhibit 83, **MAR.001.0001**, .0002 [11].

225 Exhibit 85, **DNR.006.3286**, .3296.

226 Exhibit 83, **MAR.001.0001**, .0009 [33(b)].

227 Exhibit 83, **MAR.001.0001**, .0009-10 [33(b)].

228 **DNR.003.7502**.

Golder and reviewed by Mr Starr. Appendix C was approved for issue by Mr Neumaier and Mr Hamilton, the Alliance Project Manager.[229]

Downstream geotechnical studies – general

6.112 Mr Young was asked about the '*specialist geotechnical inputs into the design*' undertaken by Golder.[230] He explained:[231]

> *Typically - well, one would do two levels of mapping here.* ***Normally, one maps downstream to a distance higher than the dam. If it's a 70-metre dam, you should be going 80, 90 metres downstream of the dam itself. That would be my normal practice****. That map would not be incredibly detailed, but it should be picking up things like the Paradise fault. That map would be done on something like 1 to 2000 scale, 1 to 1000 scale.*
>
> *The map under the foundation would be much more detailed and it would be done at a scale probably of 1 over 200 or even 1 over 100, depending how meticulous the geologist is.*
>
> *The mapping that I have seen by Golder is the highly detailed work done in the map footprint. You do this sort of detail because you know it is going to be covered with concrete and you are never going to see it again. So you want this much detail. The rock downstream you would do in less detail, because you can go back and look at it later. But I have not seen a map of, say, 1000 scale of this site.*

6.113 In Mr Young's view, mapping downstream to that extent would accord with good engineering practice. In Dr Pells's experience, however, geological mapping '*tends to be focused upon the area underneath the dam structure*'.[232] Dr Pells says that the unlined areas of dam spillways tend not to be examined closely '*until there is a problem*', adding that there is '*typically less attention paid to the rock mass of the unlined spillway and the area underneath the apron*'.[233] Mr Marley said that the extent of investigations downstream would be '*dictated by the designers and their hydraulics*'.[234]

What was Golder to investigate?

6.114 Mr Marley testified that Golder was '*not requested to extend the scope of its mapping outside the confines of the dam footprint by the Alliance contractor [SMEC]*'.[235] The 'dam footprint' for this purpose apparently included the area upon which the apron would be constructed, but not the area downstream of it.

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229 Exhibit 24, **GHD.002.0001**, .0687.
230 **TRA.500.005.0001,** .0015 ln 41-47.
231 **TRA.500.005.0001,** .0016 ln 6-27 (emphasis added).
232 Exhibit 72, **STP.001.0001**, .0009 [36].
233 Exhibit 72, **STP.001.0001**, .0009 [35].
234 **TRA.500.005.0001,** .0030 ln 2-4.
235 Exhibit 83, **MAR.001.0001**, .0024 [40(h)].

6.115 Neither SMEC nor Hydro Tasmania suggests that Golder actually *mapped* downstream of the apron.[236]

6.116 After the successful tenderer was chosen, the Alliance received documentation from the Thiess and URS consortium (the unsuccessful other tenderer for stage 2). According to Mr Herweynen, the Alliance then asked Golder to undertake an erodibility assessment 'downstream':[237]

> *… at the end of stage 2, we received the URS submissions, we saw in that particular document that they had put more emphasis on this erodibility downstream. So* ***at that point, Andreas [Neumaier] requested Golders to undertake their own independent assessment from the data to see whether erodibility downstream was an issue or not****…*

6.117 This exchange occurred between Senior Counsel Assisting and Mr Herweynen:[238]

> *Q. I'm just trying to understand. I want to suggest to you that there's no clear document in which Golders is told of the hydraulic energy and asked to prepare an erodibility based on that instruction?*
>
> *A. There is definitely an instruction from Andreas that said, "Can you please look at erodibility downstream?", and at that stage using the Annandale method. And I believe that Golders was ingrained in our team, they had access to the same thing as everyone else. They had access to the hydraulic reports, just like everyone else did, as they were produced. They were part of the design team. And they were shared and discussed at the design team meetings.*

6.118 No document before the Commission contains such a request from Mr Neumaier.

6.119 Mr Marley's evidence, however, accords with Mr Herweynen's recollection that some request was made, and that this occurred following receipt of URS's materials by the Alliance. Mr Marley stated:[239]

> *After the Report from competing bidders URS/Thiess was received, and* ***at the request of the Alliance designers Golder undertook a review of the erodibility of areas downstream of the dam.*** *We were provided with documents from that Report and a 1995 paper entitled 'Erodibility' by George Annandale (then of Golder Associates) referenced in the review.*

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236 Mr Marley says that even if Golder had been requested to do so, the presence of a downstream cofferdam (a temporary structure to provide protection of the works area from river flows during construction) would have inhibited its ability to map that area: Exhibit 83, **MAR.001.0001**, .0024-.0025 [40(h)]; **TRA.500.005.0001**, .0056 ln 30-40. The cofferdam downstream of the primary spillway was located about 10 m downstream of the apron's end sill from around chainage 250 to 500: **DNR.006.0001**, .0062-3. This covered, partially at least, the areas that suffered significant scour in the 2013 event: **DNR.006.3243**, .3248 and .3262.

237 **TRA.500.013.0001,** .0074 ln 16-25 (emphasis added).

238 **TRA.500.013.0001,** .0077 ln 28-39.

239 Exhibit 83, **MAR.001.0001**, .0004-5 [18] (emphasis added).

6.120 Mr Marley described that process as a '***review***',[240] which accords with Mr Herweynen's evidence that Mr Neumaier's request was to undertake an '*independent assessment **from the data***'.[241] That 'data' was the information obtained from the URS proposal and the Annandale paper.[242]

Outcome of Golder's 'review'

6.121 Mr Marley explained that the outcome of the review was a memorandum from Golder addressed to Mr Neumaier and Mr Herweynen:[243]

> *The review was undertaken by Cid Chenery (a Golder geologist) and formed the basis for a **Memo from Brett Collins to Andreas Neumaier and Richard Herweynen on 19 February 2004 the subject of which was 'Issues regarding potential for erosion downstream of Spillways'.** The Memo dealt primarily with the potential for erosion downstream of the secondary spillway and suggested that consideration be given to possible options for protective measures, but stated that design analysis would need to be carried out in order to assess practical treatments. The Memo also stated that the advice could also apply to the left abutment but that further consideration would need to be given to this area. A covering email for the Memo from Brett Collins stated inter alia 'It is considered that we need to review the adequacy of what protection measures are proposed'. Following issue of the Memo, on 20 February 2004, Brett Collins received a request by email from Andreas Neumaier requesting a copy of the Annandale paper which was later provided. I am not aware of any further action having been taken in relation to this issue.*

6.122 Hydro Tasmania submitted that Mr Neumaier's instruction to Golder was '*to assess the erodibility downstream of the apron*'.[244] As much may be accepted. The stated purpose of that memorandum was to:[245]

> *… review the adequacy of protection measures against the potential erosion of soils/weathered rock downstream of the apron, particularly in the areas between Chainage 720 m and 1020 m; and possibly on the left abutment.*

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240 **HYT.008.0001**, .0061 referring to Exhibit 83, **MAR.001.0001**, .0004-5 [18] (emphasis added).

241 **TRA.500.013.0001,** .0074 ln 16-25 (emphasis added).

242 Exhibit 83, **MAR.001.0001**, .0004-5 [18].

243 Exhibit 240, **DNR.020.019.3164**; Exhibit 83, **MAR.001.0001**, .0004-5 [18] (emphasis added). The memorandum is dated 19 February 2004. A memorandum regarding '*erodibility characteristics of the Goodnight Beds underlying the Primary Spillway Dissipator Apron*' appears to have been drafted by Golder. It is dated 24 November 2003: Exhibit 243, **GOL.005.0001**. In evidence, Mr Herweynen did not recall having seen this memorandum: **TRA.500.014.0001,** .0033 ln 10-23. Hydro Tasmania submitted that the evidence did not establish it had been sent: **HYT.008.0001**, .0060. Nothing turns on this memorandum.

244 **HYT.008.0001**, .0061.

245 Exhibit 240, **DNR.020.019.3164**, .3164. Chainage 720-1020 is located in the right abutment, where the secondary spillway is located: **DNR.006.0001**, .0015.

6.123 Golder was asked to consider the erodibility downstream and Golder did that. But Golder was not to map the area downstream of the primary spillway apron, nor subject it to focused study.[246]

6.124 Making plain that Golder was not undertaking its own geotechnical investigation but instead was confining its analysis to a review of the work of others, the memorandum said that Golder had:[247]

> *… **undertaken a review** of treatments proposed as part of Stage 2 works by both BDA and Thiess. The paper 'Erodability' (1995) by G.W. Annondale has also been reviewed (referred to in the Thiess submission).*

6.125 The memorandum continued:[248]

> *The following is a summary of discussions presented by both BDA and Thiess in their submissions:*
>
> 1. *The BDA Stage 2 submission includes the provision of an apron between 20m and 25m wide immediately downstream of the primary and secondary spillways; and training walls at both abutments. **There appears to be no detail for protection of natural stratum immediately downstream of the apron** (refer to drawing No. 207 & 208)*
>
> *…*
>
> 2. *The Thiess submission included estimates of hydraulic forces associated with various flow conditions, as well as an erosion protection threshold for the Goodnight Beds…**Their analysis indicated that within the primary spillway, for some flow conditions, significant erosion of weaker areas within the Goodnight Beds was possible and therefore it would be necessary to provide a downstream apron with down stream deflector (similar to the BDA design)**.*

6.126 Mr Collins's memorandum later summarises a number of 'issues'.[249] These relate only to the secondary spillway with the '*most vulnerable section*' being between Chainage 720 and 1020.[250]

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246 This finding is consistent with a 'temporary structure' (i.e. the cofferdam and access roads) then having covered at least some of the riverbed in the area immediately downstream of the primary spillway. If any instruction had been given to Golder, as Mr Marley said, it would not have been possible to map it due to the presence of the cofferdam: Exhibit 83, **MAR.001.0001**, .0024-25 [40(h)]. Such instructions would likely have precluded Golder undertaking an erodibility assessment immediately downstream of the primary spillway apron because it could not be done for the reason Mr Marley gave.

247 Exhibit 240, **DNR.020.019.3164**, .3164 (emphasis added).

248 Exhibit 240, **DNR.020.019.3164**, .3164-5 (emphasis added).

249 Exhibit 240, **DNR.020.019.3164**, .3165-6.

250 Exhibit 240, **DNR.020.019.3164**, .3165.

Was further work done in response to the memorandum?

6.127 Mr Marley said that Mr Neumaier had requested a copy of the Annandale paper referenced in the memorandum. That was provided. He said: '*I am not aware of any further action having been taken in relation to this issue*'.[251]

6.128 Hydro Tasmania submitted that the memorandum '*was discussed by Mr Herweynen with Mr Neumaier and a decision made to provide a cut-off wall as recommended by Golder and to install anchors in the apron*'.[252] That, however, was in response to a recommendation relating to a cut-off wall immediately downstream of the *secondary* spillway.[253]

6.129 There is no evidence of subsequent work being undertaken in relation to understanding possible '*weaker areas within the Goodnight Beds*' in the region of the primary spillway (referred to in the memorandum[254]).

6.130 This exchange occurred with Mr Marley and Senior Counsel for Golder:[255]

> *Q. Were you ever asked about the primary spillway, to the best of your recollection, in relation to this issue of erosion?*
>
> *A. Not specifically to my recollection, no, but there were other areas where we were asked, yes.*
>
> *Q. What were those areas that you remember?*
>
> *A. Specifically the right bank area between the secondary spillway and the diversion slot, which was the subject of that photograph that I was referred to earlier, and the secondary spillway and also to some degree on the left abutment.*
>
> …
>
> *Q. Had you ever been asked about the potential for erosion in relation to that area, that is, the area that eventually was badly eroded after the 2013 event?*
>
> *A. Not to my recollection, no.*

---

251 Exhibit 83, **MAR.001.0001**, .0005 [18]. Mr Marley expected the memorandum would have elicited a response: **TRA.500.005.0001**, .0045 ln 17. He said: '*This memorandum was produced and sent, and I would have expected that it would have elicited a response in terms of saying, "This is the sort of erosive force we're dealing with. Can we withstand it?", and I can't recall that ever having happened*'.

252 **HYT.008.0001**, .0060.

253 As Mr Herweynen recognised: Exhibit 244, **HER.001.0001**, .0025 [110]. See also Exhibit 240, **DNR.020.019.3164**, .3166. Hydro Tasmania submitted that '*This decision demonstrates that Mr Herweynen properly turned his mind and assessed the advice he was receiving and made design decisions accordingly*': **HYT.008.0001**, .0060.

254 Exhibit 240, **DNR.020.019.3164**, .3165.

255 **TRA.500.005.0001,** .0066 ln 28 to .0067 ln 3.

What did Golder advise in the memorandum?

6.131 What areas were the subject of investigation by Golder?

6.132 Golder submits it was '*never instructed to consider **in detail** the likely effect of the erosive force of water downstream of the primary spillway apron*'.[256]

6.133 An investigation involving focused study would have been necessary to reach a view about the ability of that area to withstand the erosive force of water. Golder submitted:[257]

> *Adequate advice as to whether the riverbed could have withstood the erosive force of the water there, required an understanding of what the erosive force of the water was to be: design and hydraulics determined that. Golder was never provided with that information – if it indeed existed in relation to the specific design ultimately adopted.*

6.134 According to Mr Herweynen, the outcome of the 'independent assessment' was that:[258]

> *… from that work, the prime areas that were of concern were downstream of the secondary spillway and downstream of the left abutment. **There was never a strong indication that there was an issue downstream of the primary spillway**.*
>
> *…*
>
> ***They [Golder] had assessed it that the erodibility downstream was unlikely**.*

6.135 What was requested of Golder and its advice must be considered in light of the documentary evidence. Mr Herweynen accepted that there was no single document produced dealing with erodibility[259] despite the fact that '*ordinarily*' one would expect to see such a document in a dam design.[260]

6.136 Concerns were raised by Golder about the *potential* for erosion downstream of the primary spillway apron at the Dam. However, nowhere is the erodibility of *that* area confronted and dealt with as the subject of particular and focused attention.

6.137 Golder's February 2004 memorandum does not conclude that '*erodibility downstream was unlikely*'. The memorandum reveals that, so far as erosion downstream was concerned, attention was focused on the area downstream of the *secondary* spillway. The area downstream of the primary spillway was only considered in a general sense and no instruction was given that this area be the subject of particular and focused attention.

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256 **GOL.006.0001**, .0004 [18] (emphasis added).
257 **GOL.006.0001**, .0006 [27].
258 **TRA.500.013.0001,** .0074 ln 22-34 (emphasis added).
259 **TRA.500.013.0001,** .0074 ln 6-8.
260 **TRA.500.013.0001,** .0074 ln 10-13.

6.138 That the focus of erodibility was on the secondary spillway is supported by Mr Herweynen's evidence. He said that an area of focus was:[261]

> *… the secondary spillway, the reason being, as we know, that the dam there is founded, and it was purposely founded, according to the specifications, on a more highly weathered rock than the primary spillway, and that rock is more fractured. Therefore, downstream of that, yes, it had some erodibility.*
>
> *In order to ensure that that didn't continue to undermine the structure, because that's the thing that we're interested in - erosion as such is not the issue; it's a question of whether it continues to progress to undermine the structure - we made a change to the design at that point to put - the end wall is not just an end wall, but it also goes into the foundation as a cut-off at that location.*

6.139 As Mr Marley noted, Golder also investigated the left abutment.[262] In a peer review of Golder's work from January 2004, two areas of '*potential instability*' were identified: '*a possible failed area on the upper left abutment downstream of the dam*' and '*potential movement of the basalt within the lower right abutment*'.[263]

6.140 Further, in June 2004, Mr Griggs asked Mr Herweynen and Mr Starr about protection requirements downstream of the *secondary* spillway. Mr Starr later forwarded the email to Mr Collins noting that, '*We have gone as far as possible with general advice on this matter. We have asked if SMEC want us to do an actual design of protective works'*.[264]

6.141 Mr Herweynen's assertion that the outcome of Golder's '*independent assessment*' was that '*erodibility downstream was [regarded as] unlikely*'[265] can only be correct if that was the outcome of Golder's work.

Commentary on erosion in Golder's Geotechnical Design Report

6.142 The outcome of Golder's work appears in section 2 and at appendix C to the Detail Design Report of June 2004.[266]

6.143 Golder described the general geology of the site as being characterised by '*[v]ery high strength meta-sedimentary rocks of the Goodnight Beds which underlie the whole of the site*'.[267] The 'site' in this context, according to Mr Marley, would:[268]

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261 **TRA.510.007.0001**, .0095 ln 14-41.

262 **GOL.006.0001**, .0005 [20].

263 Exhibit 232, **DNR.010.0929**, .0948. The recommendations made related to these areas of possible instability and a 'grouting program': Exhibit 232, **DNR.010.0929**, .0949-51.

264 Exhibit 83, **MAR.001.0001**, .0075.

265 **TRA.500.013.0001,** .0074 ln 33-34.

266 That appendix contains the Geotechnical Design Report. It was prepared in May 2004 and approved for issue in June 2004: Exhibit 24, **GHD.002.0001**, .0687.

267 Exhibit 24, **GHD.002.0001**, .0025.

268 Exhibit 83, **MAR.001.0001**, .0018 [40(b)].

> *… cover the area of construction works i.e. the dam wall, the diversion works and outlet works and would include associated temporary structures such as upstream and downstream cofferdams.*

6.144 The 2014 URS Review concluded that foundation mapping and treatment did not extend downstream of the apron end sill.[269] There is no document recording a considered assessment of the erodibility of the area downstream of the primary spillway.

Contest

6.145 There is a dispute about the meaning of Golder's advice concerning the susceptibility of downstream rock to erosion.

6.146 The Geotechnical Design Report contained a section entitled 'Erosion Protection' (section 8).

6.147 Mr Herweynen stated that:[270]

> *Golder considered erosion downstream of the apron but did not believe further protection was necessary beyond the 20m apron as is highlighted in section 8.3 of the Golder's Geotechnical Design Report.*

6.148 Golder, however, contends that '*section 8.3…could not, on any fair reading, support that conclusion'.*[271] It '*does not say that Golder* ***did not believe further protection was necessary*** *– indeed, it provides to the contrary, speaking of erosion and measures that would minimise that potential'.*[272] Golder submitted, on the basis of section 8 of its report, that '*[f]ar from advising further protection was unnecessary beyond the 20m apron, Golder squarely raised the proposition that erosion protection measures would be required'.*[273]

Content of section 8

6.149 Section 8.1 of Golder's Geotechnical Design Report, headed 'Background', states:[274]

> ***Protection measures will be required against potential erosion of soils/weathered rock downstream of the dam wall as a result of overflow events.***
>
> *Reference has been made to the Stage 2 submissions by BDA and Thiess and also to technical literature (Annandale G.W., "Erodibility, Journal of Hydraulic Research, Vol 33, 1995 No.4).*

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269 Exhibit 237, **SWA.512.001.0578,** .0616.
270 Exhibit 247, **HER.001.0001**, .0023 [100].
271 **GOL.006.0001**, .0002 [10].
272 **GOL.006.0001**, .0002 [10] (emphasis in original).
273 **GOL.006.0001**, .0002 [11].
274 Exhibit 24, **GHD.002.0001**, .0760 (emphasis added). This is the complete section 8.1 of the report.

The paper by Annandale indicates that for erosion to occur, the energy dissipation of the hydraulic flow must be greater than the impacted materials resistance to erosion. The paper provides energy dissipation formulae for four differing flow patterns which include open channel flow, changes in bed slope, hydraulic jumps and "heat cutting".

A material's resistance to erosion is a function of its "Erodibility Index". This index is a product of several geological parameters which can be measured in the field and the relationship between erodibility index and energy dissipation is presented graphically based on 150 field observations.

The main downstream area is discussed in succeeding sections. The main area downstream of the secondary spillway on the right bank (where the structure is founded at shallow depth within the extremely to distinctly weathered Goodnight Beds).

6.150 Section 8.3 of the report, entitled 'Main Spillway', states:[275]

The main spillway is founded in slightly weathered to fresh Goodnight Beds underlying the surface alluvium deposits. These materials, although relatively closely jointed, are generally of high strength and erosion resistant.

Design features which assist in minimising the potential for erosion include:

- *founding the spillway and apron in slightly weathered to fresh Goodnight Beds;*
- *a stepped downstream face on the spillway which assists in energy dissipation of spillway discharges;*
- *provision of downstream stilling basin (reinforced RCC apron and end sill) which will create a 'roller' effect in the downstream tailwater.*

6.151 The geotechnical advice on 'Erosion Protection' was summarised in section 2.8 of the Detail Design Report. It stated:[276]

***Protection measures are required** against potential erosion of soils/weathered **rock downstream of the dam structure** as a result of overflow events.*

Rival contentions

6.152 Although section 8.3 refers to the primary spillway being '***founded***' in materials that are said to be '*erosion resistant*',[277] Hydro Tasmania submitted that '*Golder provided*

[275] Exhibit 24, **GHD.002.0001**, .0761-2. This is the complete section 8.3 of the report.

[276] Exhibit 24, **GHD.002.0001**, .0031 (emphasis added). Section 8.3 of the Geotechnical Design Report was included at section 2.8.1 in the same terms except for the addition of this sentence: '*High tailwater levels for high unit discharges, which protect again the water jet impacting on the riverbed rock*'; and the stepped spillway was said to assist in energy dissipation for '*low unit discharges*': **GHD.002.0001**, .0032.

advice about the area immediately downstream of the apron'.[278] It said of section 8.3:[279]

> *… the design features were features to protect against the prospect of erosion in the downstream area, and so we had a geologist who gave us advice in 8.1 and 8.3 and then told us what the design features were.*
> *…*
>
> *… [Hydro Tasmania] … did receive advice about erodibility of areas downstream of the dam and was told what design features to install in that area.*

6.153 Mr Neumaier interpreted section 8 in much the same way. This exchange occurred during his evidence:[280]

> *Q. When you were designing this dam, did you have regard to what the geology immediately downstream of the apron might be able to withstand in terms of the energy of the hydraulic force coming over the dam wall?*
>
> *A. Only as far as the advice given from the geologists was that the riverbed consists of essentially Goodnight siltstone, sandstone, metamorphosed rock, which would be resistant to erosion.*

6.154 On this basis, SMEC submitted that:[281]

> *… the design of the apron and end sill was supported by geological testing that concluded that the geology in the area was high strength and resistant to erosion. At the relevant time, Mr Neumaier understood that there was no problem with the geology downstream of the dam such that, accordingly, it was not a focus of attention. That assumption was incorporated into the Detail Design Report (Exhibit 24; GHD.002.0001, .0569 at section 2.3.5.3 "With good quality rock expected in the excavation downstream of the apron").*

6.155 Hydro Tasmania submitted that Mr Neumaier received advice from the geologists '*that the riverbed immediately downstream of the apron would be resistant to erosion*'.[282]

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277 Exhibit 24, **GHD.002.0001**, .0761 (emphasis added).

278 **HYT.008.0001**, .0057.

279 **TRA.500.016.0001**, .0084 ln 43 to .0085 ln 5.

280 **TRA.500.015.0001**, .0016 ln 46 to .0017 ln 6.

281 **SMC.001.0001**, .0012 [30]. SMEC's reference to section 2.3.5.3 is in fact a section within Appendix D to the Detail Design Report, entitled 'Hydraulic Model Study Report for the Dam, Outlet Works and Fishway'. That was not a report by a geotechnical engineer; it was Mr Wallis's report.

282 **HYT.008.0001**, .0058 [189].

6.156 Golder's interpretation of section 8 is different. It submitted:[283]

> *Reliance is placed by Mr Herweynen on section 8.3 of Golder's Geotechnical Design Report issued in June 2004, however that report could not, on any fair reading, support that conclusion. In particular, the report does not say that Golder* ***did not believe further protection was necessary*** *– indeed, it provides to the contrary, speaking of the risk of erosion and measures that would minimise that potential.*
>
> …
>
> *Furthermore, section 8.3 should not be read in isolation. The entire section 8 is devoted to erosion protection. Its opening sentence is that "protection measures will be required against the potential erosion of soils/weathered rock downstream of the dam as a result of overflow events". Far from advising further protection was unnecessary beyond the 20m apron, Golder squarely raised the proposition that erosion protection measures would be required.*

Meaning of section 8

6.157 Golder's interpretation of the meaning of section 8 is, as a matter of English usage, correct. The interpretation that Mr Herweynen and Mr Neumaier would put upon section 8 ignores the caution in 8.1 that '*protection measures are required*' to deal with the susceptibility of '*rock downstream*' to erosion. They misread what Golder had written, perhaps affected by earlier reports of others that had assumed that the riverbed was generally resistant to erosion. For example, SunWater's Preliminary Design had noted the '*reasonable erosion resistance of the riverbed*'[284] and the associated initial hydraulic model study stated '*[w]ith good quality rock expected in the excavation downstream of the apron…*'.[285] Moreover, in a Peer Review of Foundation Adequacy in April 2004, Brian Shannon (Chief Design Engineer at SunWater) commented:[286]

> *Erosion of the riverbed downstream of the primary spillway will be minimised due to two factors- (a) steps on the spillway at low flows and (b) high tailwater at significant flows.* ***No doubt some erosion of the rock downstream of the dissipator apron will occur initially with little consequence.*** *Because overflows will be regular, it is desirable to try to minimise on-going erosion by identifying adverse geometry of the rock and/or suspect jointing and by treating the areas appropriately either by anchored backfill or concrete blanketing. Such a proposal has been drafted and I would support its adoption.*

6.158 Golder squarely raised the '*proposition that erosion protection measures would be required'.*[287] Although Golder, in section 8.1, summarised the effect of Dr Annandale's paper on erodibility, no erodibility assessment was carried out in respect

283 **GOL.006.0001**, .0002 [10] and [11] (emphasis in original).
284 Exhibit 96, **DNR.003.7930**, .7971 (emphasis added).
285 Exhibit 24, **GHD.002.0001**, .0569.
286 **SUN.245.003.0001**, .0006 (emphasis added). The source of this suggestion is not apparent.
287 **GOL.006.0001**, .0002.

of the area downstream of the primary spillway apron.[288] That was a summary of the paper only: no prediction was made regarding the potential for scour immediately downstream of the primary spillway apron. Golder's Geotechnical Design Report makes only a brief mention of scour, and only in relation to the secondary spillway: '*[t]he potential for erosion and scour is ameliorated by the presence of tailwater downstream*'.[289]

6.159 Accordingly, Golder's reference in section 8.3 to 'erosion resistant' materials is not to be understood as an assessment of the erodibility of the rock downstream of the primary spillway apron. Nor was it an endorsement of a 20 m wide apron.

6.160 The extent to which the geotechnical studies influenced the design of the apron first requires consideration of the hydraulic aspects of the design.

Background to design of the primary spillway apron

General approach

6.161 The '*functional requirements*' of the primary spillway apron were to prevent erosion and provide adequate energy dissipation.[290] This part discusses hydraulic aspects of dissipator design and why the designers chose an apron width of 20 m.

6.162 Important design considerations include:

a. The hydraulics of the water plunging over a spillway: the hydrology of anticipated floods;[291] the length of the hydraulic jump; and the tailwater level.[292]

and

b. The geology of the area which water spilling over the spillway is likely to affect.[293]

6.163 At the Dam, the very large catchment area produces significant discharges: '*extremely large*', Mr Lesleighter said.[294] For floods of the magnitude to be expected,

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288 **TRA.500.013.0001,** .0072 ln 44 - .0073 ln 1: Mr Herweynen said, '*There's two parts of the [erodibility] equation. One is the energy, and if we use the Annandale method, which is what was undertaken, the energy comes from the hydraulic engineer, and then the erodibility index comes from the geotechnical engineer*'. Perhaps Mr Herweynen supposed that reference to Dr Annandale's method meant that Golder had applied that method, which it had not.

289 Exhibit 24, **GHD.002.0001**, .0761.

290 Exhibit 24, **GHD.002.0001**, .0186.

291 Exhibit 238, **TRA.510.019.0001**, .0008 ln 22-24.

292 Exhibit 238, **TRA.510.019.0001**, .0008 ln 20.

293 Exhibit 72, **STP.001.0001**, .0002: '*Dams have always been designed to mitigate spillway erosion. Designers have to exercise their judgment to decide when the rock mass is of sufficient capacity to withstand the anticipated hydraulic loading'*. See also the interview with Michael Wallis, **TRA.510.024.0001**, .0008 ln 32-40: '*First, when one is asking about the length, is it true that one also has to have an appreciation of what the geotechnical and geological conditions are like immediately below where the apron is or is going to be? MR WALLIS: Yes, to ensure that once you've dissipated your energy, the velocity of the water downstream of the apron is not going to cause erosion'*.

designers would approach this task with caution; even, perhaps, conservatism. As Mr Lesleighter explained:[295]

> ***You always take some conservatism in a design****, but you're really looking at probabilities and risks and so on. But you're also looking, on the other hand, at something that is going to work; and you're looking at costs, you're looking at time to build it, et cetera. All of that has to be brought into the mix.* ***But the guiding thing of it all is you don't want to lose the dam, you don't want to have an unsafe situation.***
>
> *So if you have a condition which could threaten the dam if such an event occurred, you would have to have some conservatism built in. You might have to have a longer basin or a deeper basin or thicker concrete or better-quality concrete or something of that nature. You might have to do that, to make sure that if you get the erosion beyond that part, the dam is not going to be called into question. Therefore, if you had a longer basin, for example, of, say, 60 metres long and you had deep erosion, you might say, 'Well, that's fine, the dam is going to be safe. We might undermine the apron, but it's going to take several events like that before it ever gets back to the dam'. Someone might say, 'But we don't want to lose the spillway, either, because that's going to cost a lot of money to replace it'. Therefore, you look at the ways of protecting that spillway - armoury, cut-offs that we mentioned earlier, et cetera.*
>
> ***But whatever you're doing, you come up with a structure and a dissipator which is going to protect the dam at all costs and not lead you into some sort of major expensive repairs****.*

Focus on energy dissipation

6.164 Achieving adequate energy dissipation from the dissipator structure matters.

6.165 Mr Lesleighter said that '*you would want to achieve a large percentage of the total energy as being dissipated before it ever is released out into the natural material*'.[296] If the hydraulic jump extends beyond the apron, a designer is '*heavily reliant on the resistance of the rock downstream to erosion*'.[297] Dams are not designed to be erosion-proof during all possible flood events. The 'guiding' principle, however, is that the dissipator structure must '*protect the dam at all costs*'.[298]

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294 **TRA.500.012.0001,** .0012 ln 31. The Detail Design Report recorded the design flood as 93,457 m^3/s whilst the largest flood ever to occur on the Burnett River (the PMF) was estimated to be 104,377 m^3/s: Exhibit 24, **GHD.002.0001**, .0008. The table showing the expected discharges appears at paragraph 6.17 above.

295 **TRA.500.012.0001**, .0044 ln 18-44 (emphasis added).

296 **TRA.500.012.0001**, .0009 ln 8-11.

297 **TRA.500.006.0001**, .0014 ln 13-14.

298 **TRA.500.012.0001**, .0044 ln 2-44.

6.166 The length of the hydraulic jump is significant in the design of an energy dissipator. Calculations can be performed using the USBR Monograph to arrive at an estimated width. The Monograph states that:[299]

> *Stilling basins are seldom designed to confine the entire length of the hydraulic jump on the paved apron as was assumed in Section 1; first, for economic reasons, and second, because there are means for modifying the jump characteristics to obtain comparable or better performance in shorter lengths. It is possible to reduce the jump length by the installation of accessories such as baffles and sills in the stilling basin.*

6.167 GHD explained:[300]

> *Hydraulic jump stilling basins are the most commonly used energy dissipators. However, to ensure proper performance and energy dissipation, stilling basin should be sized to contain the roller of the hydraulic, preventing potential downstream erosion. The stilling basin is typically concrete-lined to protect the riverbed.*

The origin of the 20 m apron: the Preliminary Design for the Dam

SunWater's Preliminary Design (March 2003)

6.168 SunWater was engaged by Burnett Water to carry out a preliminary design and geotechnical investigations to provide information for potential alliance partners.

6.169 A request for proposals (stage 1) was issued in March 2003.[301] Preliminary design reports were appended.

6.170 Burnett Water did not warrant the completeness, accuracy or adequacy of the investigation results or the Preliminary Design, which were stated to be provided for information only.[302] '*Burnett Water [was] not committed to the preliminary design*'.[303] It added that '*Alliance Partners*' would '*assume responsibility for the total design of the Dam Project to meet Burnett Water's requirements*'.[304]

6.171 A proposal for a 20 m apron with an end sill first appeared in that Preliminary Design Report; and it was influential.

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299 Exhibit 235, **PDI.064.0001**, .0034.

300 Exhibit 228, **GHD.041.0001**, .0059. The 'roller' is explained at paragraph 6.235 below.

301 Exhibit 250, **SWA.500.001.2366**. The document sets out the objectives for the dam and the commercial framework for the alliance. Following the submissions by Team 1 and the URS/Thiess group, as well as a consortium led by Leightons, a stage 1 assessment was carried out: **SUN.506.002.0542**.

302 Exhibit 250, **SWA.500.001.2366**, .2379.

303 Exhibit 250, **SWA.500.001.2366**, .2379.

304 Exhibit 250, **SWA.500.001.2366**, .2379.

6.172 Mr Herweynen said:[305]

> Q. *To understand the genesis, the origin of your 20 metre apron, is that an assessment which you yourself did based upon hydraulic and geotechnical advice?*
>
> A. *Not myself, no. The starting point of that genesis was the Sunwater preliminary design, which was 20 metres, and then associated with that in stage 2 design, which was the competitive stage, we undertook hydraulic model studies, and there was some geotech information in terms of developing the model downstream, both under the foundation and downstream. Based on that, yes, we then concluded 20 metres was sufficient.*

6.173 SunWater had suggested a '*simple horizontal apron*' with an end sill. The end sill assists in the creation of the hydraulic jump.[306]

6.174 SunWater considered that apron design sufficient for energy dissipation for two reasons:[307]

a. the high tailwater ('*[t]he sequent depth for the full range of discharges over the spillway was significantly less than the corresponding tailwater level indicating that if a hydraulic jump were to form it would do so at the toe of the spillway blocks*')

and

b. what was assumed to be the '*reasonable erosion resistance of the riverbed*'.

6.175 The apron's dimensions were '*nominally taken*' as 1.5 m deep and 20 m wide, with a 1 m high end sill. The report added that '*[t]hese dimensions will be investigated further by model studies'*.[308] The apron was based on the USBR Monograph's Type II basin but had some '*slightly different characteristics*'.[309]

6.176 Mr Paton was involved in SunWater's Preliminary Design Report. He said that a '*check [was carried out] against the USBR guidelines for the design of a Type II stilling basin*'.[310] It was tested for flood events up to 1,000 m^3/s.[311] Those checks indicated that an apron length of '*17 metres or so*' would be required for that type of stilling basin to '*perform adequately*'.[312] The reason for only testing up to such (relatively small) flood events was, according to Mr Paton, a concern to ensure that

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305 **TRA.500.013.0001**, .0069 ln 27-37.
306 **TRA.500.006.0001**, .0014 ln 1-3.
307 Exhibit 96, **DNR.003.7930**, .7971.
308 Exhibit 96, **DNR.003.7930**, .7971.
309 **TRA.500.006.0001**, .0011 ln 18-23. As noted at paragraph 6.59, it did not conform to a Type II basin.
310 **TRA.500.006.0001**, .0005 ln 45-46.
311 **TRA.500.006.0001**, .0005 ln 45-46.
312 **TRA.500.006.0001**, .0005 ln 47 to .0006 ln 2.

before the tailwater was established – that is, rose up[313] – the apron could be relied upon to dissipate the energy.[314]

6.177 As Mr Paton noted, significant reliance was placed on the high tailwater that was to be expected at the Dam:[315]

> *For this site, the tailwater appeared to be particularly high, unusually high. The tailwater for the larger events exceeded the spillway crest level, so that was not something that the design team had come across before in a dam. We considered it was quite unusual and there was … a high potential to assist with energy dissipation, so we approached the design attempting to consider that high tailwater.*

6.178 Reliance on high tailwater has been described as a 'trap' for dam designers.[316] This is because the primary spillway causes the water to '*plunge to significant depths into [the] tailwater*'.[317] The result is a body of water operating at high velocities within the tailwater,[318] and potentially near the surface of the apron or riverbed. This condition was identified by SunWater in a 2D model study.

SunWater's 2D model study

6.179 Along with Mohamed Amghar, in April 2003, Mr Paton conducted a 2D physical hydraulic model study of the proposed spillway crest and apron design.[319] The study was reviewed by Peter Richardson who sent the final report to the Burnett Water and Thiess and URS consortium on 27 May 2003.[320]

6.180 The model was at a scale of 1:75.[321] There were some limitations with this model, including that the Dam site had not, at that stage, been finalised.[322] In relation to the apron, the study stated:[323]

> *A simple horizontal apron nominally 20m wide, with a 1m high sill was adopted for the preliminary design due to considerations of the reasonable erosion resistance of the riverbed and high tailwater levels. These dimensions were investigated to determine an appropriate apron length for optimal energy dissipation within the apron area.*

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313 **TRA.500.006.0001**, .0013 ln 16.
314 **TRA.500.006.0001**, .0013 ln 9-13.
315 **TRA.500.006.0001**, .0004 ln 19-27.
316 **TRA.500.012.0001**, .0011 ln 36-41. The potential problems with reliance upon tailwater are dealt with in more detail at paragraphs 6.228-6.239 below.
317 **TRA.500.012.0001**, .0011 ln 43-45.
318 **TRA.500.011.0001**, .0021 ln 17-18.
319 Exhibit 236, **SUN.018.026.5653** and **SUN.018.027.0454.**
320 **SUN.018.027.0453.**
321 Exhibit 236, **SUN.018.026.5653,** .5654.
322 **TRA.500.006.0001**, .0009 ln 18-28. See also the section entitled 'Limitations of the model' at Exhibit 236, **SUN.018.026.5653,** .5656.
323 Exhibit 236, **SUN.018.026.5653,** .5657.

6.181 Testing was conducted for discharges of 4,059, 16,000, 22,444, 32,500, 42,281 and 72,877 m^3/s. The testing demonstrated, among other things:[324]

> *A 'drowned jump' formed for higher discharges. At the drowned jump condition, instead of achieving good jump-type dissipation by the intermingling of the upstream and downstream flows, the incoming jet plunges to the bottom and carries along the basis floor at high velocity as shown in Figure 8.* [Figure 8 is reproduced below as Figure 6.10.]
>
> *To confine the hydraulic jump within the apron for discharges up to 22,444 m^3/s (approx 1 in 1,000 AEP), the apron would need to be 26.7m long*.
>
> *The apron length would need to be greater than 50m to contain the hydraulic jump for discharge from 22,444 m^3/s to 72,877 m^3/s*.

6.182 Sill heights of 1, 2 and 2.6 m were tested and the following conclusion drawn:[325]

> *The results indicated that increasing the sill height would not provide enough energy dissipation for flows greater than 22,444 m^3/s to retain the hydraulic jump within the apron*.

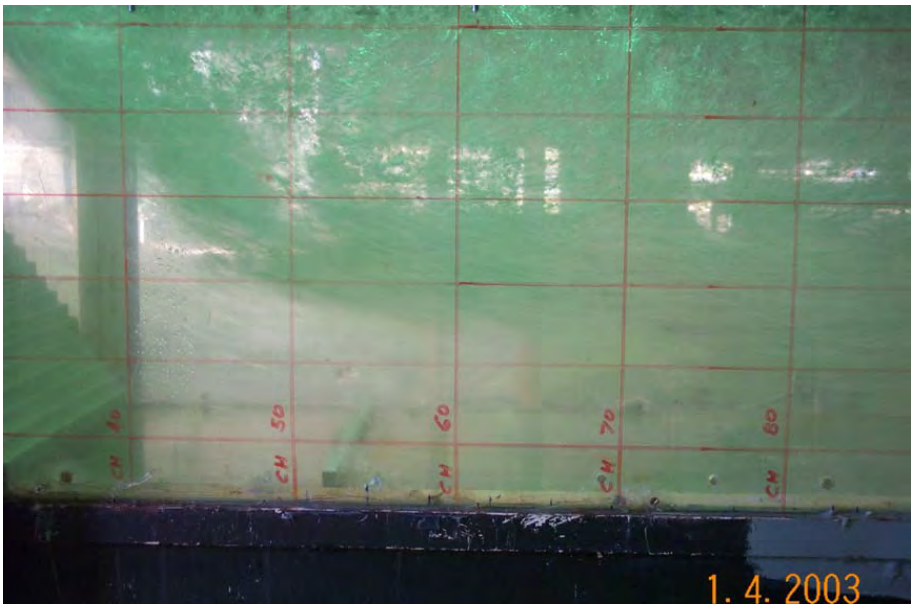

*Figure 6.10 – Image from SunWater's 2D model study showing the 'drowned jump' - the incoming water jet plunges to the bottom and carries along the basin floor at high velocity. The section of stepped spillway and end sill are visible. (**SUN.018.027.0454**, .0463)*

6.183 The '*unusually high*'[326] tailwater had influenced SunWater's hydraulic engineers to propose a 20 m apron. According to Mr Paton, *'[o]ne of the most relevant factors for*

324 Exhibit 236, **SUN.018.026.5653,** .5657.
325 Exhibit 236, **SUN.018.026.5653,** .5657.
326 **TRA.500.006.0001**, .0004 ln 20.

the design of an energy dissipation structure is the tailwater downstream of the dam.'[327]

6.184 The 2D physical model study cast some doubt on that reliance. The study revealed that:

- The hydraulic jump and the significant erosive power of the associated water would not be contained by a 20 m apron with a 1 m high sill.
- A drowned hydraulic jump would continue beyond 20 m along the riverbed at high velocity.

6.185 In recognising that a 'drowned jump' existed at higher discharges,[328] the study suggested that the riverbed immediately downstream of the apron may experience higher water velocities.

6.186 Mr Lesleighter explained that with 'submergence', the water jet '*doesn't just disappear as soon as it hits the water; it spreads and still retains quite a lot of its energy well down deep into the tailwater*'.[329]

6.187 Further model studies were to be undertaken by the tenderers and, eventually, the successful tenderer.

Further model studies

6.188 Burnett Water commissioned SunWater to construct and test three physical scale hydraulic models at SunWater's hydraulics laboratory.[330] The models included a 1:100 scale 3D model of the Dam, including the primary and secondary spillways, a 1:75 scale 2D model of a section of the primary spillway and apron, and a 1:30 scale model of the outlet works and fishway.[331]

6.189 The Detail Design Report explains that the model studies were conducted in two stages over an 11 month period:[332]

a. First, the 'Tender Design' stage between May and July 2003. During this stage, the 1:100 and 1:75 scale models were used.[333]

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327 **TRA.500.006.0001**, .0004 ln 13-15.

328 Exhibit 236, **SUN.018.026.5653,** .5657.

329 **TRA.500.012.0001**, .0052 ln 38-41.

330 Exhibit 231, **DNR.007.0477**, .0809.

331 Exhibit 24, **GHD.002.0001**, .0559. The function of the 'Outlet Works' was 'to provide required downstream releases for irrigation and environmental purposes' (Exhibit 24, **GHD.002.0001**, .0258). The 'fishway' was designed to 'enable the passage of fish past the dam in both the upstream and downstream directions' (Exhibit 24, **GHD.002.0001**, .0348).

332 Exhibit 24, **GHD.002.0001**, .0559.

333 According to URS's stage 2 tender 'Dam Design Report', when work commencing on the stage 2 bid process on 12 May 2003, a '*range of additional data that had been obtained by Burnett Water*' was provided. This included '*[t]he results of physical hydraulic model studies, including a*

b. Secondly, the 'Detailed Design' stage between October 2003 and April 2004. The 1:30 scale model of the outlet works and fishway was tested during this stage '*together with some additional testing on the 1:100 and 1:75 scale models*'.

6.190 Mr Dann, who was the design manager for the Thiess and URS consortium, explained that the SunWater physical models were:[334]

> *… set up for us to use to look at the design of the structure. Both models have their value, but when you are trying to make design decisions in a relatively short design period, I think we took the physical models as being a reasonable basis to inform the design.*

Tender proposals following SunWater's Preliminary Design Report

6.191 SunWater's Preliminary Design Report was the origin of the 20 m width of the primary spillway apron.

6.192 The stage 2 tender proposals dealt with initial design issues relating to the apron.

Hydro Tasmania consortium's proposal

6.193 A consortium consisting of SMEC, Hydro Tasmania, Walter Construction Group Limited (**Walter**) and Macmahon Contractors Pty Ltd (**Macmahon**) presented its proposal under the name 'Team 1'. It proposed a higher design head for the ogee crest (12.5 m[335]) than that in SunWater's Preliminary Design (9.8 m[336]). The Dam crest was at EL 67.6 m (as per SunWater's Preliminary Design).[337] The base was not widened, which resulted in a slightly steeper downstream face slope of 1V:0.64H.[338]

6.194 Team 1 proposed a 20 m wide apron, which was stated to have '*not changed from that proposed in the Preliminary Design*',[339] and a 1 m high end sill. The apron level was at EL 30.9 m with the left hand end sloping up to a 45 m long section at EL 34.5 m.

6.195 The 1:75 scale 2D and 3D models were inspected by Team 1.[340] The performance of the models was observed for discharges of up to about a 1:3,000 AEP flood (listed as 32,000 m^3/s in the report) for the 2D model and the PMPDF (listed as 100,000 m^3/s in the report) for the 3D model.[341]

3D model of the complete dam, a 2D model of the ogee crest at the spillway and a model of the outlet works and fishway': Exhibit 81, **DNR.007.1087**, .1120

334 **TRA.500.012.0001**, .0075 ln 14-19.

335 Exhibit 231, **DNR.007.0477**, .0550.

336 Exhibit 96, **DNR.003.7930**, .7953.

337 Exhibit 231, **DNR.007.0477**, .0529 and .0548.

338 Exhibit 231, **DNR.007.0477**, .0550.

339 Exhibit 231, **DNR.007.0477**, .0551.

340 Exhibit 231, **DNR.007.0477**, .0809.

341 Exhibit 231, **DNR.007.0477**, .0809.

6.196 The report of the model studies in Team 1's proposal noted:[342]

> *At the beginning of the Stage 2 design it* ***was realised that flow conditions at the toe of the spillway and at both abutments would be complex****, especially since the left and right abutments of the dam were to be lowered to allow overtopping. For this reason Team 1 decided to concentrate on testing of the 3D model and optimising the arrangement in preference to testing the 2D model. It was felt that greater gains in cost saving could be achieved for Burnett Water through optimising the dam alignment, main spillway length, abutment crest levels and outlet works location on the 3D model.*

3D hydraulic model testing

6.197 The 1:100 scale 3D model tests included tests to assess the merit of minimising the volume of rock excavated on the left abutment under and downstream of the apron: that is, the apron would slope upwards to the left abutment.[343]

Figure 6.11 – View of the 1:100 scale 3D model of the Dam from downstream. (Exhibit 231, ***DNR.007.0477****, .0821)*

6.198 The following design changes were made after the testing:[344] the primary spillway length was increased from 300 m to 315 m; the crest level of the secondary spillway was increased from EL 77.4 m to EL 78.0 m; and the crest level of the left abutment was increased from EL 77.4 m to EL 83.0 m.[345]

342 Exhibit 231, **DNR.007.0477**, .0809 (emphasis added).
343 Exhibit 231, **DNR.007.0477**, .0810.
344 They were not tested in the model studies but were proposed to be included in the Stage 3 design modelling test program: Exhibit 231, **DNR.007.0477**, .0820.
345 Exhibit 231, **DNR.007.0477**, .0810.

6.199 A number of different flood flows were tested in the 1:100 scale 3D model. These included the 1:10 AEP flood (approx. 4,059 m^3/s), 1:100 AEP flood (approx. 15,270 m^3/s), 1:500 AEP flood (approx. 23,210 m^3/s), 1:1,000 AEP flood (approx. 25,270 m^3/s), 1:3,000 AEP flood (approx. 32,000 m^3/s) and 1:12,000 AEP flood (approx. 53,000 – 57,000 m^3/s).[346] Testing of the PMPDF was not possible due to limited '*pump capacity*'.[347]

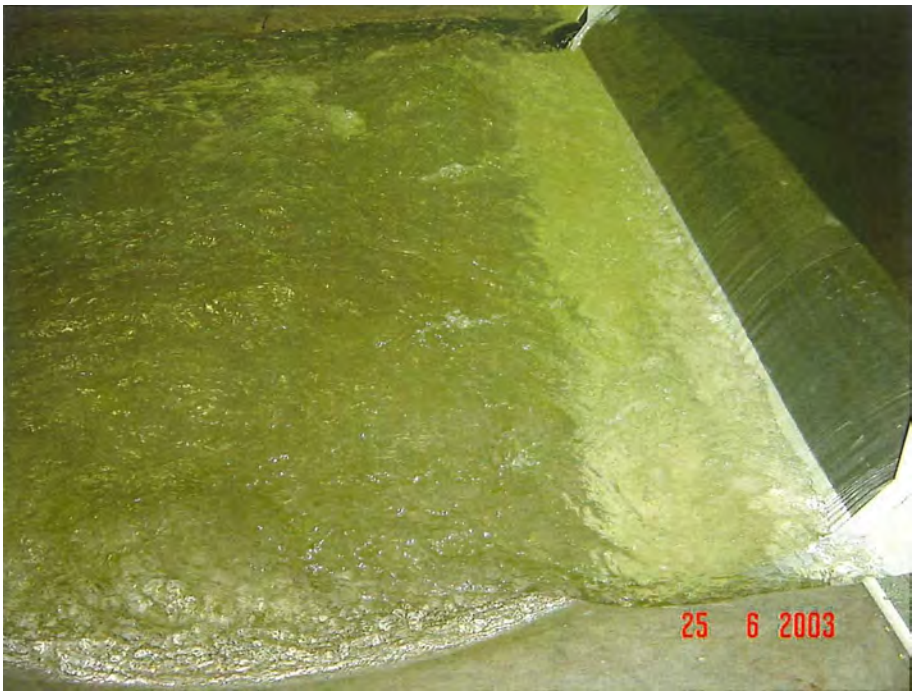

*Figure 6.12 – 1:100 scale 3D model in operation. Primary spillway in approximate 1:500 AEP flood. (Exhibit 231, **DNR.007.0477**, .0826)*

6.200 Comments were made in respect of each flow. Recirculation flows (in varying degrees depending on the section of the spillway and the discharge level) were noted as well as turbulent boils (again, in varying degrees; for example, the 1:1,000 AEP flood recorded an '*unstable boil on the right hand side of the spillway apron*'[348]). Due to the bend in the river near where the Dam was to be located, some '*slight intermittent and unstable waves were observed*' on the left side of the spillway.[349] However, they were said to be of '*no major concern to the overall performance of the spillway*'.[350]

346 Exhibit 231, **DNR.007.0477**, .0812-814.
347 Exhibit 231, **DNR.007.0477**, .0819.
348 Exhibit 231, **DNR.007.0477**, .0813.
349 Exhibit 231, **DNR.007.0477**, .0814.
350 Exhibit 231, **DNR.007.0477**, .0814.

6.201 The report stated that the changes made from SunWater's Preliminary Design had '*improved the performance of the spillway or the flow conditions in the river downstream from the dam and the overall operating performance of the dam'*.[351]

2D hydraulic model testing

6.202 A 2D model study was also conducted. The Preliminary Design model was removed from the test flume and a new 1:75 scale model constructed.[352] The model was based on the Hydro Tasmania's Consortium's proposed design of the ogee crest profile, transition curve profile, steps on the downstream face, apron and end sill.[353] 'Pressure tappings' were included in the model.[354]

6.203 The 2D model study conclusions included that the '***hydraulic jump is confined to the dissipator apron region***'.[355]

6.204 Pressures were measured on the ogee crest, downstream face and on the primary spillway apron (between 2 m and 18 m from the bottom step on the downstream face of the spillway).[356]

Conclusions reported from the 3D and 2D model studies

6.205 The Hydro Tasmania Consortium's report concluded that '*[d]issipation of energy on the spillway apron was generally considered to be good considering the flow over the spillway*'.[357] A 1 m high end sill was said to be '*preferred… from an energy dissipation perspective*' to a higher downstream cofferdam option with 3 m and 6 m high end sills.[358] A '*[r]ecirculation of flow'* on the left hand side of the spillway was noted but said to be '*less severe than on the right hand side due to less excavation into the abutment*'.[359]

6.206 No concern was identified regarding the width of the primary spillway apron.[360]

6.207 These studies were conducted at the proposal stage. So the tenderers had only 12 weeks in which to undertake a lot of work.[361] The issue of the submerged jet and drowned jump recorded in SunWater's 2D model study was not mentioned.

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351 Exhibit 231, **DNR.007.0477**, .0814.
352 Exhibit 231, **DNR.007.0477**, .0817.
353 Exhibit 231, **DNR.007.0477**, .0817.
354 Exhibit 231, **DNR.007.0477**, .0817.
355 Exhibit 231, **DNR.007.0477**, .0817 (emphasis added). The flow over the spillway was described as '*very smooth*'.
356 Exhibit 231, **DNR.007.0477**, .0818-819.
357 Exhibit 231, **DNR.007.0477**, .0819.
358 Exhibit 231, **DNR.007.0477**, .0819.
359 Exhibit 231, **DNR.007.0477**, .0820.
360 Exhibit 231, **DNR.007.0477**, .0820. The report states that, '*[t]he 20m wide spillway apron and 1m high end sill performed well in dissipating the flow downstream of the secondary spillway*'. This may have referred to the primary spillway.
361 See Exhibit 241, **DAC.001.0001**, .0005 [15].

The proposal of the Thiess and URS consortium

6.208 In the competitive tender process, Thiess/URS submitted an alternative proposal in July 2003.[362] The design was about 40% complete.[363] Mr Herweynen saw the URS proposal once the Alliance was selected as the successful tenderer.[364]

6.209 URS's primary spillway was '*in broad terms similar in concept to that ultimately constructed by the Alliance*'.[365] There were, however, some important differences according to Mr Dann:[366]

a. The primary spillway was to be 265 m long, aligned to the high flow channel of the river. The design rotated the spillway alignment from SunWater's Preliminary Design '*by approximately 3.3 degrees*' (meaning that the left abutment was to be moved approximately 30 m downstream).[367] This was done to reduce the potential for large flow recirculations on the right bank. The Alliance's primary spillway was 315 m long.

b. The apron was to be constructed of reinforced concrete, 600 mm thick and anchored into the rock foundation. It had a 1.8 m high ramped end sill. This was designed to 'lift' high velocity flow from the apron above the bed of the river to protect the downstream riverbed from erosion. The apron was to be 20 m wide. It sloped up at the left side from EL 31.0 m to EL 36.0 m near the left abutment to minimise excavation of hard rock[368]

c. The downstream (stepped) face was to be at a slope of 0.7H to 1.0V. The steps were to be 600 mm high. The spillway steps were expected to provide high energy dissipation (greater than 80%) for small floods up to 5,300 m^3/s. The URS proposal continued:[369]

> *As flood flows increase, the energy dissipation on the spillway steps gradually decreases. The steps provide approximately 50% energy dissipation in the 1:100 AEP flood and approximately 5% energy dissipation in the 1:1,000 AEP flood.*

6.210 URS also proposed a sloping apron.[370] The height difference from one end to the other was about 5 m.[371] This would '*minimise the excavation*'.[372]

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362 This is referred to as the 'URS proposal' in this part.
363 Exhibit 241, **DAC.001.0001**, .0003 [7].
364 **TRA.500.013.0001**, .0074 ln 38 to .0075 ln 40.
365 Exhibit 241, **DAC.001.0001**, .0008 [23].
366 Exhibit 241, **DAC.001.0001**, .0008-9 [23].
367 Exhibit 81, **DNR.007.1087**, .1129.
368 Exhibit 81, **DNR.007.1087**, .1125.
369 Exhibit 81, **DNR.007.1087**, .1130.
370 Exhibit 81, **DNR.007.1087**, .1105 and .1125.
371 Exhibit 81, **DNR.007.1087**, .1154.
372 Exhibit 81, **DNR.007.1087**, .1154.

6.211 The URS proposal emphasised the expected energy dissipation effects of high tailwater:[373]

> *In large floods the high downstream tailwater level provides a deep pool for energy dissipation of the primary spillway flow. Minimal energy dissipation works are required. The primary spillway design has a downstream apron, 20 m in length, with a 1.8 m high ramped end sill to ensure that drowned hydraulic jump occurs immediately downstream of the spillway. The ramped end sill also lifts the flow 'jet' above the bed of the river to minimise potential downstream erosion.*

It added:[374]

> *The assessment of energy dissipation on the spillway steps determined that there would be minimal requirement for an energy dissipation basin … Studies in the two dimensional hydraulic model testing showed that a 20 m long apron and 1.8 m high end sill would provide good hydraulic performance for managing energy dissipation across a wide range of flood flows.*

6.212 Mr Dann said that the '*unusually high*' tailwater levels provided comfort to URS in the tender stage.[375] He added that:[376]

> *We were of the view that the high tailwater level would drown out the dissipator and that you didn't need large energy dissipation works for that [PMF or PMPDF] flood. So the more frequent events were more critical in terms of erosion protection.*

Erosion protection considered in the URS proposal

6.213 The URS proposal included a section entitled 'Downstream Erosion Protection Measures' in which consideration was given to the potential for erosion downstream of the primary spillway.[377] Both the Annandale and Bollaert methods were noted as means of assessing erodibility. The Annandale Method was selected, and an assessment undertaken, based upon information available from the geological and geotechnical investigations undertaken at the proposal stage.[378]

6.214 The erosion assessment noted the '*complex flow*' over the primary spillway.[379] The physical hydraulic model studies showed that '*a strong current will exist along the bed of the river downstream of the dam if water is allowed to simply flow over the*

373 Exhibit 81, **DNR.007.1087**, .1130.
374 Exhibit 81, **DNR.007.1087**, .1153-4.
375 **TRA.500.012.0001,** .0089 ln 43-45.
376 **TRA.500.012.0001,** .0090 ln 1-9.
377 Exhibit 81, **DNR.007.1087**, .1193.
378 The Erosion Assessment is appendix 4E to the design report of the Thiess/URS consortium: Exhibit 81, **DNR.007.1087**, .1616.
379 Exhibit 81, **DNR.007.1087**, .1621.

spillway, plunge into the tailwater and flow downstream'.[380] This, it was said, would lead to potentially '*very high values of the erosive power of water*'.[381]

6.215 The erosion assessment indicated that the anticipated magnitudes of the erosive power of flow downstream of the primary spillway were '*unlikely to erode*' the slightly weathered Goodnight Beds rock that forms the underlying bedrock below the alluvium in the riverbed.[382]

6.216 The report recorded '*multiple design features*' that had been adopted to '*protect the primary spillway against scour and erosion*'.[383] These features included: founding the Dam and spillway apron in the slightly weathered Goodnight Beds – material that provides '*a stable dam foundation of high erosion resistance*'; installing a ramped end sill to deflect the flow into the tailwater, which '*decreases the strength of the currents moving along the bed and thereby minimises the magnitude of the erosive power of flow along the bed of the river*';[384] and providing a stepped profile on the downstream face of the primary spillway.

Expert review of the URS proposal

6.217 URS engaged experts to provide a technical peer review of its preliminary design report. The peer review covered hydrology, hydraulics, stability, foundation, gallery, RCC joints, outlet works, and reporting.[385]

6.218 In considering the design of the primary spillway, the panel observed (noting specifically the high tailwater):[386]

> *Rate of energy dissipation on the [spillway] steps is large and tail water elevations rise rapidly with increasing flow which combine to allow the use of only minimal energy dissipation facilities at the base of the spillway.*

6.219 The panel considered the risk posed by loose rocks downstream of the apron, especially where an asymmetric apron was proposed, adding that the design team was '*considering the possibility of sloping the basin upward on its left side in order to reduce excavation costs*'.[387] Although the panel could not point to an obvious reason why this could not be done, it recommended that the final design be developed using a 3D physical model '*to determine if special measures are needed to prevent rocks from being carried into the basin from downstream*'.[388] The panel commented that the right side of the basin may be faced with '*erosive velocities … downstream*'.[389]

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380 Exhibit 81, **DNR.007.1087**, .1621.
381 Exhibit 81, **DNR.007.1087**, .1621.
382 Exhibit 81, **DNR.007.1087**, .1193. In evidence, Mr Dann said that, at the time, he regarded the geotechnical investigations as '*comprehensive*' and that they '*provided a good understanding of the geological conditions at the dam site*': **TRA.500.012.0001,** .0092 ln 29-32.
383 Exhibit 81, **DNR.007.1087**, .1194.
384 Exhibit 81, **DNR.007.1087**, .1194.
385 Exhibit 81, **DNR.007.1087**, .1448.
386 Exhibit 81, **DNR.007.1087**, .1450.
387 Exhibit 81, **DNR.007.1087**, .1452.
388 Exhibit 81, **DNR.007.1087**, .1453.

The Alliance's design – Detail Design Report and criticisms of apron design

Background and responsibility for design

6.220 After the Hydro Tasmania Consortium's tender succeeded, work commenced on a detailed design. This culminated in a Detail Design Report in June 2004.[390] In writing that report, as Mr Herweynen said, reliance was placed on experts in '*sub-disciplines of expertise*' including Michael Wallis of Hydro Tasmania, an hydraulic engineer; and, Mr Starr of Golder. According to Mr Herweynen, Mr Wallis was '*responsible for all of the physical and numerical hydraulic modelling and hydraulic design*'[391] and Mr Starr '*led the geological and geotechnical assessments and analysis undertaken for the project*'.[392]

6.221 Mr Griggs had responsibility (under Mr Herweynen and Mr Neumaier) for the '*civil/structural design of the primary spillway apron*'.[393] In evidence, Mr Griggs said that the choice of the '*dimensions of the apron*' was based on advice: '*I took advice from the hydraulic designers on that 20 metre width*'.[394] That hydraulic designer was Mr Wallis.[395]

6.222 Mr Griggs was relatively junior. Mr Herweynen retained responsibility for the overall design of the Dam, including the apron, along with Mr Neumaier.[396] Mr Neumaier '*approved for issue*' the Dam's Hydraulic Model Study Report[397] (the **Hydraulic Model Study**) and reviewed the Hydraulic Design section of the Detail Design Report.[398]

6.223 The Alliance's dimensions for the apron had not altered since its tender proposal.

Hydrology

6.224 An hydrological model of the catchment was prepared as part of the Dam design. This included investigating anticipated rainfalls,[399] flood frequency analysis,[400] and 'critical durations', that is, the duration of a specific storm event which creates the largest volume or highest rate of water run-off.[401] The report noted that in all but one

389 Exhibit 81, **DNR.007.1087**, .1452.
390 Exhibit 24, **GHD.002.0001**, .0004.
391 Exhibit 224, **HER.001.0001**, .0009 [43(a)].
392 Exhibit 224, **HER.001.0001**, .0009 [43(b)].
393 Exhibit 287, **GRT.001.0001**, .0011 [61]; see also Exhibit 288, **TRA.510.009.0001**, .0037 ln 24-29.
394 **TRA.500.014.0001**, .0097 ln 15-22. In an interview, Mr Griggs said, '*as an input to my structural design, I was given that the apron should be 20 metres wide*': Exhibit 288, **TRA.510.009.0001**, .0040 ln 30-31.
395 **TRA.500.014.0001**, .0097 ln 20-22.
396 Exhibit 255, **HYT.514.006.0293**, .0305.
397 Exhibit 24, **GHD.002.0001**, .0554.
398 Exhibit 24, **GHD.002.0001**, .0056.
399 Exhibit 24, **GHD.002.0001**, .0040.
400 Exhibit 24, **GHD.002.0001**, .0041.
401 Exhibit 24, **GHD.002.0001**, .0046.

of the AEPs analysed, the critical duration was 48 hours with the exception of the AEP of the PMF, which was 96 hours.[402]

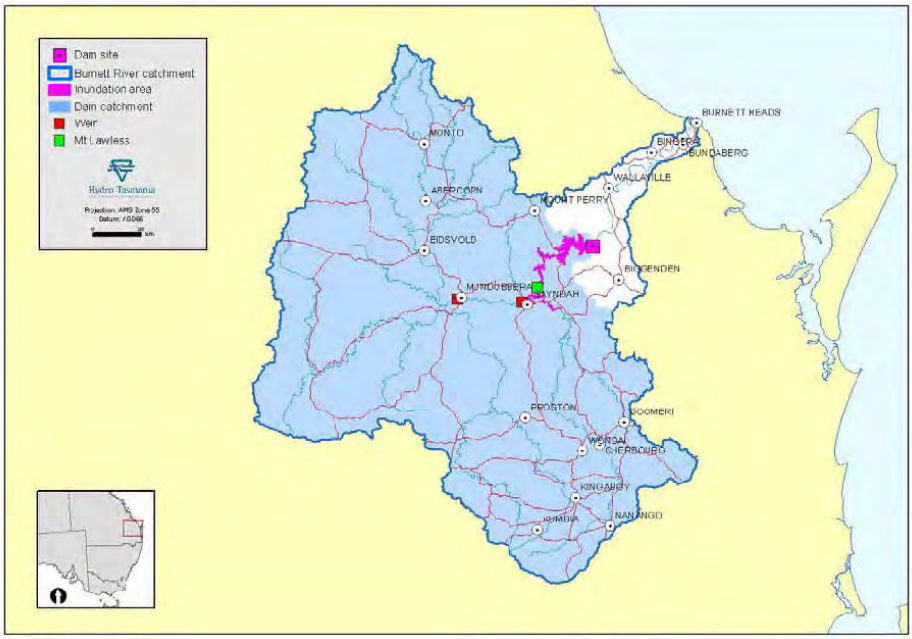

*Figure 6.13 – Overview of the Burnett Basin. The Dam's catchment comprises 'most of the area of the Burnett Basin'. (Exhibit 24, **GHD.002.0001**, .0050)*

6.225 A 'dambreak analysis'[403] was carried out as part of the flood risk assessment. The hydraulic dambreak modelling showed, among other things, that:[404]

> *… due to the low natural slope between the dam and the Burnett River mouth, floodwaters are relatively slow to discharge from the system, creating a backwater effect and high tailwater levels at Burnett River Dam.*

6.226 Flood exceedance probabilities were estimated showing peak inflows, discharges and levels (EL) for different AEP floods from 1:50 to the PMF. The Detail Design Report notes that the PMF '*adopts assumptions that are as extreme as can be reasonably expected*'.[405]

6.227 Thus, the hydrology assessment agreed with the initial estimates by a flood studies team[406] that had predicted that the tailwater at the Dam was '*high*'.[407] Indeed, it was, as Mr Paton said, '*unusually high*'.[408]

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402 Exhibit 24, **GHD.002.0001**, .0046.

403 A 'dambreak analysis' was '*carried out to assess flood risk to downstream residents and properties*': Exhibit 24, **GHD.002.0001**, .0008.

404 Exhibit 24, **GHD.002.0001**, .0049.

405 Exhibit 24, **GHD.002.0001**, .0046.

406 **TRA.500.006.0001**, .0006 ln 16-20.

407 The SunWater Preliminary Design Report referred to the '*high tailwater*': e.g., Exhibit 96, **DNR.003.7930**, .7964 and .7971.

408 **TRA.500.006.0001**, .0004 ln 19-20.

Was there over-reliance upon tailwater?

6.228 A Tailwater Rating Study was carried out.[409] The study was reviewed by Mr Wallis in January 2004. Its purpose was to '*determine a tailwater (or backwater) rating for Burnett Dam*'.[410]

6.229 The study was based on work carried out by SunWater and URS in 2003 and on hydrographic information from the Department of Natural Resources and Mines and the Bureau of Meteorology.[411] Mr Wallis signed off on the tailwater study noting that the '*[m]ost appropriate method*' had been adopted, '*given the limited calibration data available'*.[412]

6.230 The expectation of '*major floods*' and '*high tailwater*' was recorded in the Detail Design Report as follows:[413]

> *Due to the topographic and hydrologic conditions of the catchment (33,000km^2) and the river course and the relatively small storage volume,* ***major floods will pass with little attenuation through the reservoir and will result in high tailwater levels for long periods of time.*** *The high tailwater levels during floods were the main reason for not incorporating a drainage gallery into the dam design, which is unusual for a dam of this size*.

6.231 That assessment of tailwater was an important factor in the Alliance's decision to settle on the dimensions of the apron. Key references in the Detail Design Report to its effects include:

a. As a design feature which assists in minimising the potential for erosion: '*High tailwater levels for high unit discharges, which protect again the water jet impacting on the riverbed rock*'.[414]

b. The functional requirements of the primary spillway apron included, '*[a]dequate energy dissipation during low flows. During medium to high flows the high tailwater will provide energy dissipation and erosion protection to the toe of the spillway*'.[415]

6.232 The estimates of tailwater levels have not been criticised. However, in general, as Mr Lesleighter said, reliance on tailwater as having significant energy dissipation effects can be a 'trap' for dam designers.[416]

6.233 A report by Water Solutions NSW in August 2013 noted '*reliance by SunWater on high tailwater levels both in terms of cushioning the energy of the spillway flow and of*

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409 **DNR.010.0023**.
410 **DNR.010.0023**, .0028.
411 **DNR.010.0023**, .0028-32.
412 **DNR.010.0023**, .0024.
413 Exhibit 24, **GHD.002.0001**, .0010 (emphasis added).
414 Exhibit 24, **GHD.002.0001**, .0032.
415 Exhibit 24, **GHD.002.0001**, .0186.
416 Exhibit 238, **TRA.510.019.0001**, .0008 ln 34-40.

the beneficial effects on dam stability'.[417] The report stated, '*[o]ur aim is to emphasise the uncertainty of the tailwater effects*'.[418]

6.234 This 'uncertainty', or the 'trap', was explained by Mr Lesleighter:[419]

> *The words I'm using are 'a trap', because you think, 'We have such a large body of water there, the water coming over from the jet will spread and therefore you will have low velocities'. However, the jet coming over a spillway such as this will plunge, and it will plunge to significant depths into that tailwater. It's very simplistic to say, 'Well, the tailwater is there; therefore it's going to spread and everything down the bottom is going to be free of significant conditions'.*

6.235 Dr Maleki said that, where the hydraulic jump is submerged, the 'roller' cannot clearly be seen.[420] He explained:[421]

> *A. The roller indicates the length of the hydraulic jump. It shows the interaction of water and air, and it's an air bubble mixture that you can observe on the classical hydraulic jump and perhaps you can estimate the extent of your jump on the basis of that.*
>
> *Q. Is it right to understand the roller as part of the turbulence of the hydraulic jump that one needs to focus on, as a hydraulic engineer, in terms of looking at what erosive force the stilling basin or the geology needs to bear?*
>
> *A. If it is a classical hydraulic jump, that is a different type of jump. You can observe the roller and get an estimate of the length of the jump and length of the basin, and that can be used as guidance. In terms of a B type jump, that is a submerged jump, you cannot visualise the roller. The high-velocity jet can travel within the tailwater, and you cannot observe the roller.*

6.236 A submerged, or drowned, jump is not the same as an hydraulic jump 'drowned out': such that '*the high tailwater level would drown out the [energy]*'.[422] A submerged jump '*can still carry erosive force*'.[423] It is an unfavourable hydraulic condition.[424]

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417 Exhibit 5, **DNR.002.8498**, .8523. Dr Pells's view was that the '*effect of tailwater level on scour is not always clear*' largely due to uncertainty surrounding '*velocity distribution*' within the tailwater: Exhibit 72, **STP.001.0001**, .0013-14 [52].

418 Exhibit 5, **DNR.002.8498**, .8523.

419 **TRA.500.012.0001**, .0011 ln 37 to .0012 ln 1.

420 **TRA.500.011.0001**, .0020 ln 28-29. A 'B type' hydraulic jump is one '*that begins on spillway with a positive slope and ends on [a] flat stilling basin[]*': Exhibit 228, **GHD.041.0001**, .0059.

421 **TRA.500.011.0001**, .0021 ln 1-18.

422 **TRA.500.012.0001**, .0090 ln 6-7.

423 **TRA.500.011.0001**, .0021 ln 24.

424 **TRA.500.006.0001**, .0005 ln 33.

6.237 A submerged jump may mean that the tailwater is not providing sufficient energy dissipation. That the jump is submerged does not mean it has lost its energy. It might still cause riverbed erosion. This phenomenon was identified by SunWater in its April 2003 2D model study. The relevant finding based on that model testing was:[425]

> *A 'drowned jump' formed for higher discharges. At the drowned jump condition, instead of achieving good jump-type dissipation by the intermingling of the upstream and downstream flows, the incoming jet plunges to the bottom and carries along the basis floor at high velocity …*

6.238 According to Mr Lesleighter, that conclusion suggested '*that things are happening on the floor that are not just average, uniform conditions of low velocity. That might be of higher velocity, pressures*'.[426] Hence reliance on that tailwater might not be justified.[427] In relation to that finding in the model study, the following exchange occurred with Mr Herweynen:[428]

> A. *Well, we all knew that there was a high tailwater, so the drowned jump doesn't surprise me, and, yes, the water would plunge into that…I can't recall reading all those words right now, 15 years later, but I'm sure I did.*
>
> Q. *Did you read or do you recall reading the 'drowned jump' as meaning that the energy within that jump would dissipate because it was drowned in the tailwater?*
>
> A. *I did, yes, and I believe that* ***there was a lot of opinion at that stage that the high tailwater would dissipate the energy****.*

6.239 In its stage 2 proposal, URS also assumed that the high tailwater would provide good energy dissipation. Mr Dann said that:[429]

> *I suppose the focus with this project was the very large flood that had to be passed through the spillway, and that was probably the primary criterion when we were looking at the design there, of how you manage that. We concluded in a very large flood that the dissipator was drowned out due to very high tailwater level, but in more frequent floods, the energy that was coming out of the spillway was dissipated on the steps of the spillway, and then we had the end sill ramp to lift the flow up off the riverbed to dissipate the energy and manage the risk of erosion downstream.*

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425 **SUN.018.027.0454**, .0458. See at paragraph 6.181.
426 **TRA.500.012.0001**, .0055 ln 34-36.
427 **TRA.500.012.0001**, .0055 ln 38-40.
428 **TRA.500.013.0001**, .0071 ln 6-17 (emphasis added).
429 **TRA.500.012.0001**, .0058 ln 38 to .0059 ln 1.

The Alliance's model studies

Model studies: key conclusions

6.240 Three physical hydraulic model studies were carried out during the tender proposal and detailed design stages of the Dam project between May 2003 and April 2004.[430] The models included a 1:100 scale 3D model of the Dam, a 1:75 scale 2D model of a section of the primary spillway and apron, and a 1:30 scale model of the outlet works and fishway. Mr Herweynen said that the '*model was already prepared at the Sunwater labs, and we modified that model to suit our particular design*'.[431]

6.241 The results of the 3D model study reported in the Detail Design Report were these:

a. for the 1:10 and 1:50 AEP flood events, there was good uniform dissipation across the apron:[432]

> *… when the tailwater level was set correctly. However, when the spillway discharge was increased and the tailwater had not reached the correct level … a very high jet occurred downstream of the end sill.*

b. at low discharges, a slight jump in water level occurred immediately downstream of the end sill.[433] Good quality rock was, however, expected:[434]

> *With good quality rock expected in the excavation downstream of the apron, the jet flipping from the end sill should not cause any major erosion problems in the impact area during periods of low spillway discharges.*

c. in a 1:500 AEP event, the model showed *'[t]urbulent flow and boil over a 100m length of apron from the right hand end*' and '*relatively smooth tailwater conditions*'[435]

d. *'[s]ome slight intermittent and unstable waves were observed over the left hand side of the spillway*'.[436]

6.242 The significance of the identification of turbulent flow and 'boils' was raised with Mr Lesleighter. He said that:[437]

> *… those are things that you would expect, the boiling, turbulence, and so on. Then your next step would be to say, 'Well, have we really accounted for them in our design?' …*

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430 Exhibit 24, **GHD.002.0001**, .0557. See paragraph 6.188.
431 **TRA.500.013.0001**, .0070 ln 3-5.
432 Exhibit 24, **GHD.002.0001**, .0563.
433 Exhibit 24, **GHD.002.0001**, .0564.
434 Exhibit 24, **GHD.002.0001**, .0569.
435 Exhibit 24, **GHD.002.0001**, .0564.
436 Exhibit 24, **GHD.002.0001**, .0566.
437 **TRA.500.012.0001**, .0042 ln 12-15.

6.243 Mr Lesleighter explained that if the conditions were of concern to the hydraulic engineer, they should be identified in the conclusions. The scale of the model, however, might not be sufficient to suggest that there was a problem. He added:[438]

> *But if you go a step further, if you were to have a well-instrumented model, and also a well-scaled model, then you might come up with a different answer, and the different answer might be related to pressures, transient pressures and other pressures in the apron, and also the extent of the jump, because the extent of the jump would be telling you that conditions beyond the apron are significantly large in magnitude, velocities, and so on.*

6.244 A 2D model was also tested.[439] The report of the 2D model study concluded that the hydraulic jump was '*confined to the dissipator apron region*'.[440]

6.245 The report also described the flow conditions at the toe as 'complex'.[441] In an interview, Mr Lesleighter agreed, saying '*[y]es, that's for sure. It is complex, very much so. I think it's easy to miss the complexity of it or really sell it short*'.[442]

6.246 So far as it is relevant to the apron, however, the outcome of the model studies was that a '*hydraulic jump occurred on the apron for all flood events tested*' and that the 20 m apron '*performed satisfactorily*'.[443] Mr Herweynen said that the design team had relied on the conclusions contained in the Hydraulic Model Study.[444]

6.247 The Hydraulic Model Study, however, has been criticised.

Inadequate scale

6.248 According to Mr Lesleighter, the 3D model at 1:100 scale was inadequate to review the true behaviour of the energy dissipation process such that true flow behaviours would be 'masked'. An appropriate scale was important: as the apron was designed with an asymmetry, the flow over the apron did not perform in a two-dimensional manner.[445] It is '*very much three dimensional*'.[446]

6.249 Computational fluid dynamics (**CFD**) modelling was available at the time of the Dam's design but not to today's sophistication.[447] The use of physical hydraulic model studies alone was considered '*acceptable practice*' by Dr Pells, '*provid[ed] the physical modelling studies are appropriately undertaken*'.[448] Mr Lesleighter stated

438 **TRA.500.012.0001**, .0042 ln 45 to .0043 ln 5.
439 Exhibit 24, **GHD.002.0001**, .0059 and .0585.
440 Exhibit 24, **GHD.002.0001**, .0585.
441 Exhibit 24, **GHD.002.0001**, .0561.
442 Exhibit 238, **TRA.510.019.0001**, .0057 ln 2-4.
443 Exhibit 24, **GHD.002.0001**, .0066.
444 **TRA.500.013.0001**, .0077 ln 47 to.0078 ln 5.
445 Exhibit 238, **TRA.510.019.0001**, .0025 ln 20-25.
446 Exhibit 238, **TRA.510.019.0001**, .0041 ln 7-10.
447 **TRA.500.011.0001**, .0036 ln 31-34.
448 Exhibit 72, **STP.001.0001**, .0014 [55].

that, in 2004, there was '*not a great deal in CFD [tools] at that time … [t]ypically you'd go straight to the physical study*'.[449]

6.250 Mr Lesleighter described the importance of physical hydraulic modelling in these terms:[450]

> *… your model then has to be able to look at all the phenomena that's in that model. You've got to be able to look at the velocities coming in and the depths and the conditions coming in to the apron. Then you have to look at the energy dissipation that is going to take place, and you have to then identify what are the conditions in that apron which will tell us how to design the apron later on …*
>
> *The* ***model then has to have the sufficient scale to look at all those turbulent conditions and look at them relatively accurately****. Now, where you have a model, you normally design it to handle the gravitational flow conditions, which is captured in what we call the Froude number, but there is also viscosity. Water has viscosity. If you have what I call the Reynolds number too low, then you get a viscosity effect in the model.*
>
> *The 1 to 100 model would have had a fair amount of viscosity effect in it. I don't know what the Reynolds numbers would have been, but you normally would choose a Reynolds number above a certain limit, and then you can say, right, we have a model we can rely on here. There might be some interpretation, but we've got a model which is going to provide us data, we're going to make measurements and we're going to be able to believe them.*
>
> *Then going on from there, you've got the properly designed and executed model, you'll have the loading available for you for the structural designer, the structural engineer, to design the apron.*

6.251 A 1:100 scale 3D model, such as the one used by the Alliance, is '*subject to appreciable scale effects*'.[451] This would, among other things, '*mask the true behaviour of the dissipation process*'.[452] Mr Lesleighter expressed this concern in a report written after the 2011 flood damage.[453] The same report noted that:[454]

> *[n]ot only was the scale likely inadequate to detect the real flow behaviour, but it would be inadequate to properly study the entrainment and motion of rock in and around the basin sill – even if that was attempted.*

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449 **TRA.500.012.0001**, .0053 ln 26-31. Today, physical hydraulic models are still used to 'validate' CFD models: **TRA.500.011.0001**, .0014 ln 27-29 and .0014 ln 44 to .0015 ln 1. Mr Lesleighter thinks that '*CFD is going partway, but it's not the total answer*': **TRA.500.012.0001,** .0053 ln 30-31. According to Dr Maleki, both physical hydraulic and CFD models '*have limitations, so they complement each other in design*': **TRA.500.011.0001**, .0014 ln 44 to .0015 ln 1.

450 **TRA.500.012.0001**, .0021 ln 41 to .0022 ln 38 (emphasis added).

451 Exhibit 230, **DNR.006.3156**, .3241.

452 Exhibit 230, **DNR.006.3156**, .3241. See also **TRA.500.012.0001**, .0020 ln 30-41.

453 Exhibit 230, **DNR.006.3156**, .3241.

454 Exhibit 230, **DNR.006.3156**, .3241.

6.252 The masking of the true effects meant that the '*little bit of turbulence and boiling going on*'[455] reported may not have indicated that there was any 'problem' with those matters.[456] If there had been a '*well-instrumented, and also a well-scaled model*', Mr Lesleighter's opinion was that '*you might come up with a different answer*'.[457]

What scale would have been appropriate?

6.253 A scale of 1:70 for a physical 3D model might have been better. But even such a larger model can produce scale effects.[458] In relation to the scalability of the discharges expected, Mr Lesleighter stated that in 2003 and 2004 there was no laboratory in Australia that could handle the range of discharges (which were described by witnesses as '*huge*'[459] and '*massive*'[460]) required to be tested in respect of the Dam in anything larger than a 1:100 scale.[461]

6.254 The Alliance was aware that it was dealing with a relatively small scale 3D model. In evidence, Mr Herweynen said:[462]

> *at the time we knew that the scale - and everyone, including Sunwater, knew that the scale of the 3D model was relatively small, not necessarily that it was not able to be used, but that in combination with the 1 to 75 2D model was seen to provide adequate hydraulic for the spillway.*

6.255 To increase the scale, however, the designers were left with two options: use available overseas laboratories[463] or else construct a special purpose-built model in Australia. The latter would have taken '*a good part of a year*'.[464] In today's terms, its cost would be about $1,000,000.00.[465]

6.256 It is not clear that a larger model study would have revealed the flow conditions that led to the problems in 2011 and 2013.

6.257 Mr Wallis said of the 3D model adopted in the Hydraulic Model Study:[466]

> *MR WALLIS: At the time that the tender design and detail design was done, Sunwater had already built a model at the 1 to 100 scale for their preliminary design. It was a very large model. If we had chosen to adopt a larger scale, it*

455 **TRA.500.012.0001**, .0042 ln 35-39. See paragraph 6.241.
456 **TRA.500.012.0001**, .0042 ln 35-39.
457 **TRA.500.012.0001**, .0042 ln 45-47.
458 Exhibit 238, **TRA.510.019.0001**, .0016 and .0040.
459 **TRA.500.012.0001**, .0020 ln 46.
460 **TRA.500.013.0001**, .0081 ln 19.
461 **TRA.500.012.0001**, .0021 ln 4-10. Moreover, in Dr Maleki's opinion, a 1:100 scale model is still '*within the limit of the scale effects*', meaning that that scale is within the upper limit of what is considered permissible for scale effects: **TRA.500.011.0001**, .0037 ln 18-20.
462 **TRA.500.013.0001**, .0076 ln 24-28.
463 **TRA.500.012.0001**, .0023 ln 47.
464 **TRA.500.012.0001**, .0048 ln 35.
465 **TRA.500.012.0001**, .0048 ln 39-40. Mr Lesleighter considered that the design team was justified in choosing the 1:100 scale model at the time: Exhibit 239, **LEE.002.0001**, .0001.
466 **TRA.510.024.0001**, .0003 ln 34 to .0004 ln 4.

> *would have meant dismantling the existing model and rebuilding at a larger scale, which would have meant a delay in the design process. Also, Hydro Tasmania operated a hydraulics laboratory in Hobart since 1958 up until 1992, and 1 to 100 scale had been used on a number of model studies of dams, spillways and intake structures, so I felt that it was appropriate to use that scale. If the model study showed that there were areas of major concern, then we could have suggested that a larger-scale model be built for that particular area that was of concern.*
>
> *MR HORTON: Did any such concerns show up on the model that was used?*
>
> *MR WALLIS: Not that I can remember or have documented.*

6.258 Mr Wallis's evidence is consistent with an understanding that he, as the Alliance's hydraulic expert, would have raised a concern if the 1:100 scale model showed any '*areas of major concern*'.[467] No such concern, at the time of the design, has been identified.

Criticisms

6.259 Mr Lesleighter stated that the design of the apron – principally, the asymmetry – meant that the flow conditions did not perform in a '*two-dimensional manner*'.[468] Rather the apron produced a 'three-dimensional' situation.[469]

6.260 A two-dimensional model study looks at a flow straight over a '*cross-section of the spillway*',[470] or a 'slice',[471] as Figure 6.10 above shows. Mr Lesleighter said:[472]

> *In a two-dimensional, like a flume study, where they have a parallel wall flume, you put the cross-section of the spillway in there and you're just looking at flow going straight over the spillway. You could be almost forgiven for looking at this as a 315 metre wide spillway; therefore, conditions at that end and this end are going to be pretty well the same, and the whole thing is just going to go like that (indicating). But it's not. It's going to go like that, but then it's going to do all this turbulence and circulation downstream. So I've said there that it's not a purely two-dimensional manner like a flume with walls on the side.*

6.261 It was said, therefore, that a 2D model was '*largely pointless*'.[473]

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467 **TRA.510.024.0001**, .0003 ln 34 to .0004 ln 9. Hydro Tasmania submitted that it was reasonable for Mr Herweynen to expect that if particular issues arising from that advice required his attention, that they would be raised by the respective expert, as professional advisors: **HYT.008.0001**, .0060

468 Exhibit 230, **DNR.006.3156**, .3241; Exhibit 238, **TRA.510.019.0001**, .0025 ln 1-18.

469 Exhibit 238, **TRA.510.019.0001**, .0025 ln 1-2.

470 Exhibit 238, **TRA.510.019.0001**, .0025 ln 8-11.

471 **TRA.500.012.0001**, .0025 ln 17.

472 Exhibit 238, **TRA.510.019.0001**, .0025 ln 8-18.

473 **TRA.500.012.0001**, .0025 ln 12-13.

6.262 That was not the view expressed in the 2014 URS Review, although URS criticised *how* the models were used.[474] The problem was, according to URS, the 1:100 scale 3D model was used to '*confirm*', rather than '*design*', the dissipator structure. URS concluded:[475]

> *A specific model study does not appear to have been conducted to design the hydraulic jump basin. Based on the provided information, it appears that only the larger scale (1:100) 3D model was utilised to confirm the basin design, whereas the smaller scale (1:75) 2D sectional model of the spillway could have provided additional insight into the performance and design of the dissipator basin, in particular the length of the hydraulic jump, the ground roller, and velocities along the downstream river bed.*

6.263 In relation to the Alliance's model studies, URS stated that:[476]

- *no velocity measurements within the re-circulation flow patterns were made*
- *the risk of material (gravels and rocks) being drawn into the dissipator was not discussed in the design report or model study report.*

Issues raised in SunWater's earlier 2D model study

6.264 The two dimensional studies undertaken by the Alliance do not appear to have addressed the specific concerns raised in SunWater's 2D model study:

a. A drowned jump formed during higher discharges: rather than achieving satisfactory energy dissipation, it was found that '*the incoming jet plunges to the bottom and carries along the basis floor at high velocity*'.[477]

b. The apron width would need to be 26.7 m for discharges up to 22,444 m^3/s (approximately 1:1,000 AEP) and '*greater than 50 m*' for discharge from 22,444 m^3/s to 72,877 m^3/s.[478]

6.265 Mr Herweynen was given the results of the SunWater 2D model study.[479]

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474 Hydro Tasmania submitted that '*[t]here is no evidence of criticisms raised at the time of the modelling by Mr Wallis during the design and construction of the Dam'.* In its view, the criticisms raised before the Commission '*were made with the benefit of hindsight and in reliance on CFD modelling that was not the practice at the time*': **HYT.008.0001**, .0055 [180].

475 Exhibit 237, **SWA.512.001.0578**, .0636.

476 Exhibit 237, **SWA.512.001.0578**, .0629. Evidence was that the 3D model study did not incorporate transient pressure testing to measure the pressures on the surface of the structure, such as the apron. That technology has been available since the 1970s: **TRA.500.012.0001**, .0022 ln 40 to.0023 ln 7. However, even with pressure transducers in the 1:100 scale model used by the Alliance, that may not have been sufficient to *'quantify the loading on the structure'*: **TRA.500.012.0001**, .0023 ln 13-20. No witness or party suggested that the use of transducers would have identified the problems that led to the 2011 and 2013 apron damage.

477 **SUN.018.027.0454**, .0458.

478 **SUN.018.027.0454**, .0458.

479 **TRA.500.013.0001**, .0070 ln 19-22.

6.266 Mr Griggs, who had responsibility for the '*civil/structural design of the primary spillway apron*',[480] said that he was not made aware of the SunWater 2D model study at the time.[481]

Reaction to SunWater's 2D model study

6.267 Mr Herweynen was asked about the SunWater 2D model study and its suggestion that a longer apron may have been required:[482]

> Q. *Did the last two paragraphs in particular raise in your mind the need, at least at this time, for an apron substantially wider than 20 metres?*
>
> A. *No, it did not, because at that stage we had the opinion, and Golders was supporting that, that the foundation downstream was reasonable.*

6.268 Hydro Tasmania submitted that the SunWater 2D model study was '*superseded by the further hydraulic studies supervised by Mr Wallis*'. Based on that further modelling work, '*Mr Wallis concluded that the 20m apron width proposed in SunWater's preliminary design was satisfactory from a hydraulic performance perspective*'.[483] Hydro Tasmania pointed out that the later studies were '*more advanced assessments*' as opposed to the '*earlier, more preliminary study*'.[484]

6.269 That accords with Mr Dann's view, albeit in relation to URS's approach to the tender. The conclusion that the drowned jump '*plunges to the bottom and carries along the basin floor at high velocity*'[485] was, according to him:[486]

> *… superseded by what we did as part of the tender design. We were looking at the physical hydraulic models in the laboratory, in particular the 2D model, the 1 to 75 scale model.*

Conclusion

6.270 The report of the Hydraulic Model Study by Mr Wallis, the Alliance's experienced hydraulic engineer, concluded that the '*hydraulic jump was confined to the dissipator apron region*'.[487] Mr Herweynen said that the hydraulic model study '*says at the very end, by Mike Wallis, that he believes that the apron hydraulically was okay*;'[488] and, '*[w]e tested [the 20 m apron] in our physical model and in our numerical model and, according to us, it looked to be okay as well*'.[489]

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480 Exhibit 287, **GRT.001.0001**, .0011 [61].
481 **TRA.500.014.0001**, .0097 ln 24-27.
482 **TRA.500.013.0001**, .0071 ln 38-43.
483 **HYT.008.0001**, .0063 [210].
484 **HYT.008.0001**, .0063 [211]. It submitted that '*It was reasonable and appropriate that Mr Herweynen act on the basis of those subsequent and more advanced assessments*'.
485 **SUN.018.027.0454**, .0458.
486 **TRA.500.012.0001**, .0073 ln 32-35.
487 Exhibit 24, **GHD.002.0001**, .0585. The report of Mr Wallis's studies appended to the Detail Design Report was '*approved for issue*' by Mr Neumaier: Exhibit 24, **GHD.002.0001**, .0554.
488 **TRA.500.014.0001**, .0058 ln 36-39.
489 Exhibit 247, **TRA.510.007.0001**, .0098 ln 30-32.

6.271 The Hydraulic Design section of the Detail Design Report, also authored by Mr Wallis, addressed the capacity of the apron to deal with the hydraulic jump in a variety of floods and concluded:[490]

> *A 20m wide spillway apron was proposed in SunWater's Preliminary Design (SunWater, 2003a) and adopted for the final design after the hydraulic model studies showed that it performed satisfactorily.* ***A hydraulic jump occurred on the apron for all flood events tested.***

Asymmetry of the apron

Design

6.272 The apron as constructed sloped down from the left side to the right side: the right side was EL 30.845 m and the left side was EL 37.5 m.[491] The higher level was 45 m long.[492]

6.273 Mr Herweynen explained the background to the slope:[493]

> *That was all modelled in our physical hydraulic model study, that elevation. That elevation … was for a reason. We all know that the floods that we're trying to pass at Paradise Dam are extensive, and therefore the aim was to try to maximise the size of that primary spillway …*
>
> *By maximising the primary spillway, we push it into the left abutment as far as we possibly can, which meant that there was then foundation rock that was higher up at that particular level, and rather than excavate that all the way down so that we were at the same level, there was this concept - and it was in the Sunwater preliminary design, it was in our design, and it was in the URS design - to elevate the apron on that particular side.*
>
> *We modelled that in our physical hydraulic model …*

6.274 Maximising the length of the primary spillway was not the only reason for the slope. The Detail Design Report noted that '*[r]aising the apron [on the left hand side] reduced the excavation volume and gave a cost saving*'.[494]

6.275 The Hydraulic Model Study did investigate different 'invert' levels on both the left *and* right sides of the apron.[495] Both levels were reported to '*[perform] satisfactorily in dissipating energy during spillway operation*'.[496]

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490 Exhibit 24, **GHD.002.0001**, .0066 (emphasis added). As discussed below, Mr Lesleighter described the finding that the '*apron contained the hydraulic jump*' as 'fallacious': **TRA.500.012.0001**, .0046 ln 7-16.

491 Exhibit 24, **GHD.002.0001**, .0060.

492 Exhibit 24, **GHD.002.0001**, .0136. An image showing this 'asymmetry' appears at Figure 6.1.

493 **TRA.500.013.0001**, .0081 ln 11-41.

494 Exhibit 24, **GHD.002.0001**, .0568.

495 Exhibit 24, **GHD.002.0001**, .0562.

496 Exhibit 24, **GHD.002.0001**, .0568.

Damage to the apron

6.276 The asymmetry significantly contributed to the damage sustained by the apron and sill.[497] The asymmetry introduced a 'ball-mill' effect.[498] This effect is characterised by violent turbulence which retains or entrains rock into the stilling basin (i.e. the '*circulation condition … can pick up rock and dump it in the basin*'[499]) and allows it to abrade and wear down the apron.[500]

6.277 During the 2011 event, gravel and rock materials were drawn into the apron causing significant abrasion and damage to the apron and end sill.[501]

6.278 A SunWater report, *Paradise Dam Flood 2010/11 Damage Inspection and Civil Works Rectification*, explained:[502]

> *The dissipator that has a higher apron and sill level on the left hand end appears to have resulted in turbulent three dimensional roller action at the end sill and downstream of the dissipator apron.* ***The abrasion of the end sill and apron slab is most likely caused by rock drawn into the basin and then tumbled around in the roller flow.*** *The varying degrees of abrasion of the concrete in the apron and end sill would be attributed to different characteristics and intensities of the roller action across the basin.*

6.279 Mr Dann agreed that damage was caused by 'ball milling'.[503]

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497 Mr Lesleighter explained in the report that the scour at the apron edge following the 2013 event suggested both 'plunging flow' and 'strong eddy action'. There are two possible causes of this: the contraction from the width of the spillway (315 m) back to the normal width of the river channel; and the '*inevitable effect of flow which enters the apron which has a height difference of 7 m from the right half of the apron to the left end of the spillway apron*': Exhibit 7, **IGE.017.0001**, .0060.

498 **TRA.500.012.0001**, .0017 ln 25.

499 Exhibit 238, **TRA.510.019.0001**, .0021 ln 43-44.

500 **TRA.500.012.0001**, .0017 ln 19-28.

501 Exhibit 237, **SWA.512.001.0578**, .0660.

502 Exhibit 230, **DNR.006.3156**, .3178 (emphasis added). The basis for these conclusions, which relate to hydraulic matters, appears to be a report of Mr Lesleighter dated 16 November 2012 which is appended to the main report: **DNR.006.3156**, .3241-2.

503 **TRA.500.012.0001**, .0075 ln 1-6: '*It's called ball milling. When you have turbulent conditions in the dissipator and the rocks are being drawn into that structure, those rocks are pounded against the floor and the face of the end sill, and that damage was reported in 2010 quite clearly*.' Mr Lesleighter considered that the asymmetry was one of the contributing causes to the scour in 2013, adding '*but the apron obviously did not achieve anything like the degree of energy dissipation that we needed to do for the geology that we had*': **TRA.500.012.0001**, .0029 ln 40-44.

20 m width of apron

6.280 In evaluating the causes of the stability issues in respect of the apron, a question is whether it should have extended farther downstream.

Unchanged from SunWater's Preliminary Design

6.281 The apron width of 20 m was not changed from the proposal in SunWater's Preliminary Design.[504] That design, however, expressed qualifications.[505] Nevertheless, when it was suggested to Mr Herweynen that the apron was of insufficient width, he asserted that SunWater's Preliminary Design had '*determined*' that 20 m was reasonable:[506]

> *… the geology did not change between the preliminary design at that location and our design. Most of the geotechnical investigation, the drill holes that were done were already done during the Sunwater preliminary design. The geology was known, the tailwater was known at that point, and it's all the same. Based on, you could say, the rules of thumb that are used within the industry, these charts,* ***Sunwater determined 20 metres was reasonable for the preliminary design. We tested that in our physical model and in our numerical model and, according to us, it looked to be okay as well*** *…*
>
> *… maybe longer might have been better, or maybe a higher end wall. I don't know what the solution is, but, for me, the way that … we did it, was consistent with what Sunwater had concluded and it was consistent with our testing, both physical modelling and numerical modelling.*

6.282 However, Mr Herweynen accepts that the tenderers '*had to do their own investigation in relation to verifying what the proper sill and proper apron was*'.[507]

6.283 The URS proposal documents were provided to the Alliance after the Hydro Tasmania consortium had been selected as the successful tenderer.[508] Mr Griggs saw that the Alliance's apron '*was the same width as adopted by Sunwater in their preliminary design and Thiess/URS in their tender design*'.[509] Similarly, Mr Herweynen stated that:[510]

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504 Exhibit 24, **GHD.002.0001**, .0136.

505 The qualifications, described at paragraph 6.170, included that it was for '*information only*' and '*Burnett Water [was] not committed to the preliminary design*'. Burnett Water did not warrant the '*completeness, accuracy or adequacy*' of the information. Exhibit 250, **SWA.500.001.2366**, .2378 and .2379

506 Exhibit 247, **TRA.510.007.0001**, .0098 ln 23-41 (emphasis added).

507 **TRA.500.014.0001**, .0052 ln 46 to .0053 ln 2.

508 **TRA.500.013.0001,** .0030 ln 41-43: Mr Herweynen stated: '*We were given the URS submission, and we were comparing some of the things that they had proposed to what we had proposed*.' and .0069 ln 43-47: '*after stage 2, we then obtained that as well, and, yes, they had 20 metres at that point as well*'.

509 Exhibit 287, **GRT.001.0001**, .0011 [62(a)].

510 Exhibit 244, **HER.001.0001**, .0015 [58].

> *Part of the work we did in the Detail Design Phase included considering any points of difference between our design and the Thiess / URS design to see if there were any further optimisations or issues raised that we incorporated into our design process, with actions to address these. This submission had a number of similarities to our design, including an all RCC construction, the secondary spillway on the right abutment, a 20m long apron, and the apron rising on the left of the primary spillway.*

6.284 The URS design, however, differed from the Alliance's.[511] The differences included the alignment of the primary spillway, a reinforced concrete apron with a ramped end sill 1.8m high, and a different crest profile on the primary spillway. URS's proposal was subject to further modelling including a 2D model to be commissioned and built in the United States of America.[512]

Hydraulic jump

6.285 The Hydraulic Model Study had suggested that an '*hydraulic jump occurred on the apron for all flood events tested.*'[513] It was said to be '*confined to the dissipator apron region*'.[514] If by that the author meant that the hydraulic jump was no longer than 20 m, that impression was mistaken. Mr Lesleighter was emphatic:[515]

> *… There's no way in the world that [the apron] contained the hydraulic jump. The hydraulic jump was going to be a lot longer than 20 metres …*

He added:[516]

> *… that conclusion … is fallacious.*

6.286 Consistently with that view, in a 2013 report of the first TRP, Mr Lesleighter wrote that:[517]

> ***the hydraulic jump as properly defined and understood is not contained within the width of the dissipator apron*** *… the spillway apron was far shorter than what normal practice for a hydraulic jump stilling basin would dictate.*

6.287 In testifying, Mr Neumaier accepted that the hydraulic jump is not contained within the width of the dissipator apron:[518]

> *Q. At the time you designed the dam, I understand you are saying you weren't aware that the hydraulic jump would not be entirely contained within a 20 metre apron?*

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511 Exhibit 241, **DAC.001.0001**, .0008-9 [23].
512 Exhibit 81, **DNR.007.1087**, .1369.
513 Exhibit 24, **GHD.002.0001**, .0066.
514 Exhibit 24, **GHD.002.0001**, .0585.
515 **TRA.500.012.0001**, .0046 ln 16.
516 **TRA.500.012.0001**, .0046 ln 7-16. Mr Lesleighter's view was that '*a simple desk study*' would have shown a need for an apron length in the order of 60 m: **TRA.500.012.0001**, .0045 ln 5.
517 Exhibit 7, **IGE.017.0001**, .0048 (emphasis added).
518 **TRA.500.015.0001**, .0017 ln 8-15 (emphasis added).

> A. *Not - you could draw that conclusion from the hydraulic report,* ***from the model study, which says that the hydraulic jump is contained within the apron, yes, and that obviously wasn't the case****, and the calculations show it is not.*

6.288 Mr Neumaier also acknowledged that '*calculations*' would predict an hydraulic jump greater than 20 m, at least in '*some cases*'.[519]

6.289 The 2014 URS Review, in its 'Key Conclusions', stated:[520]

> *…the design of the dissipator would not meet the USBR guidelines for the hydraulic design of a Type II dissipator structure, in particular:*
>
> a. ***The length of the dissipator apron is relatively short compared to the length required by the USBR guidelines****. While engineering judgement is required to select a suitable dissipator length given the expected foundation conditions downstream of the dam,* ***precedent on other projects suggests that as a minimum the dissipator should be designed to contain the hydraulic jump from a 1 in 100 year AEP event, which would require a dissipator length of the order of 50m.*** *Given that the constructed length of the dissipator apron was 20m and that the hydraulic jump would extend beyond the dissipator structure, there is a high level of reliance in the design that the foundation downstream of the dissipator is able to withstand the hydraulic forces of the hydraulic jump downstream of the dissipator structure.*
>
> b. *Engineering judgement is required to select a suitable end sill height. However the 1m high end sill provided is less than recommended by the USBR guidelines, by at least 50% depending upon the event selected for design.*

Subsequent estimates of possible appropriate width

6.290 GHD assessed the required length of a stilling basin based on the USBR Monograph:[521]

> *A high level assessment based on the methodology given in USBR (1987) was undertaken which indicated that a stilling basin length of the order of 60-70 m. It is noted that Paradise Dam is not a text book application of this design method,*

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519 **TRA.500.015.0001**, .0002 ln 28-35: Mr Neumaier said, '*That was in relation to the width of the apron, which is 20 metres, and I said at the time that the hydraulic jump would be contained within those 20 metres, which I know is not the case. I've done some calculations since, and it would be misleading to say it's contained. The tailwater is always higher than what is required to contain the hydraulic jump, but the hydraulic jump itself would be extending beyond the 20 metres in some cases'*.

520 Exhibit 237, **SWA.512.001.0578**, .0586-7 (emphasis added). The reference to the 'USBR guidelines' was to the USBR Monograph: **TRA.500.012.0001**, .0072 ln 9-15.

521 Exhibit 234, **IGE.033.0001**, .0017-18. The report is dated March 2018 and was part of the new rectification works intended for the Dam.

particularly given the extremely high unit discharge relative to the recommended range.

6.291 In comparison, URS analysed the likely apron widths that the USBR Monograph indicated using the discharges experienced in the 2011 and 2013 events. As the table extracted from the report shows, the required basin width (referred to as 'basin length') ranged from 45 to 76 m:[522]

Event	Reservoir Head (m)	Spillway Discharge (m^3/s)	Dissipator slab at EL 37.5 m		Dissipator slab at EL 30.48 m	
			Basin length (m)	End sill height (m)	Basin length (m)	End sill height (m)
2010	5.96	8,770	45	2.1	47	2.2
2013	8.65	17,000	59	2.9	62	3.1
1:1,000	11.5	25,000	73	3.7	76	3.9

*Figure 6.14 – Table from 2014 URS Review showing estimated apron widths calculated with the USBR Monograph. (Exhibit 237, **SWA.512.001.0578**, .0620)*

6.292 GHD's CFD analysis determined that '*pressure fluctuations trend towards zero at a location approximately 80 m to 100 m downstream of the spillway crest*'.[523] Dr Maleki explained that at this location, the flow of the water would likely not result in major erosion of weak rock.[524]

Scouring

6.293 The evidence reveals that the 20 m apron did not contain the hydraulic jump. Rock material downstream of the primary spillway apron was subjected to energy that had not been dissipated. Scouring resulted.

6.294 At 20 m, therefore, the apron was not wide enough to contain the erosive forces on the rock immediately downstream. The apron, according to Mr Lesleighter, did not '*achieve … energy dissipation to a sufficient degree*'.[525]

6.295 As Mr Young said, the '*simple solution*' was to have made the apron much wider: '*so that the water would have slowed down a bit by the time it hit rock*'.[526]

6.296 URS's 2014 Review stated:[527]

522 Exhibit 237, **SWA.512.001.0578**, .0620. These estimates are not provided to show what an adequate width for the apron would be. That would require detailed modelling.

523 Exhibit 228, **GHD.041.0001**, .0059.

524 **TRA.500.011.0001**, .0030 ln 8-12.

525 **TRA.500.012.0001**, .0014 ln 2-25. He also said, that the apron '*obviously did not achieve anything like the degree of energy dissipation that we needed to do for the geology that we had*': **TRA.500.012.0001**, .0029 ln 40-47.

526 Exhibit 76, **YOJ.001.001.0001**, .0005 [16].

527 Exhibit 237, **SWA.512.001.0578**, .0586-7.

> *Our independent analysis shows high hydraulic energy conditions within a zone at least 50m downstream of the toe of the dam (potentially up to 70m) and that erosion of weathered materials, shear zones and foundations with discontinuities within these high energy zones was to be expected, even if the end sill structure was intact.*

6.297 The insufficient width exposed the riverbed immediately downstream of the apron to greater erosive forces than the rock could withstand at the locations at which scour and erosion occurred.

To what extent did the geotechnical studies influence the design of the apron?

6.298 Would the designers have left the primary spillway apron's width at 20 m if they had understood Golder's advice?

6.299 Mr Herweynen indicated that the perceived resistance of the riverbed to erosion *was* a factor in considering concerns raised by a 2D hydraulic model study of SunWater which indicated a longer apron may have been necessary.[528] However, he stated:[529]

> *Based on [the Alliance's] modelling work, I recall that Mr Wallis concluded that the 20m apron width proposed in SunWater's preliminary design was satisfactory from a hydraulic performance perspective. This length of apron, along with a 1m end wall, was demonstrated by the hydraulic modelling to contain the hydraulic jump within the apron area, for all modelled flood events.*

6.300 Mr Neumaier said that:[530]

> *… the model tests showed that the 20 metre wide spillway with a 1 metre high end sill contained the hydraulic jump, and this is the one that's causing, potentially, erosion. It was contained within those 20 metres.*

6.301 The conclusions from the Hydraulic Model Study were interpreted as showing that the hydraulic jump was contained in the 20 m apron for all flood events modelled. Mr Herweynen and Mr Neumaier were confident that the hydraulic jump was contained within the apron.[531]

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528 **TRA.500.013.0001**, .0071 ln 38-43.

529 Exhibit 244, **HER.001.0001**, .0023 [98].

530 Exhibit 302, **TRA.510.021.0001**, .0030 ln 35-41. As noted at paragraphs 6.287-6.288, Mr Neumaier now accepts that this was not correct.

531 It is speculation what any erodibility assessment would have concluded, especially given the presence of the cofferdam which may have precluded that exercise. Dr Annandale undertook an erodibility assessment using the geological characteristics recorded during Golder's original mapping of the area underneath the apron as part of the 2014 URS Review commissioned by SunWater's insurers. It referred to Dr Annandale's Erodibility Index Method (Annandale 1995, 2006). The report suggested that, if Golder had carried out the erosion assessment using the Annandale method, the potential for scour could have been identified and a conclusion reached that the rock downstream of the primary spillway would have been susceptible to scour: Exhibit 237, **SWA.512.001.0578**, .0780 and .0790.

6.302 Mr Herweynen and Mr Neumaier did not understand the meaning or significance of Golder's advice.[532] But if they had, it looks to be likely that their confidence in Mr Wallis's assessment would nonetheless have persuaded them to adopt 20 m as the apron width.

Conclusion: geotechnical issues and apron width

6.303 The width of the apron was insufficient. It did not contain the hydraulic jump. It did not protect the area immediately downstream of the apron from erosive forces. The area where the scouring occurred in 2013 could not withstand those forces.

6.304 A root cause of the 2013 scouring immediately downstream of the apron was its insufficient width.

End sill

Background

6.305 The Paradise Dam Safety Review Report of 30 October 2014 described the end sill as follows:[533]

> *No specific record was found for the design of the apron end sill. The drawings show a reinforced concrete section 1.0m wide and 1.0m high above the apron level. The sill was connected laterally to the RCC apron by N16 bars at 350mm spacing, and vertically into foundation a minimum of 2m by N24 centrally placed galvanised dowels at 2,800mm spacing.*

6.306 The 2013 event caused almost complete removal of the end sill from the apron (apart from a 7 m length[534]). Large sections of the sill, estimated to weigh 20 t,[535] were carried up to 100 m downstream of the spillway.[536] The image below shows one such section which has been pushed downstream by the force of water.

532 See paragraphs 6.157-6.160.
533 **DNR.001.5574**, .5673.
534 **DNR.001.0036**, .0090.
535 Exhibit 7, **IGE.017.0001**, .0060.
536 Exhibit 7, **IGE.017.0001**, .0060.

*Figure 6.15 – Piece of end sill downstream of the Dam following the 2013 flood. The annotations were made by URS. (Exhibit 237, **SWA.512.001.0578**, .0609)*

Investigation into failure of end sill

6.307 The failure of the end sill during the January 2013 flood was investigated by URS in its 2014 review. URS reported these '*key findings*':[537]

> 1. *Some horizontal reinforcing bars from the dissipator apron slab to the end sill structure had failed. There was evidence of necking of the bars. (Refer Plate 6.3* [reproduced below as Figure 6.16] *and Plate 6.4)*
>
> 2. *Some horizontal reinforcing bars from the dissipator apron slab to the end sill structure had either pulled out of the concrete or the concrete had eroded to a point that the bars were fully exposed. (Refer Plate 6.3)*
>
> 3. *There was evidence that these horizontal reinforcing bars had been installed as 'L' bars within the end sill structure that had subsequently been exposed by locally breaking out the end sill concrete and bending the bars into the apron slab area. This is a poor construction practise as this technique has the potential to weaken the reinforcing bars. It is unknown how the bars were bent onsite. (Refer Plate 6.4)*
>
> 4. *There was evidence that the joint between the end sill conventional concrete and the RCC apron was not treated as construction joint. (Refer Plate 6.4)*

537 Exhibit 237, **SWA.512.001.0578**, .0642-3.

5. *There was evidence that the foundation anchor bars that extend from the foundation into the end sill structure failed. (Refer Plate 6.5)*

6. *There was some corrosion evident on the bars, it is unclear whether this occurred prior to or after the end sill failure or whether it had any impact on the end sill failure. (Refer Plate 6.3)*

7. *The end sill itself had not structurally failed and was generally in intact blocks. (Refer Plate 6.3)*

6.308 'Plate 6.3', referred to in URS's key findings is reproduced below as Figure 6.12.

*Figure 6.16 – An end sill block downstream of the Dam's primary spillway following the 2013 flood. N16-350 horizontal reinforcing bars are visible. (Exhibit 237, **SWA.512.001.0578**, .0643)*

6.309 The 2014 URS Review analysed the capacity of the end sill to withstand the hydraulic forces to be expected during spillway flows.[538]

538 Exhibit 237, **SWA.512.001.0578**, .0645. See below from paragraph 6.314.

6.310 The URS Review relied on CFD:[539]

> *A series of CFD models were developed of the Paradise Dam primary spillway to*
>
> - *Simulate the hydraulic performance of the 'as-designed' spillway dissipator basin and flows downstream of the dissipator basin at the maximum discharge for select flood events: 2010 flood, 2013 flood, and the 1:1,000 AEP design flood*
> - *Assess the hydraulic forces as input to:*
> - *An assessment of the potential for erosion of the unlined river areas downstream of the end sill*
> - *Estimate the potential hydraulic forces acting on the dissipation slab and end sill*
> - *Compare the 'as-designed' model results to an end sill failure scenario for the 2013 flood event case to assess the changes in hydraulic performance and flow patterns.*

6.311 Mr Dann explained that '*the CFD modelling allowed us to get a load to apply to the end sill. It would be difficult to do that from the physical model*'.[540]

Analysis

6.312 The end sill was assessed for both sliding stability and overturning stability:[541]

> *The overturning stability was evaluated based on both the overturning Factor of Safety and the location of the resultant force, which is shown as a percentage of the base length measured from point of rotation. The strength was assessed in accordance with the requirements of AS3600 – Concrete Structures.*

6.313 The material properties adopted in the assessment were based on those set out in the Detail Design Report. The stability and strength of the end sill were assessed for the following discharge events:[542]

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539 Exhibit 237, **SWA.512.001.0578**, .0738. The criticisms made by URS, submited Hydro Tasmania, were '*made with hindsight and with the benefit of technical modelling of a standard and sophistication that was beyond the capacity of analysis available at the time*'. Hydro Tasmania also submitted that the apron, including its anchoring and reinforcing, '*was designed based on the hydraulic modelling of the forces it was likely to withstand at the time*': **HYT.008.0001**, .0073 [253]. Mr Herweynen said: '*our loads were based on our physical hydraulic model study; their loads were based on the CFD model for that. If those loads are significantly different, well, then, of course that changes the structural capacity calculation*': **TRA.500.013.0001**, .0080 ln 37 .0081 ln 1.

540 **TRA.500.012.0001**, .0096 ln 37-39.

541 Exhibit 237, **SWA.512.001.0578**, .0772. A memorandum containing the 'Assessment of Failure of Paradise Dam Spillway Dissipator End Sill' is Appendix C to the 2014 URS Review. It is dated 3 June 2014, from Reza Darabi addressed to Michael Phillips: Exhibit 237, **SWA.512.001.0578**, .0771.

- *The 2010/2011 event with 8,770 m^3/s (1:40 AEP) discharge, 5.96 m overtopping the spillway;*
- *The 2013 event with 17,000 m^3/s (1:170 AEP) discharge, 8.65m overtopping the spillway; and*
- *The 1:1000 AEP event, 10.4m overtopping the spillway.*

6.314 The '*original design intent of the connection of the end sill to the apron slab*' was said in the URS analysis to be '*unknown*' so two 'cases' were assessed:[543]

- *Case 1: End sill acts as an independent structure (i.e. connection to the apron slab is ignored); and*
- *Case 2: End sill acts in combination with apron slab (i.e. the tensile capacity of the connecting dowel bar is included in the analysis)*

The end sill was assessed at a section near the left abutment where the apron slab elevation is EL37.5m and at a section towards the centre of the spillway where the apron slab elevation is EL30.845m. The reason for assessing two locations is that the hydrodynamic loads differ at each location.

Apart from the stability analysis, an additional assessment was conducted on the structural capacity of the apron slab for the theoretically applied moments from the end sill structure.

6.315 The loads applied to the end sill were investigated. These included:[544]

a. the weight of the sill

b. the hydrodynamic forces due to spillway flow

c. the resisting strength of the N24 vertical dowel bars

d. the resisting strength of the N16 horizontal reinforcing bars (Case 2 only).

542 Exhibit 237, **SWA.512.001.0578**, .0773.
543 Exhibit 237, **SWA.512.001.0578**, .0773.
544 Exhibit 237, **SWA.512.001.0578**, .0645-6.

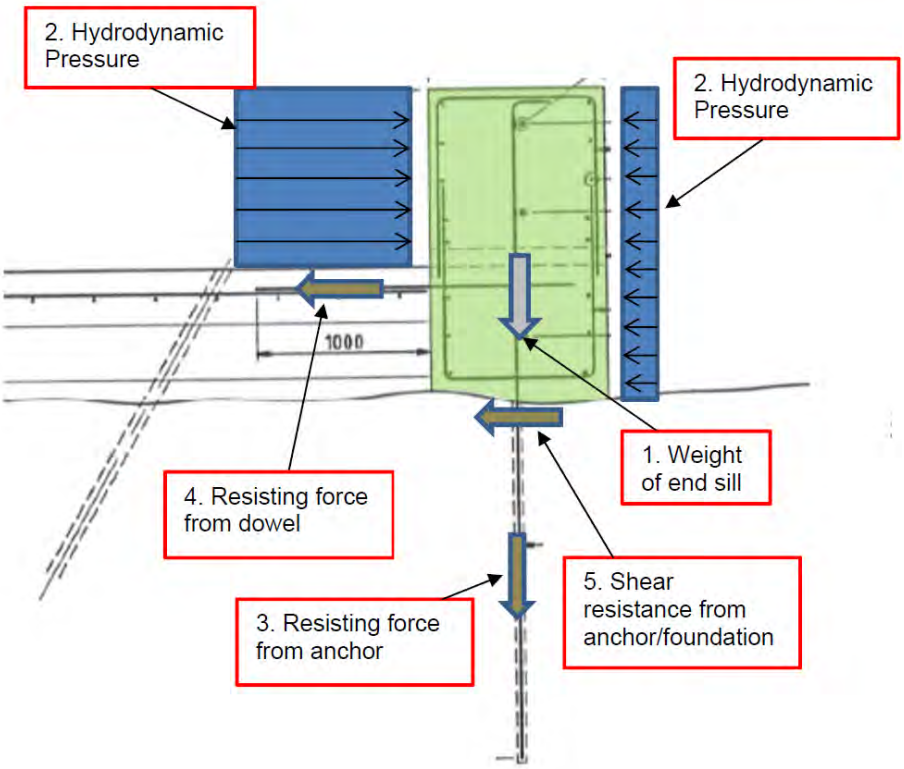

*Figure 6.17 – Loading diagram for the end sill from the 2014 URS Review. The green shaded area is the end sill; the water is shown in blue. (Exhibit 237, **SWA.512.001.0578**, .0774)*

Stability results

6.316 URS concluded that '*overturning stability*' was the '*dominant failure mode rather than sliding*'.[545] The key findings were:[546]

Case 1:

- *The end sill was found to be unstable for overturning as an independent structure for the assessed spillway discharge loading conditions. The Factor of Safety for overturning was found to be <0.35 for the hydraulic loading conditions and the position of resultant was outside of the base. Therefore it is concluded that the end sill stability is reliant on connection to the apron slab.*

Case 2:

- *The Factors of Safety for overturning for the December 2010 spillway discharge were 1.3 at the left abutment, which could be considered acceptable, but less than 1.0 at the centre of the spillway, which indicates that the end sill in this area could have failed during the December 2010*

545 Exhibit 237, **SWA.512.001.0578**, .0775.
546 Exhibit 237, **SWA.512.001.0578**, .0774-5.

event. The fact that the dissipator end sill did not fail during the December 2010 spillway discharge suggests that the as constructed end sill structure may have greater structural capacity than theoretically calculated. It is noted that some vertical and horizontal deformation between the end sill units was reported in the 2010 Five Yearly Comprehensive Dam Safety Inspection … undertaken prior to the December 2010 spillway discharge event. The observed deformation could indicate that the dowels had already yielded.

- *The overturning safety factors for the 2013 and 1 in 1,000 spillway discharge events were all less than 1.0 and the position of resultant was outside of the base. This shows that failure of the dissipator end sill structure was to be expected during the 2013 spillway discharge and the larger 1 in 1,000 AEP event.*

Strength results

6.317 An additional assessment was conducted on the '*structural capacity*' of the apron slab:[547]

> *It was assumed that the connection acted as a fixed connection and that moments were directly applied from the end sill to the apron slab. The analysis was conducted in accordance with the ultimate limit state design requirements of AS3600 and to the yield capacity of the steel to assess whether failure was considered likely.*

6.318 In terms of the 'structural connection' between the apron slab and the end sill, URS stated that the '*dowels between the apron slab and end sill are N16's at 350mm spacings which is less reinforcement than the apron slab*'.[548] This suggested to URS that it was not designed as a structural connection.[549]

6.319 Mr Dann was asked about the connection of the end sill to the apron. He said the end sill '*was not inherently connected to the apron*': it '*was a reinforcing bar into the top mat, a single bar, regularly spaced along the reinforcing*'.[550]

6.320 The key findings reported by URS were:[551]

> 1. *For the loading scenarios assessed the end sill and apron slab connection did not meet the design requirements of AS3600 for ultimate limit state design.*
>
> 2. *For the December 2010 spillway discharge:*

547 Exhibit 237, **SWA.512.001.0578**, .0777.
548 Exhibit 237, **SWA.512.001.0578**, .0640.
549 Exhibit 237, **SWA.512.001.0578**, .0640.
550 **TRA.500.012.0001**, .0079 ln 36-45.
551 Exhibit 237, **SWA.512.001.0578**, .0777-8. The 2016 Dam Safety Review reported that the anchor bars were '*clearly inadequate*': Exhibit 42, **DNR.002.3132**, .3244.

a) *the applied moments generated by the hydrodynamic loads were less than the theoretical capacity of the end sill and apron slab connection at the left abutment.*

b) *The applied moments at the centre of the end sill exceeded the theoretic capacity of the end sill and apron slab connection, which indicates that the end sill in this area could have failed during the December 2010 event. The fact that the dissipator end sill did not fail during the December 2010 spillway discharge suggests that the as constructed end sill structure may have greater structural capacity than theoretically calculated.*

3. *For the 2013 and 1 in 1,000 spillway discharge events, the applied moments to the end sill structure generated by the hydrodynamic loads exceed the theoretical capacity of the end sill and apron slab connection. This shows that failure of the dissipator end sill structure was to be expected during the 2013 spillway discharge and the larger 1 in 1,000 AEP event.*

Other possible contributing factors to the end sill's failure

6.321 Might the failure of the end sill in 2013 have been related to damage in the 2011 event?

6.322 Hydro Tasmania submitted that '*the failure during the 2013 flood may have been due to this damage, rather than structural inadequacy*'.[552]

6.323 Mr Herweynen stated that the '*damage to the end sill and damage to the slab near the end sill [in the 2011 event] … may have impacted its structural integrity for the 2013 flood*'.[553] He said that '*there was significant damage at the 2011 flood. That's evident by the photos. The structural capacity at that point has been already impacted …*'[554] Mr Neumaier said that '*the damage of 2011 may … have [been] a contributing factor to the failure in 2013*'.[555] Mr Herweynen and Mr Neumaier emphasised the '*very long duration*'[556] of the spillway flow.[557] Mr Neumaier described it as exceptional.[558]

552 **HYT.008.0001**, .0074 [255].
553 Exhibit 245, **HER.002.0001**, .0002 [7].
554 **TRA.500.013.0001**, .0080 ln 45-47.
555 **TRA.500.015.0001**, .0019 ln 25-27.
556 Exhibit 245, **HER.002.0001**, .0002 [6].
557 See Exhibit 245, **HER.002.0001**, .0003 [12].
558 **TRA.500.015.0001**, .0019 ln 5-11. The Dam spilled almost continuously for around 21 months in the period 2010-2012.

6.324 The 2014 URS Review, however, stated:[559]

> *The erosion damage to the end sill structure and the connection to the apron slab that occurred during the 2010 spillway discharge event … would have weakened the end sill structure and potentially contributed to the failure of the end sill during the 2013 spillway discharge.* ***However the structural assessment shows that failure of the end sill would have occurred even if there was no damage to the end sill structure.***

6.325 As Mr Dann explained, that conclusion was referable to the '*way in which [the end sill] was connected to the remainder of the apron*'.[560]

6.326 Testifying, Mr Neumaier accepted that URS's conclusion had '*some value*, *is valid*'.[561]

6.327 Despite its conclusions, URS acknowledged that the '*as constructed end sill structure may have greater structural capacity than theoretically calculated*'.[562] Moreover, URS's conclusions were made without the benefit of construction records.[563] So there remains uncertainty about the extent to which the design of the end sill contributed to its failure during the 2013 event.[564]

Opinions on whether the end sill's failure contributed to the 2013 scour

6.328 In general, an end sill assists in the formation of the hydraulic jump.[565] Sills may also be designed to act as a '*deflector of the flow and partly as an inhibitor to the flow*'.[566]

6.329 The 2014 URS Review concluded:[567]

> *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.*

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559 Exhibit 237, **SWA.512.001.0578**, .0648 (emphasis added).

560 **TRA.500.012.0001**, .0084 ln 22-31.

561 **TRA.500.015.0001**, .0020 ln 24-30.

562 Exhibit 237, **SWA.512.001.0578**, .0778. See paragraph 6.320.

563 **TRA.500.012.0001**, .0077 ln 30-31 and .0084 ln 9-20; Exhibit 237, **SWA.512.001.0578**, .0590. Hydro Tasmania submitted that '*causative connection between the 2010/11 flood and the damage sustained in 2013 is unclear and should not be dismissed without further investigation*': **HYT.008.0001**, .0160 [586].

564 Dr Maleki, for example, thought that the formation of the scour itself could have contributed to the loss of the end sill: **TRA.500.011.0001**, .0040 ln 41-45. The 2014 URS Review was aided by the use of CFD: something that was not as sophisticated at the time of the Dam's design as it is now.

565 **TRA.500.006.0001**, .0014 ln 2-3; Exhibit 238, **TRA.510.019.0001**, .0033 ln 4-25; **TRA.500.011.0001**, .0030 ln 43-45.

566 **TRA.500.012.0001**, .0034 ln 46-47. See also **TRA.500.011.0001**, .0032 ln 17-28.

567 Exhibit 237, **SWA.512.001.0578**, .0664.

6.330 This condition was said to be '*primarily a result of the jet in the dissipation basin diving towards the bed, instead of rising up into flow towards the surface*'.[568] The relevant CFD model images are shown in Figure 6.18 below.

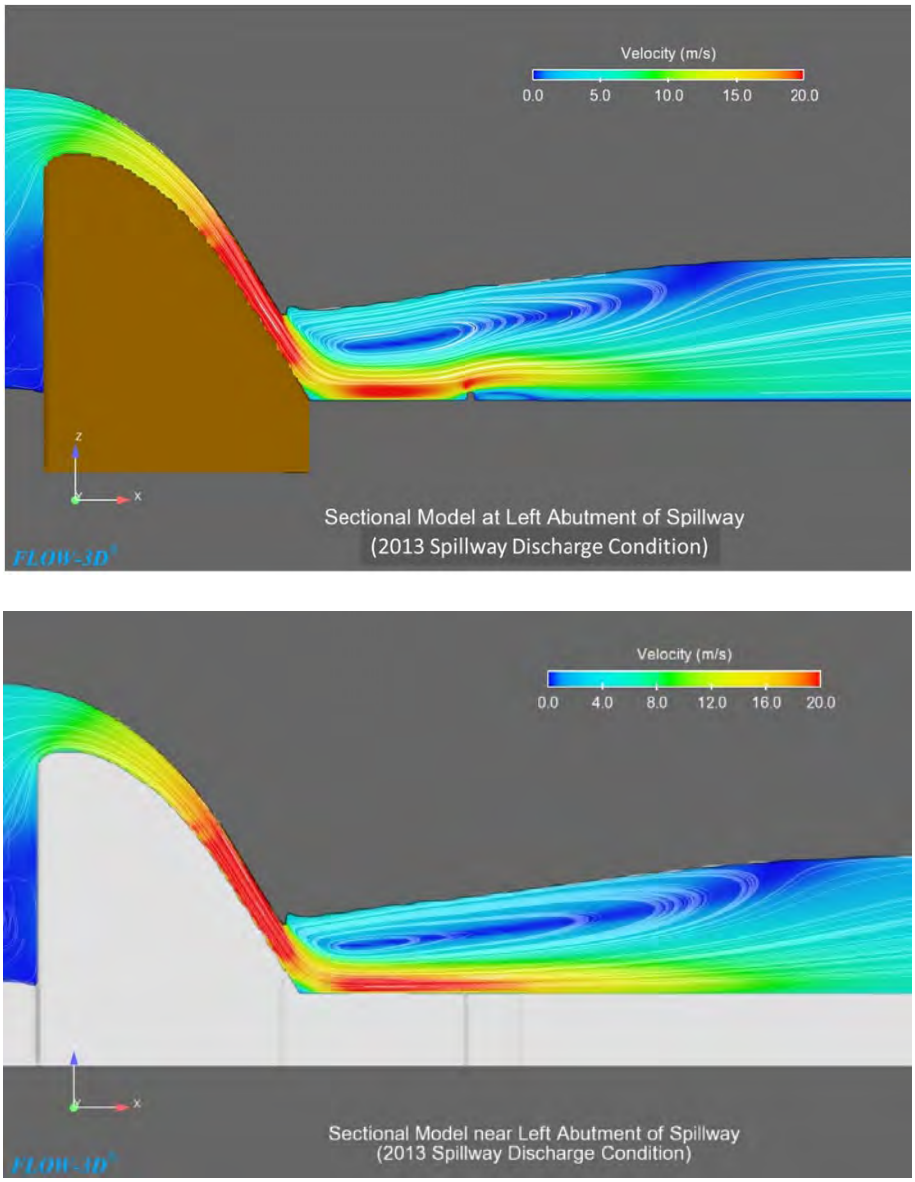

*Figure 6.18 – URS CFD modelling for the 2013 event. The top image shows an 'as-designed' scenario (i.e. with an end sill) and the bottom image shows the situation without an end sill. (Exhibit 237, **SWA.512.001.0578**, .0634)*

568 Exhibit 237, **SWA.512.001.0578**, .0634-5.

6.331 Mr Dann said that the loss of the end sill contributed to the scour:[569]

> Q. *Do you have a view about whether the loss of that end sill caused or contributed to the scour that was experienced immediately downstream of the primary spillway apron in 2013?*
>
> A. *Yes, absolutely. I think in this [2014 URS Review] report you will see modelling that discusses velocities on the riverbed downstream of the structure. When we modelled the end sill in place, the 1 metre one, you still get some lift of the jet off the riverbed, but* ***once that dissipator sill or the end sill is gone, then the jet is following directly on the riverbed****. I think we were talking of velocities* ***up to 14 metres a second on the riverbed with no end sill, compared to, say, 5 or 6 metres a second if it was in place, so that's quite a dramatic change.***

6.332 Using the CFD hydraulic modelling, the 2014 URS Review compares the energy that would impact the riverbed with and without the end sill, expressed in terms of 'stream power' (i.e. an erodibility assessment).[570] Stream power is '*a variable used to quantify the ability of flowing water to scour earth materials*'.[571] The conclusion was that the stream power increased by about 50% without the end sill in place.[572] That stream power must then be compared to the erodibility of the rock measured using the Erodibility Index Method.[573]

6.333 The finding by URS – on the basis of an erodibility assessment conducted by Dr Annandale – was that the end sill affected the *extent* of the erosion:[574]

- *Erosion of the weathered and fractured zones within the foundation would be expected during the 2013 spillway discharge event given the computed stream power from the event and the limited capacity for the weathered and fractured zones to resist erosion, regardless of having a functioning end sill structure as designed or with failure of the end sill structure.*
- *The* ***extent of expected erosion downstream of the spillway would be more extensive without an end sill structure*** *given that the foundation would see greater stream power due to the lack of energy dissipation without the end sill in place.*

569 **TRA.500.012.0001**, .0074 ln 23-36 (emphasis added).
570 Exhibit 237, **SWA.512.001.0578**, .0654.
571 Exhibit 237, **SWA.512.001.0578**, .0787.
572 Exhibit 237, **SWA.512.001.0578**, .0636.
573 Exhibit 237, **SWA.512.001.0578**, .0781.
574 Exhibit 237, **SWA.512.001.0578**, .0658 (emphasis added). The Bollaert method applied in relation to the Dam in 2016 was modelled only for an apron without an end sill: **DNR.002.0001**, .1079.

6.334 Dr Annandale, in his report, put it this way:[575]

> *The failure of the end sill of the spillway further exacerbated the situation. Its failure resulted in significantly higher stream power downstream of the energy dissipater, and therefore significantly greater scour. Regardless of whether the end sill failed, or not, scour would have occurred downstream of the spillway in 2013.* ***The failure of the end sill increased the scour extent.***

6.335 Another view was expressed by the first TRP (of which Mr Lesleighter was part). The report of the TRP, dated October 2013, considered the proposal at the time to reinstate a dissipator end sill.[576] Mr Lesleighter said that this proposal '*could be a retrograde step, because there is no way at this stage of saying that the extent of the erosion that has occurred might not have been exacerbated by the end sill*'.[577] The report added:[578]

> *… perhaps the end sill caused the damage as described rather than be a means to counteract it.* ***We do not know when the end sill actually failed, and therefore what part the end sill played in restricting or exacerbating damage.***

6.336 Mr Lesleighter explained why the end sill itself may have in fact exacerbated the scour. He said that, placed where it was, the end sill was '*probably … not [in] an optimum position …*'[579] Asked why that was not an optimum position, Mr Lesleighter said:[580]

> *It's a balance. The end sill is like something you're putting in the way to create turbulence. But on a hydraulic jump which is that long, say 50 metres long, or 60 metres, and you try to achieve everything in 20 metres by just putting an end sill, it is going to create a condition which is still very violent here downstream of it. The question therefore is, well, is that the right place to put an end sill? …*

575 Exhibit 237, **SWA.512.001.0578**, .0781 (emphasis added).
576 Exhibit 7, **IGE.017.0001**, .0023 and .0049.
577 Exhibit 7, **IGE.017.0001**, .0049.
578 Exhibit 7, **IGE.017.0001**, .0053 (emphasis added).
579 Exhibit 238, **TRA.510.019.0001**, .0033 ln 4-10.
580 Exhibit 238, **TRA.510.019.0001**, .0033 ln 17-25.

Did the failure of the end sill affect the scour?

6.337 Mr Lesleighter's opinion about the relationship between sill and scour was not in answer to a question about cause of the scour: it was in the context of considering whether reinstatement of the end sill may have been appropriate remediation[581] – an observation '*raised simply to question the extent of the reasoning that was applied*' to the proposed remediation works.[582]

6.338 The 2014 URS Review is more persuasive, however, involving detailed CFD analysis.

6.339 The loss of the end sill probably had an appreciable effect on the scour. However, it is not possible to determine the extent to which its loss exacerbated the situation.

The use of RCC

Introduction

6.340 This part is concerned with the possible causes of damage to the apron in the 2011 and 2013 events.[583]

6.341 RCC was used to build the apron. Key issues related to that choice concern:[584]

a. thickness of the RCC layers

b. reinforcing within the RCC

c. problems during construction

d. using RCC rather than CVC.

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581 Nor was it assisted by CFD analysis. At the time of the report, Mr Lesleighter stated that the concerns cannot be '*adequately answered without more detailed study than has ever been possible previously*': Exhibit 7, **IGE.017.0001**, .0051.

582 Exhibit 7, **IGE.017.0001**, .0049. A hydraulic model study was planned to '*resolve this particular but essentially short-term, concern*': Exhibit 7, **IGE.017.0001**, .0023.

583 According to Mr Herweynen, the ANCOLD Guidelines on Selection of Acceptable Flood Capacity for Dams, March 2000, contemplate some damage to a dam from extreme flood events so long as it does not put the dam at risk: **TRA.500.013.0001**, .0071 ln 31-36; Exhibit 245, **HER.002.0001**, .0003 [13]. Mr Herweynen said that designers consider two floods: a 'spillway design flood' and a 'check flood' (or 'safe flood'). The first is not meant to experience damage. The second, according to Mr Herweynen: '*can have damage, as long as it does not lead to dam failure, in other words, the mechanism doesn't progress. That's an acceptable practice within the ANCOLD guidelines*': Exhibit 247, **TRA.510.007.0001**, .0096 ln 22-31. Dr Schrader submitted that '*[i]t is common for dams to have severe damage when they experience their first major flood*': **SCE.036.0001**, .0014.

584 None of the matters has been shown to have affected the stability issue, that is, the scour immediately downstream of the apron in 2013, in a material way.

Thickness of RCC layers

6.342 The apron floor was built from two layers of RCC. The Detail Design Report stated that '*[a] minimum thickness of 2 layers (total 620mm) was considered to be adequate*'.[585]

6.343 A 900 mm thick apron had first been considered. Then, a 'declaration' was made by the Alliance that CVC was to be reduced by 30%.[586] The idea was to '*[r]educe total cost of cement by 30%*'.[587] So the thickness of the apron was reduced from 900 mm to 600 mm.[588]

6.344 The decision was '*made on the advice of Dr Schrader and only after a structural check was performed by Mr Griggs and reviewed by Herweynen*'.[589] The designers were satisfied that the reduction did not compromise the design intent. And there is no reason to suppose that the reduction contributed to the damage to the apron and sill.

Reinforcing within the RCC

6.345 Steel reinforcing was '*provided for a moderate degree of crack control in the top layer of the RCC*'.[590] The Detail Design Report said that it was placed '*centrally in the top layer*'.[591]

6.346 The first TRP, in a report dated October 2013, stated that the layers comprising the apron '*were not effectively reinforced further*' (but noted that they may have included some bedding mortar or even grout-enrichment).[592] The TRP expressed this opinion:[593]

> *What reinforcement was placed was probably too light and too widely-spaced plus the anchor bars were basically ineffective due to poor bond in the top anchorage length of the bar that was embedded at the bottom of either the top RCC layer or the next one below.*

6.347 The 2014 URS Review suggested that:[594]

> *… the dissipator slab damage was caused by … the use of RCC in lieu of conventional concrete* ***and the amount and location of reinforcement of the apron slab*** *...*

585 Exhibit 24, **GHD.002.0001**, .0188.
586 **DNR.005.4886**, .5019. The declaration was made on 18 November 2003.
587 **SUN.018.006.6124**, .6145.
588 **DNR.005.4886**, .5020.
589 **HYT.008.0001**, .0071 [243]. See **TRA.500.013.0001**, .0089 ln 24-40.
590 Exhibit 24, **GHD.002.0001**, .0188.
591 Exhibit 24, **GHD.002.0001**, .0188.
592 Exhibit 7, **IGE.017.0001**, .0023.
593 Exhibit 7, **IGE.017.0001**, .0023-4.
594 Exhibit 237, **SWA.512.001.0578**, .0588 (emphasis added).

6.348 That conclusion relates to the damage to the apron slab itself (see Figure 6.6 above). The 2014 URS Review, however, was conducted without access to construction records.[595] Mr Dann explained the basis for the report's conclusion:[596]

> *It was very difficult to work out how the reinforcing was installed. There wasn't a construction report that told us how it had been placed. I suppose my concern was that there were two layers of RCC shown on the drawings and that they would place one layer, put the reinforcing down and then put the RCC over the top.*
>
> *Now, from what I had seen from photographs, you can see the RCC within the top layer, in the lower portions, so at that point it seems that the reinforcing was within the top layer, but it didn't seem to meet any sort of precedent in terms of reinforced concrete apron slabs that I had seen before.*

6.349 The evidence does not sustain Mr Dann's reservations about the reinforcement.

6.350 In any event, there is nothing to demonstrate that better placed reinforcing would have reduced the damage that the apron sustained in 2011 or 2013.

Issues during construction

6.351 There were, it seems, problems in constructing the primary spillway apron attributable to rain and to resulting density problems in the RCC. Not all of those problems were remedied. For example, RCC placement stopped in June 2005. Mr Roberto (sometimes known as Robert) Montalvo, a site RCC engineer, recorded that rainwater drew paste to the surface '*affecting its appearance and affecting the homogeneity of the cement content in the layer*'.[597] Mr Montalvo wrote, however, that '*[i]t is expected that this will not cause structural problems, and that the excess paste on the surface will be removed after a flood*'.[598] Loose material was a concern to Mr Montalvo but by the time of his inspection the next day, 10 m of the RCC edge had been covered with bedding mix.[599] As a result, the edge could not be inspected to ensure that no loose material was present.[600]

6.352 There may also have been inadequate bonding of the lift joints. The 2016 Dam Safety Review observed:[601]

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595 **TRA.500.012.0001**, .0077 ln 30-31 and .0084 ln 9-20; Exhibit 237, **SWA.512.001.0578**, .0590. Hydro Tasmania submitted that '*causative connection between the 2010/11 flood and the damage sustained in 2013 is unclear and should not be dismissed without further investigation*': **HYT.008.0001**, .0160 [586].

596 **TRA.500.012.0001**, .0077 ln 29-41.

597 Exhibit 121, **SUN.117.004.0175**, .0177.

598 Exhibit 121, **SUN.117.004.0175**, .0177.

599 Exhibit 121, **SUN.117.004.0175**, .0177. See also **TRA.500.013.0001,** .0083 ln 35 to .0086 ln 10.

600 Exhibit 121, **SUN.117.004.0175**, .0177.

601 Exhibit 42, **DNR.002.3132**, .3244.

> *The dissipator apron floor (RCC) either cracked, broke up, and/or washed out to varying extent over an area of about 1,100 m^2 on the left apron end. This failure appeared to be caused by abrasion, delamination, and washout failure of the RCC layers, perhaps assisted by transient pressure uplift. Subsequent core drilling in the primary spillway apron and the secondary spillway abutment indicated that a lack of bond between RCC layers was widespread.*

6.353 Again, however, no causal relationship between those possible problems and damage to the apron is established.

RCC vs CVC

Background

6.354 The apron was built out of RCC. The RCC mix used on the upper layer had a higher cement content than the mix used in the mass of the Dam.[602]

6.355 The decision to use RCC was questioned. The first TRP considered the '*prime problem*' with the apron floor to be that it '*was not constructed in conventional heavily reinforced concrete, but rather in low cementitious content RCC*'.[603] Mr Dann regarded the use of it instead of CVC as undesirable.[604] In his experience, dissipators are generally designed to have reinforced concrete slabs.[605]

RCC chosen by the Alliance

6.356 Hydro Tasmania submitted that the Alliance's decision to use RCC was '*made on the basis of the preferable erosion resistance of RCC compared with conventional concrete*'.[606]

6.357 Initially, the apron was to be made of CVC. Then, Mr Herweynen said, a workshop was held to investigate changing to reinforced RCC.[607]

6.358 Advice was sought from Dr Ernest Schrader.[608] Dr Schrader produced a 1995 paper, 'Roller Compacted Concrete Cavitation & Erosion Resistance' (of which he was an author[609]) concluding that '*RCC has exceptional resistance to cavitation, erosion and abrasion*'. The paper gave examples of the use of RCC in aprons.[610] It referred to the results of a 1994 study by Omoregie, Gutschow and Rusell.[611]

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602 Exhibit 24,**GHD.002.0001**, .0188. See also **DNR.005.3763**, .3767.
603 Exhibit 7, **IGE.017.0001**, .0023.
604 **TRA.500.012.0001**, .0076 ln 24-28.
605 **TRA.500.012.0001**, .0076 ln 33-34.
606 **HYT.008.0001**, .0068 [228(b)].
607 Exhibit 244, **HER.001.0001**, .0026 [119].
608 Exhibit 244, **HER.001.0001**, .0026-7 [120].
609 Exhibit 24, **GHD.002.0001**, .2192.
610 Exhibit 244, **HER.001.0001**, .0027 [121].
611 See Exhibit 17, **PA-17**, .0407.

6.359 Mr Herweynen stated that '*[b]ased on the above, I was satisfied that the change could be implemented, and I had no reason to doubt the accuracy of Dr Schrader's expert input in this regard*'.[612]

6.360 Reference was made to Dr Schrader's paper, his examples and the 1994 study in the Detail Design Report.[613] The Detail Design Report concluded:[614]

> *Based on this information, reinforced RCC was adopted for the primary spillway dissipator apron. This decision was made not only for the resulting construction benefits, but also because it was seen to be a better design solution.*

It added:[615]

> *Reinforced RCC was considered to be less expensive than reinforced conventional concrete and therefore was the favoured material, if adequate erosion resistance could be demonstrated.*
>
> *Roller compacted concrete, when properly compacted, has demonstrated excellent resistance to erosion and cavitation damage. RCC has performed very well, and generally better than higher strength conventional concrete, when exposed to high velocity water, turbulent water, wearing by abrasion and debris, and overtopping of weirs or slabs.*
>
> *One of the reasons for this is that, due to the lower cement content of RCC, the amount of micro cracking that is likely to occur as a result of thermal and shrinkage stresses is less than for conventional concrete. It is this micro cracking that impacts on the long term durability of the material.*

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612 Exhibit 244, **HER.001.0001**, .0027 [124]. Hydro Tasmania submitted that there was no reason for Mr Herweynen to doubt the material provided by Dr Schrader: **HYT.008.0001**, .0067 [225]. SMEC submitted that '*The use of reinforced RCC for the design of the apron was on the recommendation of Dr Schrader, who had provided evidence of the material being used for a comparable purpose. Mr Neumaier and the dam design team followed that recommendation*': **SMC.001.0001**, .0010 [22].

613 The paper also appears at Exhibit 17, **PA-17**, .0395. It contained this conclusion: '*Laboratory studies, full scale tests, and field experience have all shown that, even at relatively low strengths and cementitious contents, RCC has exceptional resistance to cavitation, erosion, and abrasion damage from both high and low velocity water flows. This applies to both rough and smoothed surfaces*': Exhibit 24, **GHD.002.0001**, .0186.

614 Exhibit 24, **GHD.002.0001**, .0187.

615 Exhibit 24, **GHD.002.0001**, .0186. Hydro Tasmania submitted that the proposal submitted to Burnett Water included RCC and that '*There is no record of any concern being raised by Burnett Water, its technical advisors, SunWater, or the Regulator in relation to this decision during Stage 2, the Final Design Stage or thereafter. No concern was raised in relation to the use of reinforced RCC in the apron by the Alliance's peer reviewers*': **HYT.008.0001**, .0066 [222].

The debate concerning RCC and CVC

6.361 The apron was damaged in the 2011 and 2013 events, raising the possibility that the use of RCC made the damage more likely.[616]

6.362 The proposition that RCC was more erosion resistant than conventional concrete was described as '*unusual*' by Mr Dann.[617] His opinion was that the construction methods did not '*meet any sort of precedent*' that he had observed.[618] Mr Dann viewed the use of RCC instead of CVC for the apron surface as '*undesirable*'.[619] He elaborated:[620]

> *I have referred to precedent in dam design as being one factor in engineering judgment, about how you make decisions. Dissipator designs are generally reinforced concrete slabs. They have certain characteristics: they are anchored to the foundation with bars, where you have drains underneath them, where you have shear keys, where you have waterstops. There are just certain standard components of a dissipator design that we would run with, which is the approach that we took with our dissipator slab.*

6.363 However, Mr Dann, as he accepts, is not '*someone who has expertise, or specific expertise, in the placement of RCC*'.[621] And he does not deny that RCC is more resistant to erosion than CVC.[622]

6.364 The manual of the US Army Corps of Engineers (**USACE**) entitled 'Roller Compacted Concrete', dated 15 January 2000, states that concrete erosion is a '*major concern*'. Erosion damage can be caused by cavitation or abrasion: [623]

> *(1) Cavitation erosion. … RCC surfaces cannot be held to the same close tolerances as conventionally placed concrete with formed, slipformed, or screeded surfaces.* ***Therefore, a conventional concrete topping or facing may be required over RCC placements where the surface will be exposed to significant flowing water****. Duration of flow, however, is also a factor. For structures with infrequent, short-duration, high-velocity flows, it may be economically prudent to accept some cavitation damage in lieu of strict surface tolerance requirements.*

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616 The damage is summarised at paragraphs 6.18-6.27 and 6.276-6.279. The damage caused to the apron was consistent with a 'ball-mill' effect: see paragraphs 6.276-6.279. Hydro Tasmania said that it was possibly exacerbated by an 'asymmetrical vortex effect' **HYT.008.0001**, .0069 [230].

617 **TRA.500.012.0001**, .0077 ln 16. Including because of what he saw as the shortcomings in the location or placement of the reinforcing in the apron: see paragraph 6.347.

618 **TRA.500.012.0001**, .0077 ln 39; see also Exhibit 237, **SWA.512.001.0578**, .0588.

619 **TRA.500.012.0001**, .0076 ln 24-28.

620 **TRA.500.012.0001**, .0076 ln 31-40.

621 **TRA.500.012.0001**, .0093 ln 37-45; **HYT.008.0001**, .0067 [266].

622 **HYT.008.0001**, .0067 [226].

623 US Army Corps of Engineers, 'Roller-Compacted Concrete', EM 1110-2-2006, 15 January 2000, viewed 17 April 2020, <https://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-2006.pdf>, 5-10 (emphasis added).

> *(2) Abrasion erosion. Spillway aprons, stilling basins, and many other hydraulic structures may suffer surface erosion due to abrasion.* ***Concrete, whether RCC or conventionally placed, cannot withstand continued abrasive action from silt, sand, gravel, rocks, construction debris, or other waterborne debris without experiencing severe erosion problems****. RCC mixtures with a low water-cement ratio and large-size aggregates are expected to provide erosion resistance equal to a conventional concrete with similar ingredients. In circumstances where abrasion erosion or cavitation erosion is severe, a steel lining may be chosen to minimize maintenance and repair work. The embedments or anchorages required with steel linings do not lend themselves to RCC construction. Therefore, when steel linings are used, conventional concrete, placed to a depth sufficient to encapsulate the liner anchor system, is used over the RCC.*

6.365 However, in 'Roller-Compacted Mass Concrete: ACI 207.5R-99' reported by the American Concrete Institute (**ACI**) Committee '*caution*' is advised because '*high-velocity flows through RCC spillways have not yet been fully evaluated'*.[624] Although the report recognised that '*RCC mixtures should have comparable erosion resistance to conventional concrete of similar strength and NMSA [nominal maximum size aggregate]*', it stated:[625]

> *Spillways subjected to frequent high-velocity flows are still typically faced with conventional concrete. USACE recommends that spillways with velocities exceeding 24 ft/s (7.3 m/s) or frequent flow that is likely to result in maintenance problems by lined with conventional concrete.*

Would CVC have been a better design choice?

6.366 Dr Schrader said that CVC, '*even if it has a higher strength, has significantly less resistance to erosion than RCC*'.[626] The following exchange occurred in his evidence:[627]

> *Q. I want to suggest to you that one of the operative causes of the apron's failure in 2013 was the fact that it was constructed of RCC rather than conventional concrete?*
>
> *A. Well, you can suppose that. I think you are entirely wrong and I think the evidence would show that the conventional concrete, although it would look prettier, it probably would have failed even quicker.*

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624 American Concrete Institute, 'Report on Roller-Compacted Mass Concrete', ACI 207.5R-11, July 2011, 29.

625 American Concrete Institute, 'Report on Roller-Compacted Mass Concrete' ACI 207.5R-11, July 2011, 29.

626 **SCE.036.0001**, .0007.

627 **TRA.500.010.0001,** .0066 ln 31-37.

6.367 He added: '*If conventional concrete had been used the damage most probably would have been worse*'.[628] Mr Neumaier has a similar opinion. He believed that '*[a] conventional concrete slab could have been just as easily damaged by abrasion*'.[629] And Mr Dann accepted that damage from ball-milling could occur to a CVC apron, though he considered it '*quite speculative*'.[630]

6.368 Although, given the damage they observed to the apron in 2013, Mr Herweynen[631] and Mr Neumaier[632] would be reluctant to use RCC in such an apron again, that caution is not based upon a conviction that RCC is not suitable for the purpose.

6.369 In summary, the evidence does not establish that CVC would have been a better design choice or that the damage in 2011 or 2013 would have been different had CVC been used.

Peer review of the apron design

Background

6.370 Experts were engaged in about January 2004 to review aspects of the design work. In a memorandum dated 27 January 2004, Mr Neumaier described the peer reviewers' work:[633]

> *Each reviewer is required to prepare a review report stating the issues which have been covered by the review and provide recommendations of how, in their opinion, the proposed design could be improved.*

6.371 Mr Neumaier's memorandum attached a Peer Review Brief which was sent to reviewers. The 'Review Requirements' were broad, describing the reviewers' task as conducting '*an independent review of those components of the Burnett Dam Project in which they are recognised experts*'.[634] The tasks comprised:[635]

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628 **SCE.036.0001**, .0014.

629 **TRA.500.015.0001,** .0020 ln 4-5. Further, SMEC submitted that Mr Neumaier's concession '*says nothing about the merits of the decision to use RCC in the design of the apron at the relevant time*': **SMC.001.0001**, .0011 [25].

630 **TRA.500.012.0001**, .0094 ln 24-32.

631 **TRA.500.013.0001**, .0083 ln 27-28.

632 **TRA.500.015.0001,** .0019 ln 40-45.

633 Exhibit 232, **DNR.010.0929**, .0929. Mr MacGregor peer reviewed the 'engineering geology'. He commented on the foundation strength, permeability, stability and construction materials. He reviewed the concept design reports, reports on design investigations prepared by SunWater, the stage 2 design report prepared during tender, drawings prepared during the initial stages of detailed design and the preliminary results of investigations by Golder: Exhibit 232, **DNR.010.0929**, .0945. Mr Shannon was responsible for the peer review of foundation adequacy. In April 2004, it was reported that '*[d]ental treatment downstream of the primary spillway dissipator is desirable*'. The solution recorded is riprap downstream and '*some dental treatment of rock*': Exhibit 25, **DNR.001.0267**, .0325. Neither Mr MacGregor's nor Mr Shannon's peer review matters for present purposes. The peer reviews are summarised in the Final Design Report: Exhibit 25, **DNR.001.0267**, .0322.

634 Exhibit 232, **DNR.010.0929**, .0936.

- *Review project objectives and functional requirements*
- *Review the Stage 2 submission and subsequent modifications made during the detailed design stage*
- *Comment on critical aspects of the design in terms of safety, environmental impact, quality, functionality, scheduling, innovation and constructability*
- *Suggest, where appropriate, technical or operational improvements, cost and time savings or risk reductions*
- *Comment on adequacy and quality of design practices and management procedures*
- *Prepare a peer review report stating the issues which have been covered by the review and any suggestions of how the proposed detailed design can be improved.*

Limited peer review of hydraulic aspects

6.372 Mr Lesleighter was engaged to review the design work on '*hydraulic structures*'. According to Mr Herweynen, Mr Lesleighter was engaged '*predominantly around the intake outlet hydraulics*'.[636] Mr Lesleighter said that '*[e]ven though I was nominated as the peer reviewer for hydraulics, I was never asked to look at the spillway'*.[637] His consideration was '*restricted to the outlet works*'.[638] This left a gap.

6.373 In the 2014 URS Review, URS concluded that:[639]

> *1. … there is no evidence of independent technical reviews being carried out on either the spillway design or the physical model study. This is a concern given the large PMPDF spillway discharges for this project.*

6.374 URS's conclusion was put to Mr Neumaier, who said:[640]

> *Point 1, whether we had an independent peer reviewer for the hydraulic design. Not as such, not somebody who is a hydraulic specialist, who does that as his profession. However, the hydraulic jumps and those formulae we used – they would be very familiar to most civil engineers who worked in the dam and hydraulic structures side of work, so Mr Linard, for one, would have had a view on it, yes. But we didn't have a nominated hydraulic peer review expert on the team.*

635 Exhibit 232, **DNR.010.0929**, .0936.
636 Exhibit 247, **TRA.510.007.0001**, .0014. See also Exhibit 244, **HER.001.0001**, .0013 [52].
637 **TRA.500.012.0001**, .0007 ln 21-22.
638 **TRA.500.012.0001**, .0007 ln 36-38.
639 Exhibit 237, **SWA.512.001.0578**, .0586.
640 **TRA.500.015.0001**, .0018 ln 37-45.

6.375 Jack Linard carried out an '*internal review of the project design features*'.[641] His comments, however, were limited to the '*Dam Foundation Excavation Design*' and '*Foundation Grouting Procedures*'.[642]

6.376 Mr Lesleighter's peer review did not cover the apron; and no one else was asked to peer review its design.

Importance of peer reviews

6.377 In Mr Dann's opinion, given the significant discharges expected, it was important to have a technical review panel checking that the design team had the '*right answer[s]*'. He elaborated:[643]

> *I suppose on that point 1, we're highlighting there that the flows that you are designing the spillway structure for were very, very large, almost unprecedented, and if you are looking at what level of confidence you want in your design, to me, once you see the size of that flood that you are designing for,* ***you need that second check or that review panel check to say, 'This is almost record breaking in terms of its discharge capacity. Are we confident that we have the right answer there?'*** *That's part of the reason we built that expert review panel during our tender process, because that was well recognised.*

6.378 Speaking of peer review panels generally, Mr Dann said: [644]

> *They have a number of functions. They look at the data that we are collecting for the design, and that includes a broad range of things - hydrology, earthquakes, geotechnical conditions. They look at the concepts that have been developed, the analysis that is undertaken to support it*
>
> *…*
>
> *I've always used an expert review panel on the design projects that we have undertaken for major dam projects, whether they be upgrades or new dams. They are invaluable, really, in terms of the process. There is a commercial aspect to doing a design where there is pressure on cost, there is pressure on time, and the review panel is independent. They say, 'All right, we've got some information here'. Let's take geotechnical information as an example. 'Is that adequate?' And it's a fair question to ask, how much you need to invest in, say, your geotechnical investigation to inform your design decisions.*
>
> *Generally, there's an iterative process as you develop a design. You will get an initial phase of geotechnical investigations. You will use that to develop your concepts, and then as you progress the concepts, you will do further investigations …*

641 Exhibit 232, **DNR.010.0929**, .0931.
642 Exhibit 232, **DNR.010.0929**, .0958-9.
643 **TRA.500.012.0001**, .0071 ln 21-31 (emphasis added).
644 **TRA.500.012.0001**, .0064 ln 2-41. URS engaged a peer review panel as part of its stage 2 process: see paragraphs 6.217-6.219.

6.379 Mr Herweynen considered that, although there was '*decent governance*', an independent review panel during design and construction '*probably*' would have '*benefited the project*'.[645]

6.380 Hydro Tasmania submitted:[646]

> *Peer reviews were undertaken in respect of a number of elements of the design work in the final stages of the Detail Design Phase to check that the key objectives of the project were met and the functional requirements of the Terms of Reference were satisfied. The independent reviewers had significant expertise in fields including geology and geotechnical engineering, dam and spillway design and construction, and hydraulic modelling and design. Each peer reviewer produced a report or a number of reports and the Final Design Report contains a record of the recommendations of each reviewer and the actions taken in response.*

6.381 SMEC submitted that '*[t]he experts engaged and breadth of the review was sufficient, Mr Neumaier anticipated, to cover all material aspects of the design*'.[647]

6.382 But there was a deficiency. The peer review of hydraulics structures did not include the apron or end sill.[648]

6.383 A more expansive peer review of hydraulic aspects to include the apron would have conformed with engineering good practice.[649]

What result would a more expansive hydraulic peer review have produced?

6.384 The evidence did not disclose what the likely result of a satisfactory hydraulic peer review would have been. Who might have been chosen to review the design of the apron is speculative. Assuming it was Mr Lesleighter, there is doubt about the advice he would have given. In 2012, Mr Lesleighter did identify the '*appreciable scale effects*' that would have affected the Alliance's 3D model and the problematic flows as a result of the apron's asymmetry.[650] He considered that the problems experienced in the 2011 event would be '*likely to be present at a range of discharges – not just that experienced in 2011*'.[651] Mr Lesleighter's comments, while prescient, do not show what he would probably have advised years before. And if it had been suggested in peer review that at 20 m the apron was too narrow, there is no evidence whether such a concern would have been acted upon.

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645 Exhibit 247, **TRA.510.007.0001**, .0014 ln 36-39.

646 **HYT.008.0001**, .0162 [598] (footnotes from original omitted).

647 **SMC.001.0001**, .0009-10 [20].

648 SMEC submitted that '*As part of the design procedure, the Alliance engaged an independent review panel to consider the elements of the design work and to ensure that the overall design was acceptable and tested*': **SMC.001.0001**, .0008 [16]. Hydro Tasmania submitted that the contemporaneous evidence '*establishes that it was Mr Neumaier, as design manager, who commissioned and briefed those involved in the peer review*': **HYT.008.0001**, .0020 [51].

649 **TRA.500.012.0001**, .0063 ln 40 to .0064 ln 41.

650 Exhibit 230, **DNR.006.3156**, .3241. Mr Lesleighter's report is dated 16 November 2012.

651 Exhibit 230, **DNR.006.3156**, .3242.

6.385 However, a more expansive peer review of hydraulic aspects to cover downstream protection would accord with engineering good practice. It would have subjected to the scrutiny of other experts the adequacy of the apron and afforded an opportunity to detect and correct problems and oversights. Peer review is the accepted means to avoid problems of the kind identified. The Alliance's own Design Management Plan required the carrying out of '*verification … by suitably qualified and independent persons, to ensure that'* functional and operational requirements were met.[652]

6.386 It cannot be known with certainty what the result of peer review would have been. But this accepted means to discover and correct problems was absent. And its absence created a significant risk of inadequate design of the apron. In this sense, the lack of full peer review of hydraulic structures was a root cause of the apron's inadequate width (and, therefore, also the scouring).

Conclusions

6.387 The asymmetry of the primary spillway apron resulted in its abrasion by rock – the 'ball-mill' effect – that damaged the apron in both the 2011 and 2013 events.

6.388 A root cause of the 2013 scour immediately downstream of the apron was the apron's insufficient 20 m width. The failure of the end sill had an appreciable, but unquantifiable, impact on that scouring.

6.389 Peer review of hydraulic issues did not extend to the design of the apron. That omission, which departed from engineering good practice, was a root cause of its insufficient width and the scouring.

Recommendation # 6

The designer of a dam should give proper consideration to the erosive force of water and the capacity of the riverbed to withstand such force. This may include testing and simulation using computational and hydraulic modelling, as well as geotechnical investigations (and the interaction between those disciplines).

652 **SUN.162.002.0149**, .0169.

Chapter 7 – Governance

Introduction

7.1 The Terms of Reference require consideration of governance arrangements for the design, commissioning and construction of Paradise Dam (**the Dam**). They also invite recommendations to ensure that future dam projects are designed, constructed and commissioned to acceptable standards, as defined in Queensland legislation and regulations, ANCOLD guidelines, and engineering good practice.

7.2 This Chapter considers the applicable legislative and regulatory regimes as well as governance of the project.

7.3 The Commission issued a discussion paper on governance to parties with leave to appear. It discussed legislative and regulatory arrangements and invited submissions on such topics as the role of the Regulator and the organisational structure for the Dam's design and construction. Three parties responded: the Department of Natural Resources, Mines and Energy; the Department of State Development, Manufacturing, Infrastructure and Planning; and SunWater Limited (**SunWater**).

The role of the relevant entities

Burnett Water: the 'special purpose vehicle'

7.4 Burnett Water Pty Ltd (**Burnett Water**) was incorporated in June 2001 through an initiative of the then Department of State Development. It has always been registered under the *Corporations Act 2001* (Cth) and not as a statutory or government-owned corporation in a strict sense. Its shares were originally held by a State employee on trust for the State of Queensland acting through the then Minister for State Development.[1] The creation of this corporation was to ensure separation of the Department's different functions as proponent and assessor.[2]

7.5 One of the objects in Burnett Water's Constitution was to build, own and operate water infrastructure.[3] It was incorporated primarily to undertake the impact assessment for the Dam and to obtain necessary approvals. It was not to have long-term involvement with the Dam.[4] At the start, the period during which the company was to remain in Government ownership was uncertain. A competitive process was proposed for delivery of the project once approvals were obtained. After that, the company could be sold as a vehicle for later development of the infrastructure, if the Government so decided.[5]

7.6 Selected as a 'special purpose vehicle', initially Burnett Water was seen as '*essentially a shelf company with minimal financial and human resources*'.[6] 'Special purpose vehicle' connotes a legal entity: created to fulfil specific or temporary

1 **DSD.008.0001**, .0005.
2 **DSD.008.0001**, .0096.
3 **DSD.008.0001**, .0055.
4 **DSD.007.0001**, .0002.
5 **DSD.008.0001**, .0001.
6 **DSD.008.0001**, .0123.

objectives; which isolates the risk of the undertaking from its creator; and which is not itself a repository of technical expertise and experience, although it may access such knowledge by engaging others.

7.7 Five public servants were made available to Burnett Water by the former Department of State Development and Department of Natural Resources and Mines under a 'funding agreement'.[7] Its two directors were from the private sector.[8]

7.8 Burnett Water acquired interests in land associated with the Dam through various freeholds, a leasehold and easements.

7.9 In December 2002, the then Minister for State Development announced that the Dam was to be delivered by an alliance model (**the Alliance**),[9] chosen following advice from Deloitte Touche Tohmatsu and Evans and Peck.[10]

7.10 Upon the Dam's commissioning, Burnett Water became a wholly owned subsidiary of SunWater.

SunWater Limited

7.11 SunWater is a government owned corporation (**GOC**). It began as a statutory GOC. From July 2008, it was both a company GOC and a public company limited by shares.

7.12 SunWater contributed to planning and design of the Dam by: a Preliminary Design Report for Burnett Water in around February 2003;[11] preparation of the Failure Impact Assessment (**FIA**) in June 2003;[12] a Dam Break Analysis dated July 2003;[13] hydraulic modelling services under the direction of the Alliance;[14] and a peer review of foundation adequacy by Brian Shannon, Chief Design Engineer at SunWater.[15]

7.13 On 1 July 2004, it was announced that SunWater would acquire Burnett Water and that the Dam was to be its responsibility.

7.14 SunWater became the 'owner' of the Dam by acquiring Burnett Water (which held the interests in land for the Dam and its reservoir)[16] after the Dam had reached practical completion.[17]

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7 **DSD.008.0001**, .0109: clauses [38] and [41] Funding Agreement.
8 **DSD.008.0001**, .0002.
9 **DSD.007.0001**, .0005.
10 **DSD.006.0001**, .0003.
11 Exhibit 96, **DNR.003.7930**, .7935; the Preliminary Design Report was prepared in the name of 'Burnett Water Pty Ltd'. The executive summary recorded that the '*work was carried out by SunWater for Burnett Water Pty Ltd*'.
12 Exhibit 17, **PTA.001.0001**, Attachment PA-7.
13 **DNR.006.0395**, .0448.
14 **SUN.011.0001**, .0003 [8].
15 Exhibit 90, **DNR.010.0918.**
16 **SUN.011.0001**, .0003 [11].
17 Exhibit 306, **DNR.005.0584**, .0828.

Alliance model

7.15 The Dam's design and construction were to be delivered by adopting an alliance model.

7.16 Burnett Water was a member of the Alliance.[18] SunWater was not.

7.17 Different responsibilities were assigned to Alliance participants. By clause 6.3 of the Alliance Agreement,[19] Burnett Water was, in summary, to:

a. arrange funding

b. advise of its technical and logistical requirements for the project

c. liaise with public relations, government and other stakeholders

d. assist in cultural heritage management

e. assist in planning, engineering and construction issues

f. assist in financial management of the project

g. obtain approvals

h. secure the right to construct on site

i. assist with project management.

7.18 Under Clause 6.4 and Schedule 7, the responsibilities for the design and construction of the Dam were allocated to other parties.[20]

7.19 The Alliance Agreement reflected Burnett Water's limited role in the project. No directors of Burnett Water were members of the Project Alliance Board when the main construction work was underway.

Organisational structure

7.20 The discussion paper raised these issues regarding organisational structure:

> *Should the arrangements with respect to Burnett Water have been different, for example:*
>
> *a. ought Burnett Water to have stood outside the Alliance?*
>
> *b. ought the ultimate operator of the Dam (whoever that was to be) to have been more closely involved in the Dam's design, construction and commissioning, including to give a more direct connection between those designing, constructing and commissioning the Dam and the entity who would ultimately be responsible for its day to day operation?*

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18 **ALL.144.002.0258**.
19 **ALL.144.002.0258**, .0277-.0278.
20 **ALL.144.002.0258**, .0279, .0331.

c. *ought SunWater to have been a member of the Alliance, once the decision was made that it was to operate the Dam?*

d. *would different arrangements have assisted the better design, construction and commissioning of the Dam and if so, how, and what are the suggested other (better) arrangements?*

e. *would it have improved governance arrangements for the Dam if it had been made known earlier that it SunWater that was ultimately to have responsibility for the Dam, for example at the time the Dam was being designed?*

Efficacy of a special purpose vehicle

7.21 Burnett Water was chosen to be a 'special purpose vehicle' when the ultimate ownership of the Dam had not been decided. The then Department of State Development tested the potential for a public private partnership model for the Dam.[21] In the result, such a model was thought unlikely to achieve the timeframes needed to attract private investor interest or to offer compelling value for money.[22]

7.22 Attention was given to the possibility that SunWater would become the owner and, if so, what that could mean for its involvement with the Dam in the meantime. It was said at a business case workshop that:[23]

> *… if SunWater [were] to be the eventual owner, there would not appear to be any significant input that it would offer beyond what it should currently be offering as a service provider (design) to Burnett Water Pty Ltd.*

7.23 SunWater was to be kept '*at a distance*' to maintain integrity in the competitive tender process.[24] It was not to be involved in evaluation or decision making. It could, however, be '*used as a source of information and expert opinion in testing assumptions and findings given [its] industry knowledge*'.[25] Concerns were held about the time it might take for SunWater to commit to being the owner, if selected as such by Government.[26]

7.24 Burnett Water approached the project on the basis that there was a distinction between ownership and implementation.[27] It was to 'drive forward' the project as 'proponent' until the owner was chosen and able to take over.[28]

7.25 Potential issues in separating proponent from owner were discussed at workshops. One concern was that a potential eventual owner might be troubled about acquiring the Dam if it had not participated in design and construction. The solution was to

21 **DSD.007.0001**, .0003 [12].
22 **DSD.007.0001**, .0004 [15].
23 **DSD.008.0001**, .0122.
24 **DSD.008.0001**, .0125.
25 **DSD.008.0001**, .0125.
26 **DSD.008.0001**, .0122.
27 **DSD.008.0001**, .0114.
28 **DSD.008.0001**, .0114.

build a 'robust' dam to quality and operational specifications that would make it an attractive business proposition.[29] Another issue was whether '*inadequate participation by the client and or owner/operator*' could reduce the success of an alliance model.[30] That was addressed in this way: '*[a]lthough owner needs to make major decisions, appropriate decisions should be able to be made from an owner's perspective by government with appropriate advisers'.*[31]

7.26 Delay in deciding upon ownership meant that Burnett Water continued as a special purpose vehicle for longer than was initially intended. And as time passed and the Dam progressed, its responsibilities expanded.[32]

7.27 A discussion paper prepared by Burnett Water in May 2004 noted that private sector interest in the Dam appeared to have waned.[33] That same month, the Government decided that SunWater would be the ultimate 'owner'.[34]

7.28 Witnesses spoke of problems potentially associated with a special purpose vehicle that is not the owner.[35] Richard Herweynen would prefer to have a 'true client' – the end operator – involved in the project. Such a client, he believes, is better placed to test functional requirements.[36] Graeme Newton holds a similar view.[37] He considers that involving the end user enhances accountability.[38] Christopher Dann dealt with Burnett Water as Design Manager for the competing tender team.[39] SunWater had made David Murray available to Burnett Water to provide advice in the early stages of design but more as a 'facilitator' who, Mr Dann said, was '*very careful not to direct decisions*'. Mr Dann found that different from having the benefit of the input of an owner's perspective:[40]

> *… **we had to put our owner's hat on**, if you like to think about it from an owner's perspective, as if we were to provide a recommendation to an owner ...*

7.29 Mr Murray's involvement ceased after the tender was awarded.[41]

7.30 Consistently with its Constitution, Burnett Water had the power to extend its role to design and construction after its primary goal was completed. However, it did not itself possess all relevant expertise.[42] It was comprised initially of five employees. SunWater was needed for information and expert opinion.[43] This arrangement had its limitations. Although Burnett Water took advantage of the expertise of SunWater's

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29 **DSD.008.0001**, .0122.
30 **DSD.008.0001**, .0131.
31 **DSD.008.0001**, .0122.
32 **DSD.007.0001**, .0005 [20].
33 **DSD.008.0001**, .0145.
34 **DSD.007.0001**, .0006 [23].
35 Exhibit 247, **TRA.510.007.0001**, .0107 ln 18-42.
36 Exhibit 247, **TRA.510.007.0001**, .0107 ln 39-42.
37 **TRA.510.010.0001**, .0017 ln 37-43. Mr Newton was seconded from the Department of State Development to Burnett Water. His title was 'general manager': **TRA.510.010.0001**, .0002 ln 15-23. He was not, however, General Manager of the project, but had more limited responsibilities.
38 **TRA.510.010.0001**, .0018 ln 3, 7.
39 Exhibit 241, **DAC.001.0001**, .0003 [7].
40 **TRA.500.012.0001**, .0099 ln 42-.0100 ln 3 (emphasis added).

staff, it did so only in the design stage.[44] SunWater's aid was not sought during construction.[45]

7.31 Before the Alliance existed, the then Department of State Development acknowledged that SunWater *'holds much of the Government's expertise in dam design and construction*'.[46] It could have provided technical input and accountability as the prospective owner had it been identified as such before the Alliance was formed.

7.32 The Department of State Development, Manufacturing, Infrastructure and Planning accepts that, 'in theory', it is preferable for the ultimate owner to be involved in design, procurement and construction to promote operational efficiency.[47] That Department considers the decision about the Dam's ownership was:[48]

> *… made complex by parallel policy objectives in relation to optimising opportunities for private sector investment and ensuring the Queensland Government's commitments under National Competition Policy to challenge traditional State ownership models for water infrastructure.*

7.33 This had the effect of delaying the decision on ownership.

7.34 SunWater accepts that identifying it as future owner when the Dam was being designed could 'possibly' have improved governance.[49] That perception looks to be right.

7.35 Burnett Water's access to SunWater's resources, through contractual arrangements and less formal requests for advice, presents as an imperfect substitute for having its own expertise. Project governance would likely have been enhanced by early involvement of an eventual long term owner.

Efficacy of the alliance model

7.36 SunWater considers that there are no inherent deficiencies in alliance arrangements to build large dams.[50] Good governance is needed throughout an alliance project, to be achieved through selection of qualified participants, engagement of a skilled workforce, adequate oversight of design and construction as well as a rigorous document management system. SunWater also says that alliance arrangements typically involve an owner-participant. Were it otherwise, such a model would more resemble a 'design and construct' arrangement. Its position is that, where the ultimate owner is known before the formation of an alliance to construct a dam, it

41 **TRA.500.014.0001**, .0062 ln 2-15.
42 **DSD.008.0001**, .0118.
43 **DSD.008.0001**, .0125.
44 **SUN.011.0001**, .0002 [8].
45 **SUN.011.0001**, .0003 [9].
46 **DSD.008.0001**, .0118.
47 **DSD.006.0001**, .0003 [10].
48 **DSD.006.0001**, .0003 - .0004 [10].
49 **SUN.011.0001**, .0007 [41].
50 **SUN.011.0001**, .0004 [18].

would be beneficial for that entity to be involved in design, construction and commissioning.

7.37 SunWater suggests that Burnett Water's participation in the Alliance was important[51] and that Burnett Water's reliance on other Alliance participants was appropriate as they were experienced in design and construction.[52]

7.38 Those propositions are acceptable.

7.39 SunWater and the Department of State Development, Manufacturing, Infrastructure and Planning say that not identifying SunWater as the owner until after the Alliance was formed did impede its ability to become involved in the Alliance.[53] However, SunWater does not consider that it should have joined the Alliance once it was known that it would acquire Burnett Water.[54] In SunWater's view, it would be commercially challenging for an alliance to have both current and future owners as members because '*[i]ndustry is generally reluctant to accept a model where there are two state "masters*"'.[55] In any event, the evidence does not warrant an inference that, had SunWater been admitted to Alliance membership after July 2004, that participation would have brought better project governance.

Beginnings of the Dam project

Governmental and legislative steps

7.40 The *Water Infrastructure Development (Burnett Basin) Act 2001* established a project to investigate the feasibility of carrying out, as the 'principal component', an upgraded water storage and distribution infrastructure for cane irrigation in the Burnett Basin.[56] That Act deemed the principal component to be a 'significant project' requiring an Environmental Impact Statement (**EIS**) under the *State Development and Public Works Organisation Act 1971*.[57] The declaration of a 'significant project' was something that the Coordinator-General was empowered to make.[58]

7.41 The *Water Infrastructure Development (Burnett Basin) Act* also provided that the relevant parts of the 'Bundaberg 2000+ project' terms of reference were to be the terms of reference for the EIS to be prepared for the principal component.[59] This emerged from the 'Water for Bundaberg 2001' policy released by the Queensland Government. It had proposed the investigation of a suite of infrastructure projects in the Burnett River catchment.

7.42 The Coordinator-General evaluated the EIS under s 35 of the *State Development and Public Works Organisation Act.* One recommendation was that the land affected

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51 **SUN.011.0001**, .0005 [22]-[23].
52 **SUN.011.0001**, .0004 [20].
53 **SUN.011.0001**, .0006 [31]-[33]; **DSD.006.0001**, .0003 [10].
54 **SUN.011.0001**, .0006 [30].
55 **SUN.011.0001**, .0006 [29].
56 *Water Infrastructure Development (Burnett Basin) Act 2001* ss 4 and 5.
57 *Water Infrastructure Development (Burnett Basin) Act 2001* ss 6 and 7.
58 *State Development and Public Works Organisation Act 1971* (Reprint 2B) s 29B(1).
59 *Water Infrastructure Development (Burnett Basin) Act 2001* s 8 and Sch.

by the project be designated 'community infrastructure' under Part 6 and Schedule 7 of the *Integrated Planning Act 1997*.[60] The then Minister for State Development designated that land as land for community infrastructure in October 2002.[61] It was stated in the Notice of a Ministerial Designation of Land that Burnett Water intended to construct and operate the proposed Burnett River Dam on the land.[62]

7.43 Thus planning approval for the Dam became a responsibility of the Chief Executive of the then Department of Natural Resources and Mines and not of local councils with local authority jurisdiction over the areas to be used for the Dam and its reservoir. The Chief Executive was the 'assessment manager' for planning approval purposes[63] and issued the development permits for the Dam.

The legislative scheme

7.44 When the Dam was planned and built, the *Water Act 2000* was in force and made provision for 'referable dams'.[64] Section 481(1) provided:

> ***481 Meaning of 'referable dam'***
>
> *(1) A dam is, or a proposed dam after its construction will be, a 'referable dam' if -*
>
> *(a) a failure impact assessment of the dam, or for the proposed dam, is required to be carried out under this part; and*
>
> *(b) the assessment states the dam has, or the proposed dam after its construction will have, a category 1 or category 2 failure impact rating; and*
>
> *(c) the chief executive has, under s 487, accepted the assessment.*

7.45 The number of people at risk as identified in the FIA determined whether a dam had, or would have, a Category 1 or Category 2 failure impact rating.[65] If, upon failure, the population at risk would be two to 100 persons, it attracted a Category 1 failure impact rating.[66] If the FIA indicated more than 100 persons would be at risk upon failure, the dam attracted a Category 2 failure impact rating.[67] The Dam was assessed as having a Category 2 rating.

7.46 An FIA was to be carried out by a registered professional engineer who was not an owner, operator or employee.[68] That assessment was required only if the completed

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60 **DNR.020.018.7219**, .7244.
61 **DSD.003.0001**; *Integrated Planning Act 1997* (Reprint 3B) s 2.6.8 and Sch 7.
62 **DSD.003.0001**, .0001.
63 *Integrated Planning Act 1997* (Reprint 3B) ss 3.5.15; 3.1.7(1)(b)(i); *Integrated Planning Regulation 1998* (Reprint 3E) Sch 1A, column 1, item 2(a)(iii).
64 *Water Act 2000* (Reprint 2F) s 481.
65 *Water Act 2000* (Reprint 2F) s 481(1)(b).
66 *Water Act 2000* (Reprint 2F) s 484(1)(a).
67 *Water Act 2000* (Reprint 2F) s 484(1)(b).
68 *Water Act 2000* (Reprint 2F) s 482.

dam was to be greater than 8 m in height, more than 500 ML in storage capacity, or more than 8 m in height, a storage capacity of more than 250 ML and a catchment area more than three times its surface area at full supply level.[69]

7.47 *Guidelines for the Failure Impact Assessment of Water Dams* were published by the then Department of Natural Resources and Mines in April 2002.[70] They regarded a dam as having failed when any part physically collapsed or if there was an uncontrolled release of content.

7.48 The Dam always possessed the characteristics of a referable dam. It was proposed in the FIA to be about 35 m high (crest elevation 67.6 m AHD) and with a capacity of 300,000 ML. The FIA was carried out and certified by a Registered Professional Engineer of Queensland (**RPEQ**) (an employee of SunWater) and had a Category 2 rating.[71] That assessment was accepted, and notice of that was given to Burnett Water.[72]

7.49 Construction of a dam engaged two legislative regimes and required two types of permission: put shortly, under the *Integrated Planning Act,* to carry out works; and to interfere with water under the *Water Act 2000*. The first of these was the source of power to impose conditions on the development permit.

Integrated Planning Act

7.50 The *Integrated Planning Act* provided that operational works which would result in the taking of water[73] (as defined in the *Water Act 2000*[74]), or the construction of a referable dam[75] were 'assessable works' and required a development permit.[76] For assessable works, development could commence when the development permit took effect.[77] Works were required to be within the extent and conditions of the development permit.[78]

7.51 Applications for development approval were to be made to the assessment manager,[79] who was the Chief Executive administering the *Water Act 2000* for the purposes of the Dam.[80]

7.52 Burnett Water was granted a development permit authorising construction of operational works to facilitate the taking of, and interference with, water subject to any licence or other authorisation under the *Water Act 2000*.[81] That permit was

69 *Water Act 2000* (Reprint 2F) s 483.
70 Exhibit 17, **PTA.001.0001**, Attachment PA-2.
71 Exhibit 17, **PTA.001.0001**, .0003 and Attachment PA-7.
72 Exhibit 17, **PTA.001.0001**, .0004 and Attachment PA-8.
73 *Integrated Planning Act* (Reprint 4N) Sch 8 Pt 1 item 3B.
74 *Water Act 2000* (Reprint 3A) Sch 4.
75 *Integrated Planning Act 1997* (Reprint 4N) and Sch 8 Pt 1 Item 3C(a).
76 *Integrated Planning Act 1997* (Reprint 4N) s 3.1.4(1).
77 *Integrated Planning Act 1997* (Reprint 4N) s 3.5.20.
78 *Integrated Planning Act 1997* (Reprint 4N) s 3.1.5(3).
79 *Integrated Planning Act 1997* (Reprint 4N) s 3.2.1(1).
80 *Integrated Planning Regulation 1998* (Reprint 3H) Sch 1A, Part 2, Item 2.
81 Exhibit 17, **PTA.001.0001**, .0005 and PA-19.

subject to 'safety conditions' regarding the design and construction of the Dam. Those conditions were imposed by the Chief Executive, as assessment manager.[82]

The Regulator

7.53 The *Water Act 2000* established the 'regulator', who was also the Chief Executive.[83] When the Dam was designed and built, Peter Allen was the recipient of delegated powers from the Chief Executive under the *Water Act 2000* to fulfil the role of 'Regulator'.[84] Mr Allen's position was titled 'Director, Dam Safety'.

7.54 The Regulator's functions included reviewing and making recommendations about standards and practices and monitoring compliance with the *Water Act 2000*.[85]

7.55 By s 515:

> ***515 Regulator's general functions***
>
> *(1) The regulator's general functions are—*
>
> *(a) to keep a register of service providers registered under this Act; and*
>
> *(b) to review and make recommendations about standards and practices under this chapter; and*
>
> *(c) to monitor compliance with this chapter; and*
>
> *(d) to perform other functions given to the regulator under this Act or another Act.*
>
> (2) *In performing the regulator's functions, the regulator must consider the purposes of this Act.*

7.56 A similar provision is now in Chapter 2, Part 2 of the *Water Supply (Safety and Reliability) Act 2008.*

7.57 It was submitted for the Department of Natural Resources, Mines and Energy that the Dam became 'referable' only *after* its construction and that the powers of the Regulator under the *Water Act 2000* applied only to referable dams.[86] The practical effect of that submission is that the general functions of the Regulator would not have been engaged until the Dam was built, at least with respect to monitoring compliance with Chapter 3 of the *Water Act 2000* (in which s 515 was located).

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82 Further permits were issued in 2004 and 2005, also under the *Integrated Planning Act 1997*. (Exhibit 17, **PTA.001.0001**, .0006; Exhibit 30, **DNR.003.7192** (6 Oct 2005); Exhibit 29, **DNR.003.7159** (3 June 2004)). They dealt with changes to the Dam's design after the initial approval and are not material.

83 *Water Act 2000* (Reprint 3A) s 514.

84 The Regulator could delegate functions and powers to an appropriately qualified officer of the Department: *Water Act 2000* (Reprint 3A) ss 520(1), 1012.

85 *Water Act 2000* (Reprint 3A) s 515.

86 **DNR.021.0001**, .0002 [9] - .0003 [10].

7.58 Another interpretation seems preferable: that a dam became refer*able* (i.e. liable to being referred) upon its likely possession of certain characteristics, that being recorded in an FIA, and the acceptance of that. Were it otherwise, the Regulator's function under s 515(1)(c) would not arise until the dam was finished. That would be too late to monitor compliance with Chapter 3, which concerns (among other things) steps necessary *before* construction. For example, an FIA was required to be done by a person who proposed to construct a dam in certain circumstances and it was an offence not to comply.[87] The Chief Executive could give a notice to an owner of a dam being constructed to comply with a notice to have a dam failure impact assessed.[88] These examples show that the Regulator's functions in monitoring compliance are engaged once a dam satisfies the criteria for being referable.

7.59 The Chief Executive could apply safety conditions to a referable dam by notice to the owner when a development permit had not yet issued.[89] That the Regulator may impose safety conditions without a development permit supports the interpretation of the statute that seems preferable.

Director, Dam Safety

7.60 The Director, Dam Safety exercised some of the functions of the Regulator under delegation from the Chief Executive.[90] Mr Allen, then Director, Dam Safety attended a meeting with Andreas Neumaier, Mr Herweynen and others in February 2004.[91] The main topics discussed were the general arrangement of the Dam, its foundation, the Roller Compacted Concrete (**RCC**) mix, the 'DS [Downstream] Facing, Crest & Aprons' and the 'Dam Regulator's requirements'. Notes of that discussion record:

> ***Dam Regulator's Requirements***
>
> - *Main requirements are to document all assumptions made. The design report will provide details of these assumptions.*
> - *Design parameters*
> - *Design methodology*
> - *Also need to confirm assumptions and document on site during construction process. The construction report will be the document that provides these details.*
> - *All information that would be required to undertake a Safety Review of the dam should be documented.*
> - ***There may be Regulator audits during the construction process to check that procedures are in place to confirm the design parameters are met and that the procedures are being followed.***
> - *Burnett Water will also probably undertake there [sic] own audits.*
> - *Key items that the Regulator is interested in include:*
> - *Membrane*
> - ***Trial embankment and RCC placement process***

87 *Water Act 2000* (Reprint 2F) s 483(1).
88 *Water Act 2000* (Reprint 2F) s 483(2).
89 *Water Act 2000* (Reprint 2F) s 491(1) and (9).
90 The Commission requested a copy of the operative delegation but it could not be located.
91 **DNR.005.4886**, .4891 (emphasis added).

- ***Processes & procedures to confirm design parameters are being met.***

7.61 There is no statutory basis for those 'Dam Regulator's Requirements'. But their imposition accords with the Queensland Dam Safety Management Guidelines 2002 (**the Dam Safety Guidelines**).

Dam Safety Guidelines

7.62 The Dam Safety Guidelines were produced by the former Department of Natural Resources and Mines for use by owners, operators, employees and consultants in connection with referable dams. They were not, without more, binding on owner or operator. However, compliance with parts of those Guidelines was required by many conditions of the 2003 development permit.[92]

7.63 The Dam Safety Guidelines outlined a safety management program for 'referable dams'. It comprised policies, procedures and investigations to minimise the risk of dam failure and instructions on documentation of each procedure.[93]

7.64 The Dam Safety Guidelines relevantly provide:[94]

> ***4.4.1 Geological and geotechnical investigations***
> *These include geological and geotechnical assessments of the site and materials. They are generally carried out in stages ranging from broad scoping levels to more detailed investigations depending on the findings of each stage.* ***Each stage should be thoroughly planned to ensure that all matters, which may affect dam safety, are identified, investigated and appropriately resolved by the designer.***
>
> ***Investigations should not be limited to the dam site alone.*** *The geology, topography and the depth of water held in the storage area should be* ***considered****. This ensures that major leakages, slope instabilities and significant reservoir-induced seismic activities, which may jeopardise the safety of the dam, are considered in the design.*
>
> *All work undertaken in the geological and geotechnical investigation stage should be properly recorded and presented in a comprehensive report. This will enable the designer to define the extent of any further work required prior to finalising the design. Investigations are generally on going through the construction period as the foundations become fully exposed or the extent of any foundation work, such as grouting, is recognised. Consequently, investigative reports need to be updated and amended as construction proceeds. When construction is complete, a full and comprehensive report should be available as a reference for on-going surveillance of the dam and subsequent safety reviews.*

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92 Exhibit 28, **DNR.003.7173**. See development safety conditions: DS3(2), DS4(1), DS5(1), (3) and (4), DS6(1), DS7(1), DS8(1), DS9(1), DS10(3), DS11(1) and (2), DS12(2) and DS13(1).

93 Exhibit 17, **PTA.001.0001**, Attachment PA-18, .0007 (s 2).

94 Exhibit 17, **PTA.001.0001**, Attachment PA-18, .0015 - .0020 (emphasis added; footnotes omitted).

4.4.2 Hydrological investigations

A suite of hydrological investigations should be undertaken to develop dam safety data for the proposed dam. These hydrological investigations, which are independent of yield hydrology, involve:

- *developing an appropriate run-off model for the catchment*
- *calibrating this run-off model with historical flood data where possible*
- *assessing any operating limitations and criteria, which are to apply to spillway discharges*
- ***assessing the consequences of potential failure of the dam:***
  - ***particularly the population at risk - see NR&M Guidelines for Failure Impact Assessment of Water Dams***
  - ***for best practice purposes to determine other consequences of failure (eg economic and environmental costs) using the ANCOLD Guidelines on Assessment of the Consequences of a Dam Failure (May 2000) if appropriate***
  - ***determining the spillway design standard, spillway design flood and, if the spillway is a gated structure, determining any operating rules which are to be applied.***

All work (including documentation of mathematical models) undertaken in hydrological investigations should be properly documented and presented in a comprehensive report. ***This will enable the designer to finalise the design and will assist subsequent reviews of this aspect of the design.***

…

4.5.1 General

Factors which should be considered during the design of a dam, include:

…

2. Geotechnical information

- *material properties and availability*
- *foundation properties and treatment*
- *geological characteristics*
- *seismic loadings.*

3. Hydraulic aspects

- *type of spillway, means of flow control and energy dissipation*
- *hydrological characteristics*
- *hydraulic design and water loadings*
- *stream diversion requirements*
- *flood mitigation capacity.*

4. Stability

- *structural capacity of principle elements*

…

4.5.2 Specific Design Requirements

While the way in which these aspects are applied to a particular dam depends on its dam failure impact rating, size, importance, complexity and consequences of a dam failure, the key principles are:

- ***all dams structures should be designed to suit the loads to be applied to them in accordance with:***
  - ***ANCOLD guidelines***
  - ***relevant Australian Standards***
  - ***notices (compliance and information) issued from time to time by the chief executive***
  - ***generally accepted engineering practices***
- ***in particular, dams must be able to withstand seismic loadings, flood loadings, normal operating loadings, construction loadings, post construction loadings.***
- ***the regional and site geology must be understood and engineering geology models developed to form the basis for design***
- ***the foundations must be capable of supporting the dam structure and controlling seepage***
- ***the reservoir basin and rim must be sufficiently impermeable to prevent excessive losses of water (Any seepage must be controlled and instability must not occur at any stage of reservoir operation.)***
- ***construction materials must be identified to meet site and design requirements***
- ***the spillway size must be established on the basis of accepted engineering standards–ANCOLD Guidelines on Selection of Acceptable Flood Capacity for Dams, 2000 (Hydrological and meteorological information used in the design must be appropriate for the dam locality and dam use)***
- *the cut-off design must be established on the basis of the loadings, strength of the available materials and the need to control the seepage (For embankment dams, the designer must incorporate adequate lines of defence including properly designed drains and properly designed filters to ensure the long-term integrity of the seepage control system)*
- *the outlet works must meet the requirements for the reservoir operation and must have provisions for safe operation and maintenance*
- *provision must be made for the long-term monitoring of the structural performance of the dam and its components*
- *an appropriate dam safety management program must be developed and adhered to through the investigation, design and construction processes to ensure all matters are properly attended to and adequately recorded.*

Some of these factors may have a direct impact on dam safety, while others may have an indirect impact. ***The dam designer should be a registered professional engineer, highly experienced and with a good knowledge and understanding of dams. In some cases, dam owners may want to establish a review board of experts to provide guidance on the design of the dam. For large projects, dam owners may wish to engage a project design engineer who is assigned technical coordination responsibility for the dam during its design and construction.***

These factors influence the construction cost of a dam. The designer should develop a design, which meets accepted safety standards and the needs of the owner (including budget). *The designer should be aware of new technology and methods being adopted elsewhere, which may provide cost savings. Such savings should be critically evaluated in terms of possible long-term costs, which may occur should safety and operational problems be experienced with the dam. The more that is known about the site conditions and foundation materials the less conservative the design has to be, resulting in lower construction costs.*

The designer should establish specific onsite construction and operational inspection programs for review by appropriate design personnel and technical specialists. These programs should include frequent inspections during construction to confirm that site conditions conform to those assumed for design or to determine if design changes may be required to suit the actual conditions. A major requirement is inspection and approval by the dam designer of the dam foundation and foundation treatment before the placing of dam materials. *The final design inspection of the construction should include a complete review of the surveillance undertaken and testing of any operating equipment.*

The designer should determine surveillance requirements for the dam including:

- *inspections - operational design inspections should continue throughout the life of the project, in accordance with a formal inspection program covering all project features. The inspection program should meet the regulatory requirements specified in the dam safety conditions in the development permit*

…

4.6 Construction

The supervising constructing engineer(s) should be experienced in dams engineering and be able to detect when variations to specified procedures are necessary, or when special attention is required in relation to:

- ***foundation treatment***
- ***material selection and placement***
- *material manufacture (eg filters)*

- ***material testing***
- *stream diversion*
- *concrete manufacture*
- *construction equipment selection*
- *other issues which can affect the safety of the dam.*

The constructing engineer should have:

- ***a comprehensive understanding of the design***
- ***responsibility for technical coordination between design and construction engineers***
- ***responsibility for managing the construction staff to assure compliance with specifications.***

One of the most important aspects of dam construction is the foundation inspection. It is seldom possible to fully identify all the characteristics of the foundations of a dam during the investigation stage. Once the foundations have been fully exposed and prepared, there may be a need to amend the design requirements. Inspections by the designer are necessary to confirm any amendments. ***If unanticipated conditions such as geological features are encountered, the designer must be involved in determining appropriate design changes.***

Regular site visits and inspections by the designer and review engineers (where appropriate) are recommended.

7.65 Section 4.7 of the Dam Safety Guidelines relates to design and construction documentation. The aspects relating to the safety conditions imposed by the development permit are:[95]

4.7.2 Design Report

On most projects, a Design Report should be compiled once the design and construction stages are completed. However, on major projects, this may have to be staged. The designer should document the design and construction of the dam including:

- *Designer's Operating Criteria (DOC), eg gate operating rules and cone valve operation protocols*
- *design parameters adopted and assumptions made (and their bases)*
- *methods of analyses*
- *results of analyses and investigations (numerical and physical)*
- *hydraulic model testing of final spillway arrangements*
- *complete set of drawings and specifications*
- *summary of As-Constructed documentation and other construction information (see 4.7.3).*

95 Exhibit 17, **PTA.001.0001**, Attachment PA-18, .0020 (emphasis added).

The Design Report must contain sufficient information so that in the event of any safety problems relating to the dam, information can be quickly and easily obtained to resolve the problem.

When preparing a design report, the designer should consult the checklist of dam technology issues included as Appendix 3 - Checklist of Dam Technology Issues.

4.7.3 As-Constructed documentation

The constructing engineer should provide a complete record of the construction to assist in determining solutions to any safety problem, which may arise during the life of the dam*. As a minimum, this record should include:*

- *decisions to adapt the design to actual field conditions*
- *as-constructed drawings indicating the actual lines, levels and dimensions to which the structure is built*
- *construction processes*
- *systematically compiled and comprehensive photographs and, where appropriate, videos of the construction, with particular coverage of significant events which include:*
  - *foundation treatment*
  - *material preparation and placement*
  - *filters, cut-offs*
  - *core materials*
  - *joint preparation*
  - *foundation surface mapping of rock defects*
- ***material test results and comparison with assumed design parameters***
- *instrumentation data including precise instrument locations and initial instrument readings*
- *construction inspection reports.*

The As-Constructed documentation should be summarised and either incorporated into the Design Report or produced as a separate Construction Report.

7.66 Section 6 of the Dam Safety Guidelines relates to the surveillance measures to identify problems or unsafe conditions. Section 6.5.5 concerns regulatory audits:[96]

6.5.5 Regulatory audits

Purpose: *Independently, NR&M in its role as Regulator may audit dam safety management programs for referable dams in Queensland. These audits will generally examine compliance*

[96] Exhibit 17, **PTA.001.0001**, Attachment PA-18, .0034.

with development permit conditions dealing with dam safety and the outcomes of inspections and Safety Reviews. Such audits assist dam owners to compare their practices with industry standards.

Undertaken by: *NR&M*

Reporting: *The report may indicate:*

- *deficiencies in the dam safety management program and its documentation*
- *non-compliance with development permit conditions proposed actions by NR&M and the dam owner*
- *comments on the efficiency and the effectiveness of the dam safety management program.*

Discussion: *Generally the audit will be carried out on dams at random. Dams with a questionable management performance record are more likely to be audited. The outcome of these audits will assist NR&M to assess the effectiveness of their regulation program throughout the state.*

7.67 The Dam Safety Guidelines state that designers should consider foundation properties and treatment, type of spillway and energy dissipation, and structural capacity of principal elements (among others).[97] In s 4.5.2, it is noted that the spillway size must be established on the basis of accepted engineering standards based on the ANCOLD *Guidelines on Selection of Acceptable Flood Capacity for Dams*, 2000.

7.68 The designer should be an RPEQ and establish construction and operational inspection programs which include frequent inspections to confirm that site conditions conform with those assumed for the design.[98] The constructing engineer should have experience in dam engineering. That person should have the responsibility for managing the construction staff to assure compliance with specifications.[99]

7.69 A dam owner should compile and maintain a Data Book, which is a summary of all pertinent records and history, including design and construction records.[100]

97 Exhibit 17, **PTA.001.0001**, Attachment PA-18, .0016 (s 4.5).
98 Exhibit 17, **PTA.001.0001**, Attachment PA-18, .0033 (s 6.5.2).
99 Exhibit 17, **PTA.001.0001**, Attachment PA-18, .0018 (s 4.6).
100 Exhibit 17, **PTA.001.0001**, Attachment PA-18, .0019 (s 4.7.1).

Efficacy of regulatory governance

Dam safety conditions

7.70 On 30 October 2003, the Director-General of the then Department of Natural Resources and Mines, as 'assessment manager' under the *Integrated Planning Act,* issued decision notice IM1003BDB0003 and development permit 176904. This was the approval of the development application lodged by Burnett Water.[101] The conditions of the Development Permit included:

> *Condition DS 0 - General*
>
> *1. The dam is to be kept safe at all times.*
>
> *Condition DS 1 - Documentation*
>
> *1. Any documentation prepared in order to comply with these conditions must be stored securely until such time as the dam is decommissioned.*
>
> *2. The documentation must be made available for inspection by the Chief Executive, Department of Natural Resources and Mines, within 7 days of a written request for access being received by the dam owner.*
>
> *3. On change of ownership of the dam, all documentation prepared in compliance with these conditions must be transferred to the new owner.*
>
> *…*
>
> *Condition DS 3 - Design Report*
>
> *1. The Stage 1 Design Report for the dam has been taken to consist of:*
>
> - *'Burnett River Dam Alliance, Volume 2A: Respondent's Proposed Design'*
> - *'Burnett River Dam Alliance. Volume 2C: Respondent's Proposed Design: Drawings'*
> - *Burnett River Dam - NTAR No 30 (fax transmittal - response to additional information request).*
>
> *2. The dam owner must update the design report in accordance with this condition and the Queensland Dam Safety Management Guidelines - February 2002.*
>
> *3. The Design Report must be updated in the following stages and address the matters as outlined for each stage:-*

[101] Exhibit 28, **DNR.003.7173**.

Stage 2

1. *An update of the Stage 1 Report must be submitted to the Chief Executive, Department of Natural Resources and Mines prior to 'placement of RCC in the primary spillway area'.*

2. *In addition to the information provided in the Stage 1 report, the Stage 2 report must include, but not be limited to: -*
   - *Details of flood hydrology used in the final design.*
   - *A comprehensive Hazard Category Assessment. This assessment must indicate clearly the Hazard Category adopted, and the reasoning behind such a conclusion.*
   - *Results of any additional hydraulic model studies since the Stage 1 investigations.*
   - *Results of foundation and other investigations carried out since the investigation and preliminary design phase.*
   - *Foundation treatment in relation to seepage control including the grouting method and arrangement proposed including acceptance criteria.*
   - *Method of monitoring uplift in the foundation during grouting operations.*
   - *A clear statement of the reasoning for founding on any rock layer that may overly an alluvium layer. (eg the basalt layer on the right abutment). Results of stability and deformation calculations will also be required as part of the documentation.*
   - *Derivation of loads, load combinations, allowable stresses etc. used in the analyses, (including seismic loads).*
   - *Properties of construction materials.*
   - *Embankment design and stability analyses.*
   - *Design analyses of all structural components.*
   - *Design modifications necessary as a result of any information obtained since Stage 1.*
   - *Constraints on the operation of the dam, in particular, the operation of cone dispersion valves and radial gates in relation to the tailwater levels.*
   - *Details of any special requirements or provisions made for roller compacted concrete (eg waterproofing via membranes, erosion, and seepage protection).*
   - *Details of spillway capacities and assessment of the potential for cavitation.*
   - *Where it is considered the spillway will be subjected to cavitation under certain flow conditions, the owner will provide sufficient information to confirm the structure will not be at risk of significant damage or failure.*
   - *Details of outlets including discharge capacities.*
   - *Details of maximum flow rates through the outlet works and methodology used to manage the flow through the outlet works with*

consideration to cavitation, vibration and other potentially damaging influences.

- *Details of erosion protection (including downstream of the structure).*
- *A report by a qualified Operations and Maintenance Engineer as to the adequacy of the design from an ongoing operations viewpoint.*
- *Complete set of construction drawings and specifications.*

Stage 3

1. *An update of the Stage 2 Report for Final Design Report presentation must be prepared and a copy forwarded to the Chief Executive, Department of Natural Resources and Mines on or within three (3) months of 'practical completion of construction'.*

3. *In addition to the information provided in the Stage 2 report, the Final Design report must include, but not be limited to, the following: -*
   - *Results of foundation grouting and other investigations carried out during the construction phase.*
   - *Properties of construction materials used.*
   - *Results of strength tests on all 'in place' concrete products.*
   - *Design modifications necessary as a result of information obtained during the construction phase.*
   - *Any additional information gathered, affecting the design and construction of the dam, since creation of the Stage 2 report.*
   - *Compliance with construction specifications.*
   - *Final Instrumentation arrangement for the dam.*
   - *Finalisation and editorial changes.*
   - *Details of the as-constructed dam.*
   - *Complete and final set of 'as constructed' drawings and specifications.*
   - *Any other issues relevant to the project.*

Condition DS 4 - Design and Construction

1. *The dam is to be designed and constructed to comply with the Queensland Dam Safety Management Guidelines.*

2. *…*

3. *The dam must be constructed generally as per the drawings contained in 'Burnett River Dam Alliance, Volume 2C: Respondent's Proposed Design: Drawings' prepared by TEAM1 (SMEC, HYDRO TAS, WALTER, & MACMAHON). The principal drawings referred to include:*
   - *TEAM1-201 General Arrangement*
   - *TEAM1-205 Longitudinal Section*
   - *TEAM1-207 to 214 Typical Cross Sections*
   - *TEAM1-224, 227, 228 Primary Spillway Details*
   - *TEAM1-225 Secondary Spillway & Left Abutment Details*
   - *TEAM1-230, 231 Foundation Grouting*

- *TEAM1-234, 235 Spillway Wingwalls*
- *TEAM1-303 to 305 Outlet Works*
- *TEAM1-238, 239 Instrumentation*
- *Construction Specifications*

Condition DS 5 - Data Book

1. *The dam owner must prepare a Data Book in accordance with this condition and the Queensland Dam Safety Management Guidelines - February 2002.*

2. *…*

3. *The Data Book must include all information as is required in the Queensland Dam Safety Management Guidelines - February 2002 including:*
   - *All pertinent records and history relating to the dam.*
   - *Documentation of investigation, design, construction, operation, maintenance, surveillance, monitoring measurements and any remedial action taken during construction and subsequent operation of the dam.*
   - *Known deficiencies such as seepage, cracking.*

4. *…*

Condition DS 6 - As Constructed Documentation

1. *The dam owner must develop as constructed documentation in accordance with this condition and the Queensland Dam Safety Management Guidelines - February 2002.*

2. *The owner must provide one copy of the as constructed documentation to the Chief Executive, Department of Natural Resources and Mines, on or within three (3) calendar months of 'practical completion of construction'.*

3. *The as constructed documentation must include:*
   a) *A record of any decisions to adapt the nominated design to suit actual field conditions.*
   b) *As constructed drawings indicating the actual lines, levels and dimensions to which the structure was built.*
   c) *A description of the construction processes.*
   d) *Foundation surface mapping.*
   e) *Material test results.*
   f) *Systematically compiled and comprehensive photographs and videos of the construction, with particular coverage of significant events which include:-*
      - *foundation treatment*
      - *material preparation and placement*

- *cut-offs*
- *core material*
- *joint preparation*

g) *Initial instrumentation data.*

h) *Construction inspection reports.*

i) *Certification by a registered professional engineer under the Professional Engineers Act 2002 (RPEQ) that the works have been constructed in compliance with all appropriate engineering standards including signed statements from the dam designer that principal components of construction have been inspected and approved. Such components shall include:*

- *Dam foundation and foundation treatment.*
- *Test results of the concrete used in construction.*
- *Adequacy of any joints and waterstops in the concrete.*
- *Structural adequacy of all principal elements.*

…

Condition DS 9 - Special Inspections

1. *When directed by the Chief Executive, Department of Natural Resources and Mines, a Special Inspection must be carried out at the cost of the dam owner and a report must be prepared in accordance with the Queensland Dam Safety Management Guidelines - February 2002.*

2. *The dam owner must provide one copy of the Special Inspection Report to the Chief Executive, Department of Natural Resources and Mines within one (1) month of completion of inspection.*

   …

Condition DS 11 - Comprehensive Inspections

1. *The dam owner must carry out a comprehensive inspection of the dam in accordance with the Queensland Dam Safety Management Guidelines - February 2002, on 'practical completion of the dam' and on or before every fifth anniversary thereafter.*

2. *A Comprehensive Inspection Report detailing the findings of the comprehensive inspection in accordance with the Queensland Dam Safety Management Guidelines - February 2002 must be submitted to Chief Executive, Department of Natural Resources and Mines, within three (3) months after completion of the comprehensive inspection.*

…

7.71 Burnett Water had an obligation to comply with these conditions.[102] The Regulator had the function of monitoring compliance with them.[103] Under the *Water Act 2000*, the Chief Executive or Regulator could appoint authorised officers to conduct investigations and inspections to monitor and enforce compliance with the *Water Act 2000* or development conditions imposed under the *Integrated Planning Act*.[104] Proceedings could be brought by the Chief Executive (as assessment manager) for any contravention of those conditions.[105]

7.72 The Dam Safety Guidelines contemplated the Department having a role in regulating dams. This included auditing compliance with dam safety conditions imposed under the *Water Act 2000* and the *Integrated Planning Act*.[106]

7.73 The current Director, Dam Safety, Christopher Nielsen, said that the dam safety conditions were met in substance.[107] He does, however, consider there to have been deficiencies in satisfying conditions DS3 and DS6. These deficiencies relate only to reporting requirements.

7.74 The safety conditions imposed on Burnett Water required more than reporting. Burnett Water had *to comply* with the Dam Safety Guidelines in the Dam's design and construction.[108] There was no evidence that any monitoring or compliance activities were undertaken by the Regulator to check compliance by Burnett Water with the Dam Safety Guidelines during design and construction of the Dam (to the extent required by the dam safety conditions).

7.75 The deficiency in respect of DS3 was that there was no report regarding the adequacy of the design from an ongoing operations viewpoint. This non-compliance is not of concern to him, because, Mr Nielsen said, these days, no such report would be required unless the Regulator had concerns about the operation or maintenance of a dam.[109]

7.76 The deficiency in respect of DS6 was that there was no separate document called a 'construction report' summarising the construction records.[110]

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102 *Water Act 2000* (Reprint 2F) s 491(8): *Integrated Planning Act 1997* (Reprint 4N) s 3.1.5(3).
103 *Water Act 2000* (Reprint 2F) s 515(1)(c); Exhibit 320, **NIC.001.0001**, .0010 [41].
104 *Water Act 2000* (Reprint 2F) ss 739 and 740.
105 *Integrated Planning Act 1997* (Reprint 4N) ss 4.3.3 and 4.3.18(3)(a).
106 Exhibit 17, **PTA.001.0001**, Attachment PA-18, .0011, (s 3.2).
107 **DNR.021.0001**, .0013 [34]; Exhibit CN-8 to the statement of Christopher Nielsen, **NIC.001.0001**.
108 *Water Act 2000* (Reprint 2F) s 491(8); *Integrated Planning Act 1997* (Reprint 4N) s 3.1.5(3).
109 Exhibit 320, **NIC.001.0001**, .0012 [50].
110 Exhibit 320, **NIC.001.0001**, .0012 [50]; **DNR.021.0001**, .0013 [37].

The Construction report: Condition DS6(1)

7.77 Condition DS6(1) required Burnett Water to develop 'as-constructed' documentation in accordance with the Dam Safety Guidelines.[111] Section 4.7.3 of those Guidelines stated that the 'as-constructed' documentation should be summarised and either incorporated into the design report or produced as a separate report.

7.78 Burnett Water was, therefore, obliged to generate a summary report in an update to the Final Design Report or else to produce a separate construction report.

7.79 The Alliance prepared a Project Completion Plan for the Dam in August 2005.[112] Section 4 outlined the document management plan to be implemented for completion of the project. It required a 'detailed construction report' to be prepared. Section 4.1.1 of the Project Completion Plan provided:

> ***4.1.1 Detailed Construction Report***
>
> *The Construction Report is required to provide the dam owner with a detailed summary of the construction process for the dam. It shall also include a brief summary of any decisions to adapt the nominated design to suit actual field conditions.*
>
> *Along with the summary of the construction process, the Construction Report will include certification by a registered professional engineer that construction has complied with all appropriate engineering standards including signed statements from the dam designer that principle construction has been inspected and approved. These components include:*
>
> - *Dam foundation and foundation treatment*
> - *Test results of concrete used in construction*
> - *Adequacy of any joints and waterstop in the concrete*
> - *Structural adequacy of all principle elements*

7.80 In November 2005, Paul Rickert of the Alliance emailed David Ryan at the Department of Natural Resources and Mines and provided a list of the design, construction and quality assurance documentation for the Burnett River Dam Project.[113] One of the listed documents was a 'construction report' which included quality assurance documentation.[114] The 'construction report' section stated:

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111 Exhibit 28, **DNR.003.7173**, .7181.
112 **SUN.025.001.0001**.
113 **DNR.008.3325**.
114 **DNR.008.3325,** .3327.

	30.0 - Construction Report (Includes QA Documentation)	
1	Photos	
2	Design Change RFI's	- Volume 1
3	Design Change Register	- Volume 1
4	Historical Events File	
5	Foundation Inspection and Grouting Records Refer Section 30.08.05 for full foundation excavation detail. Refer Section 30.14A for Foundation Grouting Details.	Golders Pimple Report Dam Foundation Inspection Register
6	As Constructed Drawings - As Constructed Concreted Survey Data	Precast Panels
7	Instrumentation and Monitoring Data	Precommissioning Post Commissioning Initial Surveillance Report
	Non Conformance Reports	- Volume 1 Volume 2
8	RPEQ Sign Off and Certification	Copy of Offical Letter of Sign Off.

7.81 The quality assurance documentation was listed separately and included documents relating to, among other things, excavation of the dam foundation (5 volumes), basalt treatment (3 volumes), foundation grouting (9 volumes), RCC trial mix specification (1 volume), apron wall (2 volumes), RCC placement (total of 19 volumes) and Quality Control reports.[115]

7.82 The Department requested certain sections of the files:[116]

a. Introduction/Overview

b. Design Calculations Files

c. Design Report Stage 2

d. Design Report Stage 3 – Final Design Report (incl. outstanding files)

e. Construction Report (comprising files 30.1 to 30.8 inclusive)

f. Design and Construction Program.

7.83 The Alliance delivered the requested material to the Department in December 2005.[117]

7.84 The Commission sought production of the 'Construction Report' from SunWater,[118] Macmahon,[119] Hydro Tasmania,[120] and the Department of Natural Resources, Mines

115 **DNR.008.3325**, .3328-.3331.
116 **DNR.008.3325**, .3325.
117 Exhibit 320, **NIC.001.0001**, .0013 [56].
118 Requirement to Produce Documents to SunWater Limited dated 8 January 2020.
119 Requirement to Produce Documents to Macmahon Contractors Pty Ltd dated 11 February 2020.

and Energy.[121] No 'construction report' that summarises the as-constructed documents was produced.

7.85 The Department has not identified any 'construction report' in the voluminous material it received from the Alliance.

7.86 Mr Griggs prepared a construction report table of contents and identified it to the Commission.[122] He was not sure if anyone had actually prepared the report.[123] It seems not.

7.87 The evidence indicates that no summary report of the as-constructed documentation required by Condition DS6(1) and s 4.7.3 of the Dam Safety Guidelines was prepared.

7.88 Such a report can be useful to those who operate a dam in determining solutions to problems that arise.[124] A construction report should be seen as an essential part of good record keeping.

7.89 The absence of the summary report, however, has not prejudiced safety. Nor has the lack of it put the ongoing proper management of the Dam in jeopardy. The essential documentation reached the Department. It remains available to be used for the purposes that a construction report serves.

7.90 The material was also available to the Commission. Some was produced.

7.91 The absence of a construction report[125] did not impair the work of the Commission.

Role of the Regulator

7.92 The Regulator's statutory powers were limited. The Regulator, relevantly, could review, recommend, and monitor compliance with safety conditions imposed under the development permit. Mr Allen was the delegate of the Regulator's statutory powers and the Director, Dam Safety when the Dam was designed. In February 2004, he told the Alliance that he may undertake audits during construction *'to check that procedures are in place to confirm the design parameters are met and that the procedures are being followed'.*

7.93 The Regulator was empowered to appoint an authorised offer to conduct investigations and inspections to monitor and enforce compliance with the safety conditions imposed.[126] There was, however, no obligation on the Regulator to

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120 Requirement to Produce Documents to Hydro-Electric Corporation trading as Hydro Tasmania dated 11 February 2020.

121 Requirement to Produce Documents to Department of Natural Resources and Mines dated 11 February 2020.

122 **TRA.500.014.0001**, .0077 ln 36-37, .0079 ln 7 - 15; **MCM.012.0001**; **MCM.013.0001**.

123 **TRA.500.014.0001**, .0077 ln 41-45.

124 Exhibit 17, **PTA.001.0001**, Attachment PA-18, .0020 (s 4.7.3).

125 i.e. a summary report of the kind identified.

126 *Water Act 2000* (Reprint 2F) ss 739 and 740.

exercise that statutory power and no on site audit did take place during construction.[127]

7.94 According to Mr Nielsen, it has never been the practice to perform formal inspections or to conduct audits of the building of referable dams.[128] After construction, the Department does conduct site audits to address compliance with safety conditions.[129] The possible audit that Mr Allen foreshadowed related to procedures to confirm that the design parameters were met. The evidence did not explore what might have been done in carrying out such an audit. In these circumstances, there is no foundation for an inference that the kind of audit foreshadowed would have assisted in resolving the issues on which the Commission must focus.

RPEQ certification: Condition DS6(3)(i)

7.95 The safety conditions required '*certification by a registered professional engineer … that the works have been constructed in compliance with all appropriate engineering standards including signed statements from the dam designer that principal components of construction have been inspected and approved'.*

7.96 Mr Herweynen furnished two certifications as the RPEQ and Principal Dam Designer. The first, in a memorandum in October 2005 relating to approval for impoundment, certified *'that the works as constructed have been undertaken in a manner which meets the design requirements for the dam'.*[130] The second, by memorandum in November 2005 dealing with practical completion, stated: '*I can now give my certification that, subject to the replacement of [3 precast panels], the dam is at 'practical completion' and the dam can be safely filled to Full Supply Level*'.[131]

7.97 At first, Mr Herweynen was reluctant to accept that those memoranda were the sign off by the RPEQ required by Condition DS6(3)(i) of the Development Permit.[132] He testified that he was not sure if those were the certificates that the Regulator anticipated receiving but was nonetheless 'happy' to give those certifications.[133] Mr Nielsen considers them to have satisfied DS6(3)(i):[134] in his conception, certifying that the Dam is safe to fill is equivalent to the certification that DS6(3)(i) prescribes.[135] That is a generous interpretation of the memoranda.

7.98 Condition DS6(3)(i) of the Development Permit called for certification that the Dam had been 'constructed in compliance with all appropriate engineering standards ...'. The first certification stated that the works as constructed were 'undertaken in a

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127 **TRA.500.015.0001, .0059** ln 46 - .0060 ln 1, .0060. The Submissions of Department of Natural Resources and Mines dated 23 March 2020 accepted there was no record of a 'formal' audit: **DNR.021.0001**, .0008 [21]. The Commission did not locate any record of an audit.

128 Exhibit 320, **NIC.001.0001**, .0014 [64].

129 **TRA.500.015.0001**, 0060 ln 21-41; Exhibit 320, **NIC.001.0001**, 0013 [13]. An audit in the nature of a review of the reporting required under the safety conditions was undertaken in 2008.

130 Exhibit 280, **SUN.126.001.0001**, .0001.

131 Exhibit 306, **DNR.005.0584**, .0828.

132 Exhibit 247, **TRA.510.007.0001**, .0106 ln 15.

133 **TRA.500.013.0001**, .0057 ln 23-31.

134 Exhibit 320, **NIC.001.0001**, .0015 [68].

135 **TRA.500.015.0001**, .0067 ln 34.

manner' which met the design requirements for the Dam. The second said that the Dam was safe to fill to Full Supply Level (**FSL**). Neither fits the terms of the condition. The first falls short of certifying to the several matters stipulated in the condition. As to the second, a dam may be safe to fill to FSL yet not have been constructed in conformity with 'all appropriate engineering standards'. And neither certificate was accompanied by the necessary assurances about inspection and approval.

7.99 As it happens, however, the shortcomings in the certificates have no significance for the present uncertainty concerning the stability of the Dam. The language in which Mr Herweynen expressed himself has not contributed to the difficulties that attend the doubts about stability.

Independent expert panel

7.100 SunWater submitted that the project may have benefited from an independent technical review panel.[136] Such panels, according to SunWater, provide an important check on the labours of others. The idea is endorsed by ANCOLD.[137]

7.101 Mr Nielsen has been considering whether independent technical reviews for referable dams should be mandated, in the Dam Safety Guidelines or through the imposition of conditions.[138] He is reflecting on such issues as: who should comprise the panel; its structure and independence; how the panel would function; what responsibilities should be assigned to it; and how it would report.

7.102 An expert technical review panel, independent of the designers and builders, has advantages for Queensland dam projects.

7.103 Mark Hamilton, the Alliance Project Manager, spoke of governance at Wyaralong Dam. He was a member of its Project Alliance Board.[139] Wyaralong Dam was built soon after Paradise Dam. Project administration worked well there.[140] Governance was different at Wyaralong: for one thing, Wyaralong had a technical review panel. It was a good aspect of the arrangements – in Mr Hamilton's assessment, a '*maturing of the Paradise Dam process*'. It provided another level of governance – '*a check and balance*', in his view.[141] It added value and came at relatively low cost.[142]

7.104 Mr Herweynen believes that a technical advisory panel for the Dam would have assisted in making 'key decisions':[143]

> *… the concept of a client-engaged independent technical review panel that is from the very start of a project to the very end would have helped.*

136 **SUN.011.0001**, .0006 [34].
137 **SUN.011.0001**, .0007 [38].
138 Exhibit 320, **NIC.001.0001**, .0015, [70].
139 **TRA.500.015.0001**, .0027 ln 30, .0030 ln 8-9.
140 **TRA.500.015.0001**, .0029 ln 39.
141 **TRA.500.015.0001**, .0029 ln 42-46, .0030 ln 3-4.
142 **TRA.500.015.0001**, .0031 ln 33-38.
143 Exhibit 247, **TRA.510.007.0001**, .0107 ln 18-42.

> *Now, some of that you can say is a bit in retrospective, and some of it is based on projects I've done since Paradise Dam and that I have found it to be very useful having that panel that would challenge back some of these key decisions.*

7.105 A technical review panel that is expert and independent of designer and building contractor can be a valuable, affordable resource to promote better dam design and construction, especially where novel construction methods used are proposed. Such a panel can provide detached advice and scrutiny that is unlikely to be influenced by the time and commercial constraints under which designers and contractors usually operate.

7.106 Such a panel may have the incidental benefit of strengthening the overall regulatory regime.

Peer review

7.107 The Chapters on RCC (Chapters 4 and 5) and Downstream Protection (Chapter 6) describe shortcomings in the peer review of those elements of design. The topic is mentioned here because the quality of peer review matters to project governance.

7.108 Proper peer review processes can contribute to ensuring that future Queensland dam projects are designed, constructed and commissioned to acceptable standards and engineering good practice.[144]

Recommendations

7

The Regulator should consider suitable means of routinely monitoring compliance with conditions of development permits and other approvals relating to the construction of dams, including by audits and checks during construction.

8

To the extent practicable, the entity that is ultimately to own or operate the dam after its commissioning should have an opportunity to influence its design and construction; and if there is an alliance, preferably as part of that structure.

144 The Alliance did cause an Independent Performance Review to be undertaken in May 2004 to audit key performance areas in construction performance, cost performance, progress of the design and construction. The report of that review is no substitute for proper peer review; nor did it recognise that there had not been adequate peer review of the apron or the RCC.

Appendix 1 – Order in Council

Commissions of Inquiry Act 1950

Commissions of Inquiry Order (No. 1) 2019

Short title

1. This Order in Council may be cited as the Commissions of Inquiry Order (No. 1) 2019.

Commencement

2. This Order in Council commences on 6 December 2019.

Appointment of Commission

3. UNDER the provisions of the *Commissions of Inquiry Act 1950* the Governor in Council hereby appoints the Honourable John Harris Byrne AO RFC as Chairperson and Commissioner, and Emeritus Professor John Phillip Carter AM FAA FTSE FRSN FIEAust FAIB as Commissioner, from 6 December 2019, to make full and careful inquiry in an independent manner with respect to the following matters:

   a) the root cause of structural and stability issues identified in engineering and technical studies conducted on the Paradise Dam between 30 January 2013 and 30 November 2019;

   b) where the root cause is attributable, or attributable in part, to the design, construction and/or commissioning stages of the Paradise Dam, the facts and circumstances that contributed to the structural and stability issues having regard to:

      i. the governance arrangements in place including expert third party review and response to any issues raised;

      ii. the scope and effectiveness of processes and systems to ensure quality in design, construction and/or commissioning, adopted by individuals, entities and government bodies involved in the design, construction or commissioning of Paradise Dam; and individuals, entities and government bodies involved in giving the necessary approvals required for the Dam;

      iii. the reporting arrangements and obligations in place during design, construction and commissioning;

      iv. remedial measures taken during design, construction and commissioning;

      v. any other matter relevant to the Inquiry.

4. THE Commissioners may make any recommendations arising out of the evidence, considerations or findings of the inquiry in relation to the matters set out in paragraphs 3a) and b) above that the Commissioners consider appropriate to ensure future Queensland dam projects are designed, constructed and commissioned to acceptable standards, as defined in Queensland Government legislation and regulation, Australian National Committee on Large Dams guidelines and engineering good practice.

Commission to report

5. AND directs that the Commissioners make full and faithful report and recommendations which they consider appropriate on the aforesaid subject matter of inquiry, and transmit the same to the Honourable the Premier and Minister for Trade and the Honourable the Minister for Natural Resources, Mines and Energy by 30 April 2020.

Application of Act

6. THE provisions of the *Commissions of Inquiry Act 1950* shall be applicable for the purposes of this inquiry, except for section 19C (Authority to use listening devices).

Conduct of Inquiry

7. THE Commissioners in making their inquiry:

   a) may seek information from Sunwater; relevant individuals, entities and government bodies involved in the design, construction or commissioning of Paradise Dam; individuals, entities and government bodies involved in giving the necessary approvals required for the Dam; and any sources of expert advice considered appropriate by the Commissioners;

   b) may seek and consider public submissions in such a manner as may be necessary and convenient;

   c) may conduct public hearings in such a manner and in such locations as may be considered necessary and convenient by the Chairperson;

   d) may conduct interviews with any person who has information relevant to the terms of reference either with the person's consent or pursuant to a requirement under section 5 of the *Commissions of Inquiry Act 1950*.

Endnotes

1. Made by the Governor in Council on 5 December 2019.
2. Notified in the Gazette on 6 December 2019.
3. Not required to be laid before the Legislative Assembly.
4. The administering agency is the Department of Natural Resources, Mines and Energy.

Appendix 2 – Commission's establishment and operations

Establishment of the Commission

The Commission commenced on 6 December 2019.

Office premises were obtained on Level 23 of the State Law Building, 50 Ann Street, Brisbane.

The Commission established a website: www.paradisedaminquiry.qld.gov.au.

In December 2019 and January 2020, key personnel were recruited, the website, information technology and communications infrastructure were put in place and governance and procedural frameworks were established. A schedule was created to determine the extent of work required, supporting tasks and their interdependency to enable the Commission to complete its work on time and within budget.

The Commission engaged nine staff, in addition to four Counsel Assisting. The staff and their positions are listed in Appendix 3. They came from a variety of backgrounds, resulting in a diverse range of skills and expertise including management, legal, technical, procurement, policy, research, communications, and administration. Staff underwent a criminal history check, signed contracts of confidentiality, and were required to disclose any conflicts of interest.

Staffing numbers fluctuated and were reduced by 3 in the final month of the Commission's operations.

The Commission engaged third party vendors through robust procurement processes which considered efficiency, experience, cost and suitability of services.

The entities and the purpose of their engagement are as follows:

Entity	Purpose
Creative Curiosity Pty Ltd	Web hosting, publishing and creative services
Crown Law	Legal Resourcing
EPIQ Australia Pty Ltd	Online document management system
EPIQ Australia Pty Ltd	Recording and transcription services
Corrivium Pty Ltd	Livestreaming of the hearings
Law Image Services (Aust) Pty Ltd	Printing and copying services

Site visit

The Commissioners and others inspected Paradise Dam (**the Dam**) on 18 December 2019. Counsel Assisting visited the site on 12 February 2020.

Records Management

Records were managed in accordance with the following legislation and policy:

- *Public Records Act* 2002
- Commissions of Inquiry Retention and Disposal Schedule[1]
- General Retention and Disposal Schedule (GRDS)
- *Right to Information Act* 2009
- Records Governance Policy[2]

Administrative records were retained on a shared network system. An intranet utilising SharePoint was developed as the central accessible point of reference for all staff.

Evidence collection and management

The Commission relied on its powers under the *Commissions of Inquiry Act 1950* to seek information and documents from organisations and individuals.

A Document Management Protocol outlined the Commission's intention to receive all materials electronically. The Protocol explained how material was required to be collected, digitised and provided to the Commission. The Protocol enforced a Document ID system to uniquely identify every record received.

The Commission used this system to assist in determining whether and to what extent there had been compliance with notices to produce documents, and to ensure the lifecycle of each record was tracked.

The Commission collected and analysed more than 37,700 documents.

Submissions

Submissions were invited via the website.

The majority of submissions addressed the Inquiry's terms of reference or provided information related to the Dam. Submissions not deemed confidential were published on the website. Where submissions contained personal information, they were redacted. A list of submissions received appears at Appendix 6.

Publication and confidentiality

Effort was made to keep the public informed of the Commission's progress. This included publication of witness statements and statutory declarations including supporting documentation and witness curricula vitae, (again, redacted to protect privacy where justified). Practice Guidelines, Terms of Reference and hearing schedules were also published.

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1 QDAN 676 v.2 issued by the Queensland State Archivist under the *Public Records Act 2002.*

2 Queensland Government Enterprise Architecture, Queensland Government Chief Information Office, accessed on 12 December 2019 <https://www.qgcio.qld.gov.au/documents/records-governance-policy>.

Custodianship

At cessation of the Commission, permanent physical records were accepted by the Queensland State Archivist. Digital and hybrid records as well as temporary physical records were transitioned to the Department of Natural Resources, Mines and Energy, designated as the Responsible Public Authority by Queensland State Archives through regulation developed in accordance with the *Public Records Act 2002*.

Applications to access the Commission's records should be made to the Department of Natural Resources, Mines and Energy by writing to GPO Box 2454 Brisbane, Queensland 4001, or by email to rtiservices@des.qld.gov.au.

Hearings of the Commission

The Commission held public hearings in Brisbane and Bundaberg at the times and locations listed below. The Commission had planned its final hearing to take submissions in Bundaberg. In response to public health concerns and directives regarding the COVID-19 virus, the Commission conducted that hearing virtually, on 6 April 2020.

Dates	Venue	Location
20 February 2020	Court 4, Level 1 Brisbane Magistrates Court	363 George Street, Brisbane
2 – 6 March 2020	Supreme/District Court Room Bundaberg Courthouse	44 Quay Street, Bundaberg
9 – 13 March 2020	Court Room 4, Level 1 Brisbane Magistrates Court	363 George Street, Brisbane
16 – 19 March 2020	Court Room 4, Level 1 Brisbane Magistrates Court	363 George Street, Brisbane
6 April 2020	Online	Virtual

To facilitate access to the hearings, audio-visual services were utilised to live stream the proceedings.

Witnesses and Witness Statements

34 persons attended interviews or gave evidence in public hearings.

The Commission received a total of 23 witness statements and statutory declarations, most of which were received into evidence as exhibits. Selected persons statements and curricula vitae were published to the website[3]. Commissioner Carter attended (by telephone) some interviews of experts.

The Commission heard experts testify concurrently. This provided the experts with an opportunity to engage with each other in the witness box and to ensure that matters within their expertise were fully explored. This approach reduced the time taken to hear each witness

[3] <https://paradisedaminquiry.qld.gov.au/hearings/witnesses/witnesses-statements/>

separately, provided the opportunity for the experts to present and clarify their opinions, and to respond to matters raised by others.

On 10 March 2020, five expert witnesses gave evidence concurrently, three in person and two by telephone link from the United States of America. Their evidence concerned shear strength testing of roller compacted concrete and related matters. On 11 March 2020, two expert witnesses gave concurrent evidence, in person, about roller compacted concrete.

The Protocol and Agenda used for these sessions is Appendix 8.

A joint interview was conducted on 17 March 2020 by Senior Counsel Assisting of Mr Willey and Mr Griggs to explore matters of technical details relevant to the Dam's sliding stability. Commissioner Carter attended that interview, along with legal representatives of the interviewees.

Procedural Fairness

Where it was identified that adverse comment might be made in the final report about individuals or entities, including in ways that might affect reputation, notices were sent by Senior Counsel Assisting to the person or entity. Those notices afforded an opportunity to make submissions and to adduce further evidence as to why the possible findings should not be made, ought be made differently, or explained in some way. Submissions were received in response to those notices. The hearing on 6 April 2020 was an additional opportunity for parties to make submissions in response to the notices of potential adverse findings, and generally as to the Terms of Reference.

Technology

As the administering agency, the Department of Natural Resources, Mines and Energy facilitated the necessary access to information and communications technology.

EPIQ Systems Australia Pty Ltd (EPIQ) supported the electronic management of records and the electronic delivery of the hearings.

By arrangement with EPIQ, live streaming of the hearings was conducted by Corrivium Pty Ltd.

Communications and media

The Commission adopted a set of guiding principles for engagement. These principles helped inform media and communication actions including stakeholder liaison, advertising, event planning and media contact.

A website to disseminate information was released on 20 December 2019. The website was regularly updated to provide reliable and current information.

Website analytics

An overview of web user analytics[4] for the site from December 2019 to April 2020 is given below.

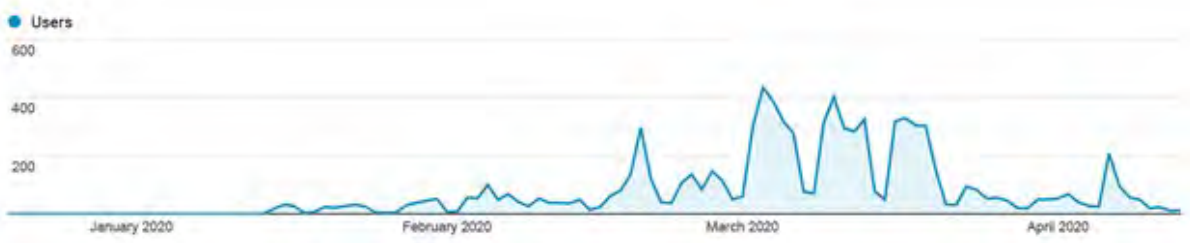

Figure A2.1 – Number of website users during January-April 2020

Figure A2.2 – Breakdown of user types during January-April 2020

Media engagement

The Commission's media strategy was to keep the community informed about its work. National and regional media were notified of the Inquiry's progress through direct contact, briefings and statements.

Media Guidelines to assist journalists were published on the website.[5] Press coverage achieved is highlighted below. Journalists at the Bundaberg News Mail, the ABC Wide Bay, and the Australian were active in their pursuit of stories.

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4 Google.com, Google Analytics, accessed 14 April 2020 <https://analytics.google.com/analytics/web/>.

5 <https://paradisedaminquiry.qld.gov.au/media/media-guidelines/>.

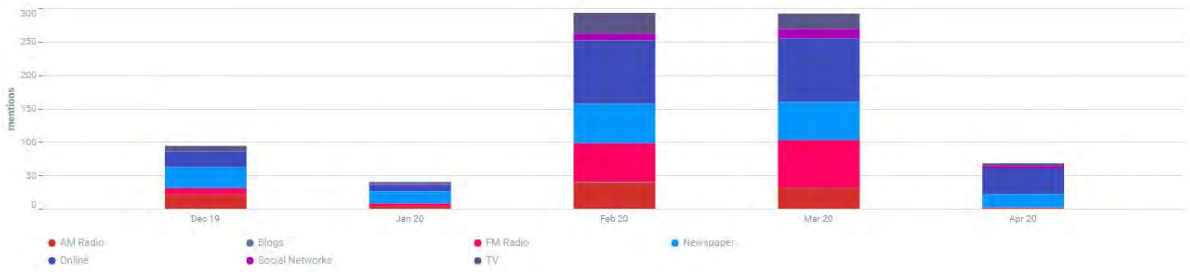

Figure A2.3 – An analysis of coverage by media type over time. Copyright 2020 Isentia.

A digital downloads platform was established for media to share audio-visual footage of the live streamed hearings.

Live streaming

Public hearings were live streamed through the website.

More than 3,800 unique viewers from 18 different countries watched the live stream.

Advertising

Community notice advertisements placed in local and national press promoted the website, hearing dates and invited anyone with information to give it to the Commission:

- *Regional*
  - Bundaberg News Mail
  - Maryborough Herald
  - Central North Burnett Times
  - Fraser Coast Chronicle
  - Gladstone Observer
  - Rockhampton Morning Bulletin
  - Sunshine Coast Daily
- *State-wide*
  - The Queensland Times
  - Courier Mail
- *National*
  - The Australian

Engagement

Seventy people registered an interest in receiving updates about the Inquiry's progress and opportunities to contribute. Brief updates were provided by email.

Counsel Assisting met with representatives of Bundaberg Canegrowers Limited and of Greensill Farming Group during their visit to Bundaberg before the commencement of public hearings.

Appendix 3 – Commission staff

Commissioners	
The Honourable John Byrne AO RFD	Chairperson and Commissioner
Emeritus Professor John Carter AM FAA FTSE FRSN FIEAust FAIB	Commissioner

Office of the Commission	
Suzanne Stone	Executive Director / Secretary
Rachel Scalongne	Executive Manager
Paul Brisbane	Executive Manager – *to 27 March 2020*
Johanna Clark	Media and Communications Manager
Monique Newman	Executive Officer
Tina Kloiber	Records Management Officer

Jonathan Horton QC	Senior Counsel Assisting the Commission
Jane Menzies	Counsel Assisting the Commission – *from 22 January 2020*
Samantha Amos	Counsel Assisting the Commission – *from 22 January 2020*
Alexander McKinnon	Counsel Assisting the Commission – *from 10 February 2020*
Sunny Munasinghe	Counsel Assisting the Commission – *to 17 January 2020*
Rachael Murray	Special Counsel
Thea Hadok-Quadrio	Special Counsel – *to 25 March 2020*
Salwa Marsh	Legal Editor – *from 30 March 2020*
Ally Hannon	Research Officer – *to 17 April 2020*

Appendix 4 – Parties with leave to appear

Granted leave to appear	Counsel	Instructing solicitors
Hydro-Electric Corporation trading as Hydro Tasmania	Declan Kelly QC Duncan Marckwald	Corrs Chambers Westgarth
SunWater Limited	Tom Sullivan QC	Allens
Burnett Water Pty Ltd	Tom Sullivan QC	Allens
Department of Natural Resources, Mines and Energy and any of its officers, past and present	Melanie Hindman QC Lee Clark	Crown Law
Department of State Development, Manufacturing, Infrastructure and Planning	Melanie Hindman QC Lee Clark	Crown Law
GHD Pty Ltd	Nicholas Andreatidis QC Sophie Gibson	GHD Pty Ltd
SMEC Australia Pty Ltd and employees Mr Andreas Neumaier, Jonathon Reid and Francisco Lopez	Christian Jennings	Clyde & Co
Macmahon Contractors Pty Ltd	Scott Seefeld	Johnson, Winter & Slattery
Golder Associates Pty Ltd	Rob Anderson QC	Golder Associates Pty Ltd
Dr Ernest Schrader	Self-represented	

Appendix 5 – Opening remarks by Chairperson and Counsel Assisting

Chairperson's opening remarks

The Honourable John Byrne AO RFD

By Order-in-Council dated 5 December 2019, Emeritus Professor John Carter AM and I have been appointed as the Commissioners to conduct an inquiry under the *Commissions of Inquiry Act 1950* into certain structural and stability issues relating to Paradise Dam. I also chair the Commission.

Paradise Dam was built between 2003 and 2005.

The Dam sustained some damage through flooding in late 2010 and early 2011.

In January 2013, heavy rainfall associated with ex-tropical Cyclone Oswald created a major flood event for the Dam.

The primary spillway was overtopped by about 8.6 m at its peak. The peak outflow was a 0.5% AEP event: that is, a 1 in 200 Annual Exceedance Probability flood event.

The flooding caused substantial damage to the Dam, mainly to its spillway apron. There was also significant downstream scouring of the river bed.

The considerable extent of the 2013 damage to the Dam and of erosion of rock immediately downstream from the apron had not been anticipated: the Dam had been designed to pass, safely, a 1 in 30,000 Annual Exceedance Probability flood event.

Investigations and studies since 2013 have raised questions about the Dam's safety.

Against that background, this Commission was established.

The Terms of Reference within the Order-in-Council direct us:

3. […] to make full and careful inquiry in an independent manner with respect to the following matters:

    a) the root cause of structural and stability issues identified in engineering and technical studies conducted on the Paradise Dam between 30 January 2013 and 30 November 2019;

    b) where the root cause is attributable, or attributable in part, to the design, construction and/or commissioning stages of the Paradise Dam, the facts and circumstances that contributed to the structural and stability issues having regard to:

i. the governance arrangements in place including expert third party review and response to any issues raised;

ii. the scope and effectiveness of processes and systems to ensure quality in design, construction and/or commissioning, adopted by individuals, entities and government bodies involved in the design, construction or commissioning of Paradise Dam; and individuals, entities and government bodies involved in giving the necessary approvals required for the Dam;

iii. the reporting arrangements and obligations in place during design, construction and commissioning;

iv. remedial measures taken during design, construction and commissioning;

v. any other matter relevant to the Inquiry.

4. [to] make any recommendations arising out of the evidence, considerations or findings of the inquiry in relation to the matters set out in paragraphs 3a) and b) above that the Commissioners consider appropriate to ensure future Queensland dam projects are designed, constructed and commissioned to acceptable standards, as defined in Queensland Government legislation and regulation, Australian National Committee on Large Dams guidelines and engineering good practice.

It is not for us to propose what should happen with the Dam. Others are considering what works ought to be done to improve the Dam to protect life, property and economic interests. This Commission must conduct its Inquiry within the Terms of Reference. So we shall examine the structural and stability issues identified in those "studies" referred to in the Terms of Reference to discover their "root cause" (or causes).

Those "structural and stability issues…" will soon be the subject of comment by Senior Counsel appointed to assist the Commission, Jonathan Horton QC. The identification of those issues is something upon which others may make submissions.

Since the Commission began its investigations, it has collected, collated and made progress in analysing more than 30,000 documents and photographs. Some documents are of considerable length. Many deal with complex geotechnical, hydrological and engineering matters. Counsel assisting the Commission have interviewed more than a dozen potential witnesses. Using its website as well as advertisements in national and regional newspapers, the Commission has sought to gather more information.

Professor Carter and I recognise that the Dam and its future are important to communities in the Wide Bay – Burnett region, especially for those who rely on the Bundaberg Irrigation Scheme for their livelihoods. We visited the Dam last December. Lawyers assisting the Commission have inspected it too. They have also spent time in Bundaberg, speaking with people interested in the Commission's work.

But much remains to be done; and quickly. The Commission is to complete its investigations and report to the Honourable the Premier and Minister for Trade and the Honourable the Minister for Natural Resources, Mines and Energy by 30 April this year.

The first witnesses will give their evidence in Bundaberg. The Commission returns to Brisbane for hearings over the following fortnight. We plan to return to Bundaberg in April for final submissions.

To facilitate public access to our work, the hearings will be live streamed through the Commission's website.

Finally, I encourage anyone with relevant information to send it to the Commission Secretary, Ms Suzanne Stone, preferably by 3 March 2020.

Senior Counsel Assisting's opening remarks

Jonathan Horton QC

Introduction

Commissioners, may I tender, first of all, the Commissions of Inquiry Order to which the Chairperson referred.[1]

The Paradise Dam (originally known simply as the Burnett River Dam) was built between 2003 and 2005. It lies about 20 km North West of Biggenden and 80km south west of Bundaberg. Its primary purpose was to provide supply to the Bundaberg Irrigation Scheme which in turn provides water used to produce sugar, fresh market tomatoes and many other fruits and vegetables.

The Dam creates a 45 kilometre long and narrow reservoir, with a surface area of about 3,000 hectares and a storage volume of some 300,000 megalitres. One of the important features of the Dam is its very large catchment: some 31,000 square kilometres. As a result of the large size of the catchment relative to the Dam's capacity, there are large inflows into it.

Paradise Dam is a gravity dam built from roller-compacted concrete (RCC). Gravity dams achieve stability by reason of their geometric shape and the mass and strength of the concrete within the wall and associated structures.

Those assisting you visited the Dam on 12 February 2020 and had the benefit of a detailed inspection of it and its immediate surrounds.

Concrete gravity dams can be constructed in two main ways: using conventional concrete and using RCC. The design of both is similar, but there are differences in construction methods, concrete mix design and the nature of appurtenant structures.

RCC is mixed on site and delivered by trucks or conveyor where it is placed in 30 centimetre layers by heavy machinery from one abutment to the next in a continuous front. To achieve a void-free matrix, RCC is consolidated using vibrating rollers. When hardened, RCC should have the same properties as conventional concrete.

To be placed and compacted effectively, RCC must be dry enough to support the weight of the construction equipment, but also have enough water to permit the hydration process that causes concrete to harden. The mix must be of a consistency that is workable enough to achieve the necessary compaction of the RCC and prevent undesirable segregation and voids.

You will hear, Commissioners, evidence about the nature of RCC from numerous engineers, many of whom are recognised as international experts in that field. There are several RCC dams in Australia, including Cotter Dam in the ACT and Wyaralong Dam near Beaudesert, but there are few examples only of them in this country before Paradise Dam was built. We understand that it was one of the first dams of this kind to be built in Australia, although the

[1] PDI.024.0001 – Commissions of Inquiry Order (No 1) 2019, dated 5 December 2019

RCC method had been used in Queensland as early as 1984. It was claimed at the time to be the largest volume RCC dam in Australia.

Paradise Dam was designed and built by the Burnett Dam Alliance. One of the participants in it was Burnett Water Pty Ltd, the owner of the Dam. That company had been registered by the Queensland Government for that main purpose. An employee of the State held all the shares in it when it was first incorporated.

The tender process

An invitation to submit a registration of interest (in effect a tender invitation) was issued by Burnett Water for the design and construction of the Dam in January 2003.[2] It was premised upon a special purpose vehicle model: organisations would come together for the purpose of designing, building and commissioning the Dam. That left the Dam in effect as a 'deliverable' to then be run by others after handover. This model may have been a factor that bore upon, perhaps contributed to, the issues that subsequently developed. For example: a lack of knowledge about what testing was done during construction; and difficulties afterwards with ready availability of the full documentation about the design, construction and commissioning of the Dam. We have, for example, attempted to locate a 'construction report' which ought to exist, but a full version of it has not yet been able to be found.

The special purpose vehicle brought other challenges: the Alliance, it has been suggested by some witnesses to whom we have spoken, may have lacked a real 'client' who tested what was being proposed from the point of view of person or body that would be an actual owner and operator of the dam. Other aspects of governance will be explored, namely:

1. the appropriateness of a so-called 'declaration' by the Alliance to reduce the amount of conventional concrete utilised in the construction of the Dam by 30%;

2. the extent and degree of reliance placed by the Alliance upon Dr Ernest Schrader, a world authority on RCC of the low cementitious kind. He, as will be seen, played a central role in the Dam's design and construction and made a number of recommendations and judgments in the course of those stages, some of which are controversial. He is also the author of a Lift Joint Quality Index (LJQI) which I will mention presently.

Three groups tendered to design and build the Dam. The successful tenderer was the group comprising Walter Construction Group, Macmahon Contractors Pty Ltd and SMEC Australia Pty Ltd and Hydro Tasmania (the Hydro-Electric Corporation of Tasmania). The insolvency of the Walter Construction Group during construction necessitated the adoption of its responsibilities by Macmahon Contractors. There were, because of this, two Alliance agreements: one in 2004 and one in 2005.

The Alliance and its approach to Dam design and construction

The successful alliance group had proposed building the Dam using an RCC mix with low-cementitious content. There would appear, as I have said, to be two 'schools' in RCC dam design: the high-cementitious and low cementitious approaches. Both types are internationally recognised. High cementitious RCC suspends aggregate in a matrix of cementitious material,

[2] ALL.155.008.0001 – Invitation to submit registration of interest, Burnett Water, January 2003.

which provides strength. A lean RCC mix is made of graded aggregate and cementitious material fills only the smallest voids. It is the interlocking of particles that gives a lean mix its strength.

The Alliance here said that the cement content could be reduced perhaps to as low as 60 or 63 kg per cubic metre for the 'mass' of the dam. This puts it firmly in the low-cementitious category. It was claimed by the head designer of the Dam at the time that Paradise Dam was the first lean mix dam in Australia.

One of the questions to be investigated is whether the consequences associated with this choice were adequately dealt with, including the need for sufficiently sound construction practices to seek to achieve the design intent of placing, as the specification said, '*the entire roller-compacted concrete mass with sufficient continuity so that it hardens and acts within each monolith as one block without discontinuous joints or potential planes of separation*'.[3]

Shear strength of lift joints is a property for RCC gravity dams likely to loom large in this Inquiry. The strength of those joints depends on cohesion between joints and the frictional resistance between them to sliding. The extent of bonding across a joint is in turn dependent upon the quality of the surface of the underlying RCC layer. A method used here to evaluate the quality of lift joints was a 'Lift Joint Quality Index'. It was created and used by Dr Ernest Schrader, who consulted on the project and seems to have been very influential in it. The Index is mentioned in the 2013 Australian National Committee on Large Dams (ANCOLD) Guidelines on Design Criteria for Concrete Gravity Dams. But it is only mentioned, not, as we read it, positively endorsed. Moreover, it only appears in that Guideline in 2013, not in the guideline that was current at the time the Dam was built.

There are questions about the extent to which it is appropriate to rely upon an index which is one man's creation (albeit derived, as he will say, from numerous projects around the world). There are also questions about the appropriateness of using subjective value judgments to make apparently quantitative assessments, which were then used to derive shear strength parameters.

In August 2005, members of the construction team estimated shear strength parameters of the Dam based on LJQI scores. The estimations were better than the design values.[4] Giving lift joints a numerical score (which was used to estimate shear strength parameters), may have conveyed an impression of certainty which does not withstand objective and dispassionate scrutiny.

As we understand it, there was, at the time the Dam was built and immediately after, no firmly quantitative evidence to support the shear strength of RCC lift joints for this particular Dam that were estimated using the LJQI scores. While coreholes were drilled in 2006, the samples were tested in compression and tension only. No shear strength testing was undertaken. We wish to look more closely at what occurred in late 2005 and early 2006 with respect to decisions about what could and should be the subject of shear testing, and any analysis or observations from that time about whether the coreholes showed bonded or unbonded joints.

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3 DNR.003.8385 – Burnett River Dam Specifications Civil Construction Part 2 of 2, at page .8466.

4 ALC.001.001.1874 – Sunwater due diligence – Schrader-Montalvo presentation, at page .1882.

We are taking this topic up with witnesses who might be able to assist in understanding this issue.

Those assisting you have seen quality control reports that suggest that problems were experienced in the laying of the RCC. We are investigating these: what was done in response to those expressed problems, including by way of remediation; what this meant for overall dam stability; and what were the decisions made during the course of design and construction about cement content, composition of the RCC mix generally, and the use, for example, of bedding mix between layers.

On 2 August 2004, for example, Dr Schrader suggested that the requirements for bedding could be 'relaxed' based upon a view about this dam achieving stability with friction values alone.[5]

The accuracy of that and related assumptions will be the subject of evidence. That memorandum is, on its face concerning, because it is directly contrary to the specifications for the Dam. Dr Schrader reduced the amount of bedding mix to be spread over the upstream face of a type I cold joint from 25% to 10% of the lift surface, and for a type II cold joint from 30% to 15%. He called this a '*new requirement*'. Questions remain about why the memoranda from Dr Schrader urged the decreased use of bedding mix in a way that seems to have contradicted the specifications with respect to cold lift joints. Those joints are ones which have been left too long so that the cement in the lower layer of RCC has already hardened, so that a good bond could not be formed without further treatment.

Further issues arise because the witnesses we have spoken to hold differing understandings whether this memorandum's command was acted on. We know that some 8,536 cubic metres of bedding mix was claimed to have been used in the Dam wall. That is reported in the August & September 2005 Quality Control Report at page 91.[6] There is no document of which we are aware at present which seeks to amend the specifications in the respect expressed by Dr Schrader as the 'new requirement', and none which records how this design change was considered, nor how the premise upon which it was expressed was tested (i.e. that:

> *…. the Burnett River dam essentially achieves stability with current friction values alone …*

It is not clear to us how, if at all, this assertion was verified in terms of checking that this was in fact consistent with the stability assessment, what was meant by the important word 'essentially', how the 'new requirement' (a relaxed requirement for bedding mix) was assessed by those with the relevant governance responsibilities in the project and what procedures existed to assess and communicate this important design change to those with responsibility for seeing the specifications given effect.

Construction of the Dam was completed in December 2005. At that stage it was formally named Paradise Dam and Burnett Water Pty Ltd became a wholly owned subsidiary of

5 DNR.011.1255 – Cold Joint Treatment & Criteria 20040802.
6 ALC.002.001.0750 – Burnett QC Report, August-September 2005.

SunWater Limited by the transfer to it of all the shares. Since that time, SunWater has been responsible for the management of the Dam.

Before the Dam was cleared to be filled with water, the Design Team Leader, or Principal Dam Designer, Mr Herweynen, certified on 5 October 2005 that the works as constructed had been undertaken in a manner which met the design requirements for the dam.[7] This included achieving '*the design intent for all aspects of the Dam, including RCC, foundation preparation, … apron and end wall …*'.

We will contend that this is most likely the signoff contemplated by Condition DS-6 of the development permit of 30 October 2003 which required '*certification by a registered professional engineer … that the works have been constructed in compliance with all appropriate engineering standards … including structural adequacy of all principal elements'*.

2011 and 2013 Flood Events and effect on the Dam

As you have mentioned, Chair, the Dam suffered substantial damage in January 2013, primarily to its primary spillway apron and by downstream scouring after the rain event associated with ex-Tropical Cyclone Oswald. A large scour hole some 15 metres deep developed immediately downstream of the apron below the primary spillway. The evidence suggests that, had the flood event been more prolonged, there was a real risk of undermining the wall and causing a failure of it through its overturning. Some understanding of the damage sustained can be gained from the photograph that appears in the SunWater briefing presentation on 6 November 2019.[8]

The 2013 rain event produced, as you have said, Chair, a significant flood event that overtopped the spillway by 8.65 metres at its peak. This meant the volume passing through the Dam was equivalent to it filling and emptying several times per day.

The dam was designed to pass a 1:30,000 Annual Precipitation Event (which is a 0.0033% likelihood of a flood event of that magnitude occurring in any one year). That does not mean that some rectification works might not be necessary after significant flood events. But perhaps the public is entitled to question why, for a dam built so recently, and for a flood well below what it was promised to withstand, its stability was threatened, and there was a need for such extensive repairs.

The design basis for the Dam was for a Probable Maximum Precipitation design flood with a peak discharge of 93,000 cubic metres per second (8 million megalitres per day) with a corresponding headwater (flood) level of 20.1 metres height above the primary spillway crest (which is the full supply level).

The Dam was designed so that flow passes over the primary spillway first, before the secondary spillway overtops, which was designed to commence at a 1 in 1,000 AEP flood event, with the left abutment overtopping at a 1 in 10,000 AEP flood event. The secondary spillway did not overtop in the January 2013 event. It has never overtopped, despite being designed to do so. The secondary spillway can be seen to the left of the photograph taken on

7 SUN.126.001.0001 – Design signoff prior to impoundment.
8 DNR.002.7814 – Paradise Dam Sunwater presentation, 6 November 2019 at page .7817.

10 November 2008[9] and at the far end of the photograph taken from the left abutment on 10 November 2008.[10] Its general arrangement is shown in the General Arrangement Plan topographical drawing.[11] The damage the Dam sustained in 2013 may be indicative of issues in the Dam's design and/or construction, at least with respect to the size and adequacy of the downstream protection given the scour hole that resulted from the flood event and the damage that the apron sustained.

One complicating factor is that the apron was damaged as a result of an earlier flood event in 2011 when much of South East Queensland was faced with higher-than-usual rainfall. An inspection of Paradise Dam was carried out in September 2012 by SunWater. It spilled for some considerable time after the 2011 event which may have restricted the ability to inspect the Dam and take corrective action.

Damage that had not been rectified by the time of the 2013 flood included repairing damage to the sill at the end of the apron. There was also exposed steel reinforcing in the dissipator apron, which had not been reinstated and backfilled with traditional concrete. In a review of damage safety management actions during the 2013 flood event, NSW Public Works said that the unrectified damage had no *'significant effect on the damages which occurred during the January to March 2013 flood event'.*[12] One source of this conclusion may be SunWater, which informed the Director-General of the Department of Energy and Water Supply on 3 May 2013 that '*The majority of the damage identified above is totally unrelated to the damage to the spillway dissipator in 2013 that has led to the current concerns*'.[13]

We expect that this conclusion will be controversial. In particular, whether the unrepaired damage that the sill sustained in 2011 made it more susceptible to destruction in 2013. One theory that has been suggested to us is that once the sill was destroyed, the hydraulic jump was no longer contained within the length of the apron, thus causing the scour damage that developed. Associated questions are whether because the apron itself had been abraded and the reinforcing exposed in 2011, that may have contributed to the loss of the apron and/or its end sill in 2013.

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9 SUN.024.001.0002 – Photograph of Paradise Dam dated 10 November 2008.

10 SUN.024.001.0003 – Photograph of Paradise Dam dated 10 November 2008, from angle of left abutment.

11 SUN.024.001.0001 – Paradise Dam, topographical drawing, General Arrangement Plan.

12 DNR.002.8498 – Paradise Dam – Review of Dam Safety Management Actions at page .8559 (page 11-1).

13 DNR.013.0571 – Letter from SunWater to Director-General, DEWS, 3 May 2013 at page .0572.

Corrective action

The work that was done in response to the 2013 flood event was, in summary:

1. urgent flood repairs (completed by December 2013) consisting of anchoring and repairing the dissipator slab on the left side, filling the scour hole downstream of the left abutment with mass concrete, shotcreting the upstream face of the left side of the scour hole and constructing a conventional concrete wall anchored to the upstream face of the left scour hole.[14] This cost some $35 million.

2. undertaking a dam safety review and comprehensive risk assessment. These reviews were supported by a Technical Review Panel (comprising different members than a later one which was convened) and highlighted a number of potential risks associated with the Dam in light of the performance of the Dam during the 2013 event.

   These actions led to what is known as the Paradise Dam Improvement Project, which was established in 2015.

3. In 2017, it was decided to strengthen the base of the primary spillway at monoliths D and K (at each end of the spillway) by constructing reinforced concrete buttresses at the downstream foot of those monoliths. Monolith D is to the left hand side of the dam looking downstream and monolith K is on the other side. Those works cost some $30 million.

The people of Bundaberg and of the North and South Burnett have taken a close interest in the Dam and what is to be done as a consequence of what is being learned about its structural integrity and stability as part of the Paradise Dam Improvement Project. The project has involved conducting further technical investigations that have identified and revised risks to Paradise Dam, including the structural and stability issues that are now the subject of this Inquiry, to which I will shortly turn. The issues that those investigations have identified are the basis upon which essential works to the Dam have been accelerated.

Current position

From September of last year, the water level in the Dam was drawn down to 42% of its capacity, by releasing some 105,000 megalitres over ten weeks finishing in early December. The Dam is to remain at or about 42% with releases being made from time to time to keep the Dam at that level, including to manage inflows following recent rainfall. There are various means to make releases, including two irrigation outlets. On 4 February of this year, Queensland Parliament decided, following a recommendation from technical experts, to reduce the height of the primary spillway crest by 5 metres as an interim measure. We understand that further testing will be done of the RCC lift joints during the lowering of the primary spillway crest.

Different views exist about the merits of decisions to proceed with immediate essential works. That is not the focus of this Inquiry; it is concerned with identifying root causes of structural and stability issues that have been identified with the Dam as constructed.

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[14] DNR.001.0005 – Letter from SunWater to Director-General, DEWS re Paradise Dam remedial works, 23 April 2013.

Part of the Paradise Dam Improvement Project is considering options for longer-term upgrades to the primary and secondary spillways, to be implemented between 2020 and 2026. Building Queensland has been engaged by SunWater to develop a detailed business case to assess the long-term solutions to the problems which have been experienced with the Dam. As we understand it, decisions are to be made by Building Queensland about what future works to the Dam will be recommended to the Queensland Government later this month. Mr Dewar has indicated this in a statement he has given to the Commission. I will tender that presently.[15]

No part of this Commission's Terms of Reference permit this Inquiry to agitate the merits of these improvement decisions. The focus of those assisting you, in accordance with the Terms of Reference, is on better understanding and investigating the structural and stability issues and their root cause or causes. It is true, however, that some of the recent decisions made about the dam do, to a limited extent, enlighten us about the structural and stability issues that the Dam is perceived to have.

Our work to date

As you indicated Chairman, this Inquiry was established late last year. In the 20 days or so between then and the commencement of the Christmas period, the Commission was staffed, and Counsel Assisting retained under the administrative oversight of Ms Suzanne Stone, the Commission's Executive Director and Secretary.

Work commenced identifying persons and organisations likely to have information relevant to the Terms of Reference. Documents were requested from various persons and entities, a document management system was established, and work continued interviewing witnesses and preparing for public hearings.

Some 17 interviews have been conducted to date, most of them about technical subject matter. Professor Carter participated in a small number of them, when they involved engineers, and as appropriate. Some interviews were conducted under the exercise of the powers in s 5 of the *Commissions of Inquiry Act*, and others were undertaken based wholly upon the willingness of the witness to assist without compulsion.

The Commission has received some 30,000 documents which have had to be read, categorised and understood. Some 17 Requirements were issued for the production of those documents. The process of reading and categorising those documents is to some extent ongoing, but we have identified issues that require further investigation and most of those documents have been read and categorised on the basis of their relevance and subject matter. Our review of the documents is well advanced.

I am pleased to say that those assisting you have interviewed more than half of the persons likely to give evidence to the Inquiry or who are considered material to our investigation. Further interviews are to take place tomorrow and next week. We will make available statements and records of interview to parties as appropriate as they are finalised. There are many statements which the Commission has taken which we are having finalised by witnesses and they are being made available. There is, necessarily, some delay between the statement

[15] DES.001.001.0001 – Statement by Stephen Dewar dated 20 January 2020.

being taken and it being checked and witnessed (or notarised if the witness is overseas) and being made available. Yesterday we made available statements and records of interview to parties.

The technical and engineering studies

The Terms of Reference, as you have said, Chairperson, direct the Commission to the structural and stability issues identified in engineering and technical studies conducted on Paradise Dam between 30 January 2013 and 30 November 2019. The studies of which we are presently aware which fall into that description are as follows:

a. Tranche 1 Technical Review Panel Reports;

   a. No 1 dated October 2013[16]

   b. No 2 dated January 2014[17]

   c. No 3 dated November 2014[18]

   d. No 4 dated 15 December 2015[19]

b. Reports of the 'Tranche 2' Technical Review Panel. Numbers 1 and 2 of them are dated 29 May 2019[20] and 23 September 2019[21] respectively.

   (There is also a third report dated 9 December 2019 which is outside the date range given by paragraph 3(a) of the Terms of Reference, but which, you will hear, confirms the views expressed in TRP Report No 2.[22]

c. A Report from Tatro Hinds 'Shear Strength Evaluation Comments' dated 25 November 2019.[23]

d. Two memoranda from GHD (Mr James Willey) dated 5 September 2019[24] and 25 November 2019[25]

e. An Inspection Report of the Dam Safety Regulator in April 2013.[26]

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16 IGE.017.0001 – Report 1 of Technical Review Panel dated 1 October 2013.
17 IGE.018.0001 – Report 2 of Technical Review Panel dated January 2014.
18 IGE.019.0001 – Report 3 of Technical Review Panel dated November 2014.
19 IGE.020.0001 – Report 4 of Technical Review Panel dated 15 December 2015.
20 SUN.009.003.0613 – Tranche 2 Technical Review Panel – Report 1 dated 29 May 2019.
21 IGE.051.0001 – Tranche 2 Technical Review Panel – Report 2 dated 23 September 2019.
22 SUN.009.002.0001 – Paradise Dam Safety Review Project, Technical Review Panel Report No. 3 dated 9 December 2019.
23 TAT.001.0001 – TatroHinds Report, Shear Strength Evaluation Comments, dated 25 November 2019.
24 DNR.001.2363 – Memorandum of James Willey, GHD dated 5 September 2019.
25 GHD.005.0001 – Memorandum of James Willey, GHD dated 25 November 2019.
26 DNR.012.9331 – Dam Safety Regulator Report – April 2013.

f. SunWater's Dam Safety Review, Revised Report, in 2016.[27]

Those assisting you have obtained statements from authors of those reports. We propose to call most of them and tender the statement from each as they are called. Messrs Francisco Lopez and John Reid (both TRP members) have provided statements, but they are persons who we presently do not intend to call, unless parties indicate that they have further matters they wish to be put to them.

It is important to make particular mention of the recent work (which is ongoing) of GHD.

SunWater commissioned GHD to develop a preliminary design for improvement works to the Dam. As part of that work, GHD prepared two relevant memoranda mentioned above and a draft Preliminary Design Report for the Paradise Improvement Project dated July 2019. In the first of the memoranda, GHD concluded that there were likely to be widespread zones of debonding and segregation on the lift joints of the RCC with which the Dam was constructed. Section 6 of the memorandum states that investigations since the Dam was built indicate that 60 to 90% of lift joints are unbonded. Accordingly, GHD assessed the stability of the Dam using lower lift joint shear strength and cohesion parameters than those adopted during the design.

The results of GHD's stability assessment were published in the second memorandum. The stability assessment suggests that Paradise Dam's sliding factors of safety are below the relevant guidelines published by ANCOLD. In section 6 of that memorandum, GHD concluded that the main reason for the poor assessment of stability was the significantly lower shear strength of the lift joints than had been assumed in the design.

It should be emphasised that the Preliminary Design Report is a draft only. Mr James Willey in his statement will, as we understand it, explain its status and say that it ought be treated with caution in certain respects because it has been superseded in part. It is a working document, and we are conscious, as with the memoranda, that GHD's work which was in progress has, by reason of the circumstances of this Commission and our reporting deadline, been somewhat accelerated.

In 2019, SunWater established the Technical Review Panel to review GHD's preliminary design options. (There was an earlier one and I will seek to tender the reports from it as well shortly). The focus, however, is on the work of the later tranche TRP for present purposes.

Section 3.2 of TRP Report No 2 expresses the view that the condition of the Dam is the result of poor construction practices. The TRP said that problems with cold joints were not rectified notwithstanding evidence that issues with poor bonding and segregation had been identified early in the project. At page 13 of TRP Report No 2, the TRP expresses the following opinion:[28]

> *A review of construction photographs made available to the TRP during meeting #2 suggests that the organization of equipment, personnel and RCC placing operations had the potential of contributing to the poor lift surface bond strengths. ... There was evidence that multiple layers had been discontinued at several different locations which created*

27 DNR.002.3132 – Dam Safety Review Revised Report, Sunwater, 2016.

28 IGE.051.0001 – Technical Review Panel Report number 2, 23 September 2019, at page .0013.

multiple shoulders susceptible to poor compaction and the potential for "rock pockets" (uncompacted RCC) to be covered over and not properly prepared to receive the next lift. There were obvious large expanses of dried out surfaces with little or often no water curing visible. There were also areas where the old lift surfaces were being contaminated by traffic, dust and debris. Whether or not these conditions were mitigated before new RCC was placed on those surfaces cannot be ascertained from photos but suggests the possibility of those areas not having been properly prepared.

It has been difficult to find evidence of bedding mix on cold joints in recent and previous drilled RCC cores. Additionally, horizontally drilled cores extracted in the plane of horizontal lifts observed during a laboratory visit on 27 August 2019 exhibited dust and/or mud on the RCC joints suggesting little or insufficient surface preparation.

The views expressed by the TRP in this passage appear to have been based on two bundles of construction memoranda that SunWater had provided it. Shortly I will tender that material so that it is clear the basis upon which the TRP view was expressed. Those documents are: Paradise 2004 select memos RC 20191112,[29] and Paradise 2005 select memos RC 2019111.[30]

We hope to call witnesses involved in the quality control process, especially Mr Roberto Montalvo and Mr Jose Lopez. The evidence will show those men to have been engaged based upon their reputation and experience in the proper practices for the laying of RCC and that one of them was always on site for that purpose whenever RCC was being laid.

A considerable body of work has been done by technical experts and engineers in considering the structural issues with the Dam. I have attempted to focus on those which we assisting you consider to be the most important of them. If there are other studies of which the Commission ought be aware for the purpose of defining the issues which arise under paragraph 3(a) of the Terms of Reference, then I would invite parties or interested persons to identify those studies immediately to the Commission's Secretary, so that consideration can be given to whether they are ones engaged by the Terms of Reference.

Structural and stability issues

The following structural and stability issues which those assisting you have drawn from the technical and engineering studies I have just mentioned are as follows:

1. GHD's assessment of the sliding stability of the Dam does not meet the ANCOLD factors of safety for different flood scenarios. This issue is identified in the GHD memorandum of 25 September 2019.[31]

2. The sufficiency of the apron as designed and constructed. These issues are identified in the first TRP report of October 2013,[32] and an Inspection Report by the Dam Safety

29 SUN.009.002.0147 – Selected memoranda 2004.
30 SUN.009.002.0203 – Selected memoranda 2005.
31 GHD.005.0001 – Memorandum dated 25 September 2019.
32 IGE.017.0001 – Technical Review Panel Report, October 2013.

Regulator in April 2013,[33] and SunWater's Dam Safety Review, Revised Report in 2016.[34]

3. The sufficiency of geological studies and investigations and the reliance on those matters by the design team. The first TRP report of October 2013 highlighted concerns about the foundation mapping during construction.[35] Exploratory boreholes taken in 2019 identified open contact between the RCC and bedrock at the foundation of some parts of the Dam. This issue is identified in section 2.4 of the second report of the present TRP.[36]

In seeking to determine the root cause of these issues, we are currently pursuing the following lines of inquiry:

1. In terms of RCC:

   The adequacy of the bond between lift joints, which in turn raises questions about the adequacy of the mix, of the construction practices adopted, of decisions made in the course of construction about the use of bedding mix, and about what testing and checking was done to verify that the lift joints were of a quality that produced a sufficiently stable dam wall.

   Also relevant is the use of bedding mix on lift joints: 1) on the upstream side of each and every joint, as per the specifications, 0.5m wide; 2) on 'cold joints' as defined at section 11.10 of those Specifications;[37] and 3) if other requirements of the specification were not met, such as keeping the joint cured and preventing damage [to] the surface by equipment or rainfall.

   The memorandum from Dr Schrader of 2 August 2004 that I have mentioned and especially the 'new requirement' of which it speaks are contrary to the requirements in the specification for cold joint treatment.

33 DNR.012.9331 – Dam Safety Regulator Inspection Report, April 2013.
34 DNR.002.3132 – Paradise Dam Safety Review Revised Final Report, April 2016.
35 IGE.017.0001 – Technical Review Panel Report No. 1, October 2013, at page .0014.
36 IGE.051.0001 – Technical Review Panel Report No. 2, 23 September 2019.
37 DNR.003.8385 – Burnett River Dam Specifications, Civil Construction Part 2 of 2 at page .8469.

2. In terms of the apron:

   The main issue is adequacy of downstream protection (principally the size and quality of construction of the apron). There are questions about the accuracy and adequacy of the hydraulic modelling, including whether too great a reliance was placed on the energy dissipation effects that tailwater would offer, and whether the complexity of anticipated flood flows had been accounted for in the apron's design. Additionally, there is a question as to whether the apron was constructed of sufficiently strong material to withstand the erosive forces.

   The appropriateness and sufficiency of geological investigations prior to and during construction has been raised as a possible issue.

   We are still investigating how it could be that the apron suffered such extensive damage from flood events that the Dam was designed to pass, and how it could be that such a large scour hole developed during the 2013 flood which threatened the Dam's stability. These are matters we are still investigating and they involve technical questions of hydraulics, hydrology, geology and geotechnology.

3. In terms of the Dam's foundation: principally, the issue is the adequacy of contact between the bottom of the Dam wall and the rock beneath it given the particular geology of the site. On the material presently available to the Commission, this issue is one which the experts say is not necessarily a problem. It is an issue they still need to satisfy themselves about.

4. In terms of governance and reporting arrangements:

   a. whether the special purpose vehicle (here, Burnett Water Pty Ltd) had weaknesses in terms of separating the design and build from the ultimate owner and operator of the Dam;

   b. Whether the 'declaration' I have mentioned to use less conventional concrete was something that was desirable and a product of the Alliance-structure. We remain uncertain of its origins;

   c. whether that structure, and the use of an Independent Review Panel for Paradise Dam could have improved governance and provided a wider lens across design and construction activities, including oversight of what may have been excessive reliance upon one advisor, namely Dr Schrader;

   d. whether the Dam Safety Regulator adequately discharged his statutory functions, including in properly conditioning the development permits for the dam and ensuring those conditions were met.

There is, within each of these, sub issues which those assisting you are investigating. Practice Guideline No 2 paragraph 5 foreshadows that Counsel Assisting will provide a document setting out the key issues on which the Commission intends to focus during the course of the hearings on or before 24 February. That document is being prepared and will be made available. It is likely to follow the general scheme just outlined.

If there are additional issues which interested persons or parties consider ought be inquired into by this Commission, I would urge those persons to advise the Commission's Secretary immediately.

Root causes being pursued

The investigation of these matters by those assisting you is directed to finding their root cause. For that purpose, we are looking, in particular, to:

1. the tendering process for the selection of an alliance;

2. geological and geotechnical investigations undertaken before construction and as part of the Dam's design;

3. what the successful tender Alliance promised to deliver;

4. the governance arrangements that applied to the Dam's design, construction and commissioning, including the involvement of Dr Schrader;

5. the processes and systems directed to ensuring quality (including the application for, grant of, and the imposition of conditions upon the development permit granted for the dam on 30 October 2003 (and later amended on 30 June 2004 and 6 October 2005);

6. the reporting arrangements that were in place during, in particular, the design and construction phases of the Dam with respect to RCC, including quality control monitoring, inspection and test plans, LJQI checksheets and non-conformance reports. Of particular interest in this regard are the documents that were issued while the Dam was under construction;

7. what was done in the course of the Dam's construction to remediate problems as they arose. Documents created during the Dam's construction raise problems with construction of the RCC layers. We are investigating whether and to what extent the problems that were identified were fixed; and

8. governance processes. There may be three aspects to this:

    a. how the special purpose vehicle was required to report to its ultimate client, the Queensland Government;

    b. the risk allocation between Alliance members under the two Alliance Agreements;

    c. the role of the Dam Safety Regulator within the Department of Natural Resources (then called Department of Natural Resources and Mines, now Department of Natural Resources, Mines and Energy) with respect to governance (including to the extent to which the development application was considered and conditions imposed on it) and, more broadly, taking such steps not only to secure the essential documentation for the Dam (including testing and construction information) but also such information as could later be relied on including by those who now find themselves questioning the stability of the Dam to put their concerns to rest.

In a more general sense, we are investigating the causes of the unsatisfactory situation where the stability of the Dam remains uncertain so soon after it was built. Engineering opinion appears to be divided on that question.

We will pursue, and have pursued, this issue with witnesses. What means are and were available to give reasonable satisfaction that the Dam is stable? What could have been done differently to give that satisfaction when the dam was being designed, constructed and commissioned?

Would different forms of testing have offered that satisfaction? What forms of testing were available for low-cementitious content RCC? If the forms and type of testing were limited, was it made known at the outset that the use of low-cement RCC presented this limitation? To what extent was it represented that the quality of the construction methodologies and practices might be decisive of an assessment of the Dam's stability? We do not know the answers to these questions, but give early notice of our wish to investigate them further.

Statements

I will seek to tender shortly some statements whose authors I do not intend to call:

1. A Statutory Declaration of Mr Peter Allen given 28 January 2020.[38] He was the Director Dam Safety within the Department of Natural Resources and Mines at the time the Dam received Development Approval. It seems unlikely that Mr Allen can be available to give evidence for health reasons.

2. Mr Stephen Dewar, Program Director at Building Queensland, statement dated 20 January 2020.[39]

Tender of documents

May I now tender those documents to which I have referred and others that are key to the Inquiry at this stage:

1. A copy of the Alliance Agreements for the Burnett River Dam, dated 27 February 2004[40] and 23 May 2005.[41] The latter became necessary because of the insolvency of Walter Construction Group, one of the Alliance members.

2. Specifications: generally and RCC

    a. Specifications Part 1[42]

    b. Specifications Civil Construction Part 2 of 2 (Contains the RCC Specs)[43]

38 PTA.001.0001 – Statutory declaration of Peter Allen dated 28 January 2020.
39 DES.001.001.0001 – Statement of Stephen Dewar dated 20 January 2020.
40 SUN.009.002.0020 – Burnett River Dam Alliance Agreement – 27 February 2004.
41 ALL.144.002.0389 – Alliance Agreement Burnett River Dam dated 23 May 2005.
42 DNR.004.4559 – Civil Construction Specifications, Part 1 of 2, April 2004.
43 DNR.003.8385 – Civil Construction Specifications, Part 2 of 2, April 2004.

c. Method Statement for RCC & Trial Mix Specifications – Undated[44]

d. Roller Compacted Concrete Specifications - Undated[45]

3. The Detail Design Report dated June 2004.[46]

4. The Final Detail Design Report dated November 2005.[47] It deals only with the changes that occurred in construction, so it is considerably shorter than the 2004 Detail Design Report.

5. A Report of the Inspector-General of Emergency Management dated 19 December 2019.[48] That report is not within the date range of the technical and engineering studies referred to in the Terms of Reference, and nor is it, in substance, a technical and engineering study. But it is nevertheless an important report that is recent in its analysis of matters affecting the Dam. Also the SunWater submission to that process.[49]

6. The Development Permits that applied to the Dam:

   a. Dated 30 October 2003.[50]

   b. Amended Dated 3 June 2004.[51]

   c. Amended Dated 6 October 2005.[52]

7. What seems likely to be the RPEQ sign off mentioned in Condition DS-6 of the Development Permit of 30 October 2003.[53]

8. The material provided to the 2019 TRP before its second meeting about the laying of the RCC (mentioned on page 13 of that report), being:

   Paradise 2004 select memos RC 20191112[54]
   Paradise 2005 select memos RC 20191112.[55]

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44 DNR.010.8266 – Method statement for RCC & trial mix specifications.
45 ALC.002.001.1176 – Roller compacted concrete specifications.
46 GHD.002.0001 – Paradise Dam Detail Design Report dated June 2004.
47 DNR.001.0267 – Final Detail Design Report dated November 2005.
48 IGE.084.0001 – Inspector-General Emergency Management Report dated 19 December 2019.
49 IGE.076.0001 – SunWater submission to Inspector-General Emergency Management dated 28 October 2019.
50 DNR.003.7173 – DNRM Development permit application decision notice dated 30 October 2003.
51 DNR.003.7159 – DNRM Development details notice 176904 dated 3 June 2004.
52 DNR.003.7192 – DNRM Development details notice 176904 dated 6 October 2005.
53 SUN.126.001.0001.
54 SUN.009.002.0147 – Selected memoranda 2004.
55 SUN.009.002.0203 – Selected memoranda 2005.

9. The Review of Dam Safety Management Actions, Report for the Office of Water Regulation by NSW Public Works NSW Water Solutions dated 22 August 2013.[56] This report is a review of past actions (especially those taken immediately before and as a result of the 2013 floods). It informs a consideration of the structural and stability issues but at present we have not included it as a source itself of structural and stability issues.

Proposed schedule of witnesses

May I now indicate the proposed schedule for witnesses over the coming weeks:

1. First, witnesses who give evidence about the structural and stability issues that have been identified in the engineering and technical studies. These are the GHD engineers and members of the Technical Review Panel.

   We hope to focus first on two generalist (i.e. dam design and safety) witnesses: Mr Willey (GHD) and Mr Foster (TRP).

   Then we will move to evidence about the geology of the site and geotechnical considerations: Mr Young (TRP) and Mr Marley (from Golder Associates). Golder Associates was retained to advise on the geology of the site and to supervise the laying of the dam's foundations.

   Then we will call Mr Jonathan Williams, also from GHD and whose expertise is in the shear testing that was undertaken and which is brought into question by, among others, Mr Tatro. Mr Tatro says that aspects of the testing are, in effect, an insufficient basis to make the stability assessment which GHD seeks to do.

   This is expected to conclude the witnesses to be called in Bundaberg.

   Then we will call the remainder of this category of witness: those from the TRP and GHD who are knowledgeable about RCC and the testing which is used to determine what shear strength lift joints in the RCC have. There is the potential to be a difference of opinion between the experts on that topic and we expect you may hear some detailed technical evidence on that point.

   We will also call some technical witnesses who are knowledgeable about scour and hydraulics. These witnesses go to the question why the Dam's apron (sometimes called the dissipator apron) does not seem to be big enough and, perhaps, strong enough to withstand the erosive force of water coming over the spillway and to protect the wall from the risk of being undermined by the creation of the deep hole that was created by the 2013 floods.

2. After doing so, we propose to call witnesses who were involved in the design, construction and commissioning the Dam and in the governance arrangements that applied to it:

   a. Dr Ernest Schrader (RCC specialist consultant), a world expert in RCC and low-cementitious mixes, and an advisor to the Alliance, who wrote several important

[56] DNR.002.8498 – Paradise Dam – Review of Dam Safety Management Actions.

memoranda and gave advice about the RCC, its composition and its placement. He is also the author of a 'Lift Joint Quality Index' that was used for the dam to assess the adequacy of the RCC lift joints;

b. Mr Richard Herweynen, the lead designer of the Dam;

c. Mr Tim Griggs, who worked under Mr Herweynen;

d. Mr Andreas Neumaier (Design Manager for the Alliance and a SMEC engineer). Mr Herweynen reported to him;

e. Mr Bruce Embery (Construction Manager who worked for Macmahons Contractors); and

f. Mr Mark Hamilton (Alliance Project Manager, Burnett Dam Alliance 'Builder').

There will be others in this category who will be advised as the investigation progresses. We have attempted to make contact with Mr Jose Lopez (Senior RCC Engineer, Burnett Dam Alliance) and Mr Roberto Montalvo, RCC Manager. Both live overseas. To date, we have not been successful in speaking with them, but wish to do so. We will keep you advised, Commissioners, of developments in that regard.

A number of these witnesses are located overseas. For that reason, some will be called to give evidence by phone or video link and that may require us to request, Commissioners, that you have earlier than usual start and finish times given the time differences. We will try to minimise the inconvenience to you and to parties in making these arrangements.

We expect these witnesses can be examined in the course of some three weeks of public hearings. That will require considerable expedition from all involved so that the Commission can meet its reporting date of 30 April 2020.

Further information from interested persons

Commissioners, you have invited any person with information relevant to the Commission's Terms of Reference to submit that material in writing to the Secretary by 31 March. If that information may require testing and further investigation, it is to be submitted as soon as possible, and in any event before 3 March.

Those who have leave will have access to the Commission's electronic 'public book' and be able to view and download documents using that system. Most of the documents I will seek to tender today were made available to parties earlier in the week. A larger body of documents that we have identified as potentially relevant to the terms of reference were made available yesterday. Parties wishing to obtain access should contact the Commission's Secretary if they have not already.

If there are persons who require access to the public book for the purpose of making a submission or giving evidence, I would ask that they make contact with the Commission's Secretary as soon as possible about what arrangements need to be made in that regard.

Appendix 6 – Interviews, statements and submissions to the Commission

Interviews

Interviews were conducted with the following persons:

Name	Method of interview	Date
Mr Malcolm Barker	Telephone	14 January 2020
Mr Ben Brampton	In person	20 February 2020
Mr David Brett	Telephone	17 March 2020
Mr Daryl Brigden	In person	26 February 2020
Mr William Curlewis	Telephone	7 March 2020
Mr Christopher Dann	In person	28 February 2020
Joint Interview: Mr Alistair Dawson APM and Mr Michael Shapland MBE	In person Telephone	19 December 2019 6 January 2020
Mr Stephen Dewar	Telephone	13 January 2020
Mr Bruce Embery	In person	21 February 2020
Mr Peter Foster	Telephone	15 January 2020 and 22 January 2020
Mr Timothy Griggs	In person	14 February 2020
Mr Mark Hamilton	In person	25 February 2020
Mr Richard Herweynen	In person	13 February 2020
Mr Eric Lesleighter	In person	21 February 2020
Mr Francisco Lopez	Telephone	5 February 2020
Mr Jose Lopez	Telephone	11 March 2020
Dr Shayan Maleki	In person	27 February 2020
Mr Michael Marley	In person	29 January 2020 and 25 February 2020
Mr David Murray	In person	9 March 2020
Mr Graeme Newton	In person	18 February 2020
Mr Andreas Neumaier	Telephone	24 February 2020
Mr Russell Paton	In person	24 February 2020
Mr Ken Pearce	In person	24 February 2020
Dr Steven Pells	Telephone	30 January 2020
Mr Jonathon Reid	Telephone	6 February 2020

Name	Method of interview	Date
Dr Ernest Schrader	Telephone	10 February 2020 and 26 February 2020
Mr David Starr	Telephone	10 January 2020
Mr Glenn Tarbox	Telephone	24 January 2020
Mr Stephen Tatro	Telephone	30 January 2020
Mr Michael Wallis	Telephone	27 February 2020
Mr James Willey	In person	23 January 2020 and 14 February 2020
Mr Jonathan Williams	Telephone	21 January 2020
Mr John Young	Telephone	20 January 2020
Joint Interview: James Willey Timothy Griggs	In person	17 March 2020

Witness Statements

Statements were provided by the following witnesses who were also called to give oral evidence at the Commission's hearings. Witness statements for select persons are available on the website.

Name	Statement Date
Mr Christopher Dann	9 March 2020
Mr Peter Foster	19 February 2020
Mr Tim Griggs	12 March 2020
Mr Richard Herweynen	Two statements, both dated 12 March 2020
Mr Jose Lopez	21 March 2020
Dr Shayan Maleki	11 March 2020
Mr Michael Marley	3 March 2020
Mr Russell Paton	4 March 2020
Mr Christopher Nielsen	17 March 2020
Dr Ernest Schrader	9, 10, 11 and 16 March 2020
Mr Glenn Tarbox	3 March 2020
Mr Stephen Tatro	20 February 2020
Mr James Willey	21 February 2020
Mr John Young	3 February 2020

Statements and Statutory Declarations were provided by the following witnesses who were not required to give oral evidence at the Commission's hearings. Witness statements for select persons are available on the website.

Name	Organisation	Date
Mr Peter Allen	Department of Natural Resources, Mines and Energy	28 January 2020
Mr Malcolm Barker	GHD Pty Ltd	5 February 2020
Mr William Curlewis	Hydro Tasmania (at relevant time)	19 March 2020
Mr Stephen Dewar	Building Queensland	20 January 2020
Mr Francisco Lopez	Technical Review Panel – SMEC Australia Pty Ltd	Undated
Dr Steven Pells	Pells Sullivan Meynink	27 February 2020
Mr Jonathon Reid	SMEC Australia Pty Ltd	19 February 2020
Mr Greg Rogos	Golder Associates Pty Ltd	Undated
Mr Jonathan Williams	GHD Pty Ltd	20 February 2020

Oral evidence

Persons who gave oral evidence at the Commission's hearings:

Name	Date
Mr Daryl Brigden	2 March 2020
Mr James Willey	3 March 2020
Mr Peter Foster	4 March 2020
Mr John Young (via telephone from Canada)	5 March 2020
Mr Michael Marley	5 March 2020
Mr Roberto Montalvo (via telephone from Peru)	6 March 2020
Mr Russell Paton	6 March 2020
Mr Glenn Tarbox (via telephone from USA)	9 March 2020
Concurrent session: Dr Ernest Schrader Mr Timothy Dolen Mr James Willey Mr Stephen Tatro (via telephone from USA) Dr Paul Rizzo (via telephone from USA)	10 March 2020

Name	Date
Concurrent session: Dr Ernest Schrader Mr Timothy Dolen	11 March 2020
Mr Bruce Embery	11 March 2020
Dr Ernest Schrader	12 March 2020
Mr Jose Lopez (via telephone from Colombia)	13 March 2020
Dr Shayan Maleki	13 March 2020
Mr Eric Lesleighter	16 March 2020
Mr Christopher Dann	16 March 2020
Mr Richard Herweynen	17 and 18 March 2020
Mr Timothy Griggs	18 March 2020
Mr Andreas Neumaier (via video link from Sydney)	19 March 2020
Mr Mark Hamilton	19 March 2020
Mr Christopher Nielsen	19 March 2020

Submissions

Public Submissions

Name of submitter	Organisation	Date Received
Ms Bree Grima	Bundaberg Fruit & Vegetable Growers Cooperative Limited	28 February 2020 and 31 March 2020
Mr Stephen Bennett MP	Member for Burnett, Shadow Minister for Child Safety, Prevention of Domestic and Family Violence; Youth, Shadow Minister for Veterans	5 March 2020
Mr Ken Pearce	Retired	31 March 2020

Submissions in response to Discussion Paper dated 13 March 2020

Party	Date Received
SunWater Limited and Burnett Water Pty Ltd	24 March 2020
Department of State Development, Manufacturing, Infrastructure and Planning	24 March 2020
Department of Natural Resources, Mines and Energy	24 March 2020

Submissions in response to Notices of Potential Adverse Findings dated 23 March 2020

Party	Date Received
Macmahon Contractors Pty Ltd	01 April 2020
Hydro Tasmania	01 April 2020
SMEC Australia Pty Ltd	01 April 2020
Golder Associates Pty Ltd	03 April 2020
Dr Ernest Schrader	06 April 2020

Submission in response to Invitation to Comment on Possible Findings dated 24 March 2020

Party	Date Received
SunWater Limited	03 April 2020

Appendix 7 – Practice guidelines and related information

Practice Guideline No.1

Leave to Appear, Communicating with the Commission, Public Hearings, Witness Statements and Other Matters

Part A. Authority to Appear and Legal Representation at Public Sittings

1. Any person summoned to attend before the Commission to give evidence pursuant to s 5(1)(a) of the *Commissions of Inquiry Act* 1950 (Qld) may be represented by a lawyer while that person is giving evidence.

2. Otherwise, appearances and legal representation before the Commission at its public sittings will not be allowed without the Commission's leave.

3. Leave to appear entitles a person or body to participate in the proceedings of the Commission, subject to the Commissioners' control and to such extent as the Commissioners consider appropriate. Any leave to appear may:

   (a) be subject to a condition that no evidence may be tendered or adduced in chief other than by Counsel Assisting the Commission, with the consequence that any evidence the person with leave to appear seeks to have admitted must be included in a witness statement by that person which has been provided in advance to the Executive Director for the attention of Counsel Assisting;

   (b) be limited by restrictions concerning the topic or topics on which the person (or the person's legal representative) may cross-examine any witness or witnesses, or make any submissions;

   (c) be limited by restrictions on the ability of any person (or any person's legal representative) to make oral submissions;

   (d) be limited to making submissions on matters within the Terms of Reference of which they have particular knowledge or expertise.

4. Any leave to appear or to be legally represented may be varied or withdrawn or made subject to additional conditions at any time in the discretion of the Chairperson.

5. Any persons (or organisations or group of persons) wanting leave to appear or leave to be legally represented at any public sitting of the Commission should send a brief written application to the Executive Director by email to secretary@paradisedaminquiry.qld.gov.au, as soon as possible, but by no later than 31 January 2020.

6. The application for leave should:

   (a) identify the person, group or body wanting leave to appear or to be legally represented;

(b) identify those parts of the Terms of Reference in which the person or body is interested or in respect of which their interests may be materially affected by the Inquiry;

(c) identify the grounds on which those interests exist or may be materially affected;

(d) identify those parts of the Terms of Reference in which the person or body has particular knowledge or expertise enabling that person or body to assist the Commission by submissions, together with the sources of that knowledge and the extent of that expertise;

(e) outline (by reference to the Terms of Reference) the subject matter of the proposed submissions;

(f) where leave to be legally represented is sought, give the name and contact details of the proposed legal representative;

(g) provide an email address and telephone number for correspondence.

7. In some cases it will be clear that an appearance or representation is warranted. In those cases the Commission's Executive Director will communicate, on behalf of the Chairperson, a written notification to the relevant individual, as identified in the application. In other cases, it may be that the Commission will be assisted by further information as to the basis upon which leave to appear ought be granted. It is anticipated that such applications be dealt with at the commencement of the Commission's public hearings.

8. Nothing in these Guidelines prevents a person or body from seeking leave to appear or to be legally represented at any time if something occurs which leads the person to believe their interests may be materially affected. The person or body or their legal representative should contact the Executive Director on +61 7 3096 6324 to make arrangements for that application to be received and considered.

9. It is not necessary to appear at the Inquiry in order to make a submission to it.

Part B. Public Hearings

10. The Commission will convene an initial public sitting in mid-February 2020 when:

(a) the Chairperson and Counsel Assisting will make general introductory remarks concerning the nature and scope of the Inquiry;

(b) applications will be heard for leave to appear or to be legally represented at the future public sittings of the Commission (so far as they have not previously been determined on the papers); and

(c) further information as to the conduct of the Inquiry, including likely public sitting dates, will be provided.

11. Subject to the Commissioners' discretion to exclude the public or any portion of the public from any of its sittings, the Commission's hearings will be open to the public and live-streamed via its website.

12. All witnesses giving evidence at the public sittings of the Commission will be called and examined by Counsel Assisting the Commission.

13. The Chairperson in his discretion will allow the cross-examination of a witness on behalf of a person considered by him to have sufficient interest to do so.

Part C. Communicating with the Commission

14. So far as possible without unfairness to any person affected by the work of the Commission, written communications from and to the Commission will occur by email or, where the Commission provides general notice of procedural matters, via the Commission's website.

15. Any person, agency or organisation communicating with the Commission by email should do so initially via info@paradisedaminquiry.qld.gov.au.

16. Unless otherwise specified by the Commission, all witness statements (including attachments) must be provided to the Commission electronically, to secretary@paradisedaminquiry.qld.gov.au in fully text-searchable, multi-page PDF/A format, together with relevant metadata as defined in any document management protocol to be published on the Commission's website.

17. Unless otherwise specified by the Commission, all other information, relevant documents and submissions must be provided to the Commission electronically, in the format specified in paragraph 16, by email to secretary@paradisedaminquiry.qld.gov.au.

18. The Commission may make exceptions to the requirement for information to be provided electronically in the format specified in paragraph 16. Anyone seeking an exception or assistance in meeting this requirement should contact the Executive Director to discuss the way the information might be provided to the Commission. Email: secretary@ paradisedaminquiry.qld.gov.au or telephone: +61 7 3096 6324.

Part D. Evidence and Submissions

19. These Guidelines distinguish between documents containing factual matters within any person's knowledge or belief (evidence), opinion based upon fields of specialised knowledge (opinion evidence) and documents setting out arguments or assertions as to the conclusions the Commission should reach in relation to all or any part of the Terms of Reference (submissions).

20. The Commission:

    (a) seeks evidence from all persons who can provide factual information or historical documents relevant to any of the Terms of Reference;

    (b) it also seeks, but from qualified persons only, opinion evidence relevant to the Terms of Reference;

(c) will invite submissions from members of the public and, at the conclusion of public hearings, from parties who have been given leave to appear.

21. However, any persons who may have particular knowledge of or expertise in the subject matter of the Terms of Reference, enabling them to provide assistance to the Commission by submissions, should seek leave to appear in accordance with Guideline A above.

22. Any person seeking to make written submissions in respect of the subject matter of the Terms of Reference, but not intending to seek leave to appear, should contact the Executive Director by email to secretary@paradisedaminquiry.qld.gov.au outlining the subject matter of the proposed submissions and why the Commission should receive them. The Commission will inform the person whether the proposed submissions will be accepted.

Part E. Summonses

23. The Commission will issue summonses pursuant to s 5 of the *Commissions of Inquiry Act* 1950 (Qld) requiring persons to attend to give evidence and/or to produce documents and/or to give information and answer questions. However:

    (a) the Commission encourages any person with evidence (whether or not contained in documents) or information relevant to the Terms of Reference to volunteer assistance to the Commission; and

    (b) any person in this category who wishes to avoid the issue or operation of a summons should notify the Executive Director immediately so that such a request can be considered by the Commissioner. Notifications can be made by email to secretary@paradisedaminquiry.qld.gov.au

24. Unless otherwise directed by the Chairperson, the primary evidence of witnesses before the Commission (whether summoned or not) is to be given in the form of a written statement.

Part F. Witness Statements

25. Witness statements:

    (a) must either be in affidavit form or verified as statutory declarations under the *Oaths Act* 1867 (Qld);

    (b) must set out comprehensively and so far as possible, in chronological order, the evidence which the witness can give in relation to each aspect of the Terms of Reference;

    (c) must contain only statements of factual matters within the direct knowledge of the witness, unless (d) or (e) below apply;

    (d) may contain statements of factual matters of which the witness has been informed, or believes, if the source of the information or the basis for the belief is clearly identified in the witness statement;

(e) may contain statements of opinion, provided the witness possesses specialised knowledge in a field relevant to the Terms of Reference and exhibits to the statement a copy of his or her curriculum vitae;

(f) must have exhibited to them (by attachment or accompanying presentation) all documents or true copies of documents relating to the evidence given by the witness or otherwise to the Terms of Reference which are in the witness's possession or control, or describe as precisely as possible any such documents which are not in the witness's possession or control and, in that case, state where the witness believes the documents to be located;

(g) must present those exhibits in a way that will facilitate the Commission's efficient and expeditious reference to them, and in particular –

i. where possible, in electronic form, by providing them in fully text-searchable, multi-page PDF/A format in accordance any document management protocol published on the Commission's website;

ii. alternatively, with respect to hard copies, by placing a letter, number or other identifying mark on each exhibit, and by indexing and paginating (or placing tabs in) bundles of documents;

(h) should be prepared by the witness's legal representative where leave to appear and to be legally represented has been granted;

(i) where the witness has no legal representative, may be prepared by the witness with the assistance of Commission staff by arrangement between the witness and (initially) the Executive Director.

26. Following receipt of a witness's primary statement, Commission staff may request or require the witness to:

(a) attend an interview in relation to the contents of the statement or any aspect of the Terms of Reference; and/or

(b) provide a supplementary statement or statements in respect of any matter relating to the Terms of Reference.

27. The Commission may require persons to attend to provide information to, and answer questions asked by, Counsel Assisting and/or Commission staff, concerning any matter relating to the Terms of Reference, before any witness statement has been prepared.

Part G. Publication and Confidentiality

28. Subject to the Chairperson's determination of any application for confidentiality, all information, witness statements (including exhibits to those statements), documents or submissions provided to the Commission may be published on the Commission's website or otherwise made publicly available.

29. Any person who provides a witness statement or any other document to the Commission, and who wishes to apply for confidentiality and/or non-publication orders in relation to the fact of the material being provided or in relation to the whole or any part of the material:

    (a) if it is considered necessary to make any such order *before* providing any material, should contact the Executive Director by email at secretary@paradisedaminquiry.qld.gov.au, to discuss arrangements;

    (b) subject to any arrangements made under (a) above, should provide the material to the Commission under cover of a written notice stating:

        i. the part of the information or material in respect of which confidentiality is sought;

        ii. whether confidentiality is sought in respect of the world at large or subject to acceptance of publication to some person or categories of persons; and

        iii. the grounds on which such confidentiality is asserted to be necessary and appropriate despite the public nature of the Inquiry;

    (c) subject to alternative arrangements being made with the Executive Director, should organise the material provided in such a way as to indicate on its face where confidentiality is sought.

30. Where confidentiality is applied for in relation to material provided to the Commission, either:

    (a) the Chairperson shall decide the application on the papers and notify the person or their nominated legal representative accordingly. If confidentiality is refused, the material or information in question will nevertheless be kept confidential for seven days from notification of the decision; or

    (b) the Commission shall notify the person or his or her nominated legal representative that they will be required to appear before the Chairperson on a date to be advised for further consideration of the application. The material or information in question will be kept confidential until (and in accordance with) the Chairperson's decision following that appearance.

Part H. Further Practice Guidelines

31. The Commission anticipates issuing further practice guidelines concerning matters such as:

    (a) the receipt of submissions from the public;

    (b) arrangements for public sittings, including dates, sitting times, lists of witnesses who may be called from time to time, and transcripts;

    (c) access to documents and information in the online data room;

    (d) access to exhibits tendered at public sittings;

    (e) written submissions.

JOHN H BYRNE AO RFD
Chairperson and Commissioner
6 January 2020

Practice Guideline No. 2

Publication of Witness Statements, Evidentiary and other Matters and Public Hearings

Part A. Provision of Information to Commission

1. Any person with information relevant to the Commission's Terms of Reference may submit that material in writing to the Executive Director by email to secretary@paradisedaminquiry.qld.gov.au by 31 March 2020.

2. Where that information may require further investigation, it ought to be submitted as soon as possible and in any event before 3 March 2020.

3. The submission of any electronic documents (including witness statements and their exhibits, submissions, and all other information) to the Commission is to be in accordance with the Document Management Protocol published on the Commission's website.

4. In addition to the requirements of Part G of Practice Guideline No. 1, any person who provides a witness statement or any other document to the Commission who wishes to claim privilege over all or part of that material should contact the Executive Director by email to secretary@paradisedaminquiry.qld.gov.au as soon as possible.

Part B. Publication of Witness Statements and Evidentiary Materials

5. Subject to any orders by the Chairperson prohibiting publication of any document or information provided to the Commission, witness statements (including attachments) and other evidentiary materials which are provided to the Commission, including documents produced on summons, will be accessible as follows:

   (a) where the material has been lodged with the Commission but has not yet been admitted into evidence - only to persons authorised by the Chairperson;

   (b) for witness statements (and attachments) which have been admitted into evidence at a hearing - to the public via the Commission's website;

   (c) other evidentiary materials, including expert reports and submissions from persons (or groups of persons) having particular knowledge or expertise in the subject matter of the Terms of Reference which have been admitted into evidence at a hearing - usually to the public via the Commission's website (subject to the Chairperson's further consideration in light of the public importance and quantity of such material and any confidentiality or privilege which might properly attach to such material).

6. At hearings conducted by the Commission (subject to the direction of the Chairperson and in addition to the requirements of Part B of Practice Guideline No. 1):

   (a) a witness's evidence-in-chief will be given primarily by way of the witness statement (in cases in which it has been practicable to obtain such a statement) or record of interview that he or she has provided to the Commission;

(b) no document may be tendered in evidence other than by Counsel Assisting;

(c) a person who has leave to appear before the Commission will be given an opportunity to examine a witness who gives evidence-in-chief, subject to any conditions attaching to the order giving them leave to appear and any further order by the Chairperson;

(d) the order of examination of each witness will be at the discretion of the Chairperson, and duplication and repetition must be avoided, but a witness will usually be examined –

i. first, if necessary, by Counsel Assisting to supplement, correct or clarify matters arising on the face of the witness's statement or to take account of material that became available after the witness completed his or her statement or was interviewed by Commission staff;

ii. next, by those representing persons with leave to appear other than the witness;

iii. then, by those representing the witness;

iv. last, by Counsel Assisting;

(e) in the interests of order and expedition, the Chairperson may at any time impose restrictions on the issues about which a witness may be examined and the time available for examination by any other person; and

(f) at the completion of the examination of a witness, the witness shall, unless excused from further attendance, be taken to have been stood down only and to be subject to recall at the direction of the Commission.

7. By 5:00pm on 24 February 2020, Counsel Assisting will provide all parties or their legal representatives with a document setting out the key issues on which the Commission intends to focus during the hearings.

8. Subject to any orders the Chairperson may make, while public hearings are on foot:

(a) where possible, the Commission will publish regularly to the parties and/or on its website a list of the witnesses to be called to give oral evidence and the proposed dates and times of their evidence;

(b) the published list of witnesses will be updated regularly (and remains, therefore, subject to change);

(c) if the witness's statement has not already been made available to the parties, the Commission will, where possible, make the witness's statement available to the persons with leave to appear at least 2 business days before the witness is called;

(d) where possible, 4 business days before a witness is called, the Commission will give the witness or his or her legal representative notice of the Commission's area

of interest and a list of the documents to which the witness may be taken to (other than those attached to or referred to in the witness's statement) and provide all other parties with an interest in such issues or documents with copies of the notice and the list;

(e) at least 2 business days before the witness is to be called to give evidence, any person with leave to appear who wishes to cross-examine the witness must give notice to the Executive Director by email to Secretary@paradisedaminquiry.qld.gov.au specifying –

i. the name of the witness proposed to be cross-examined;

ii. a considered estimate of the time which will be required for the cross-examination;

(f) if the person giving a notice of proposed cross-examination anticipates showing the witness any document –

i. if the document has already been provided to the Commission, it must be identified in the notice;

ii. if the document is not already available on the Commission's website (whether as an attachment to a witness statement or otherwise), a copy of it must be provided with the notice in one of the following electronic formats:

- Text for plain text records;
- Fully text searchable PDF/A or PDF for formatted document type records;
- TIFF for images such as plans;
- JPEG 2000 or JPEG for photos;
- MPEG4 for videos;

(g) any person with leave to appear who wishes to have evidence adduced from a witness other than a witness proposed to be called by Counsel Assisting must give notice to the Executive Director by email to secretary@paradisedaminquiry.qld.gov.au accompanied by a proof of evidence from the witness.

9. Nothing in this Guideline prevents a person seeking leave to cross-examine a witness at any time during the Inquiry if something occurs during the Inquiry which leads that person to believe that his or her interests may be adversely affected.

10. Any person with leave to appear who wishes to raise a procedural matter must give notice to the Executive Director by email to secretary@paradisedaminquiry.qld.gov.au identifying the matter, stating the outcome sought, and summarising the submissions to be advanced in support of that outcome.

Part C. Public Hearings

Initial Hearing

11. The Commission will hold an initial public hearing on 20 February 2020 at Court Room 4, Brisbane Magistrates Court, Level 1 363 George Street, Brisbane, Queensland.

12. No witnesses will be called at the initial hearing.

13. Applications for leave to appear or to be legally represented at future hearings (so far as the application has not previously been determined on the papers) will be heard.

14. The Chairperson will make opening remarks.

15. Senior Counsel Assisting the Commission will make opening submissions.

Future Hearings

16. Public hearings will be held on the following dates and locations indicated in the table below:

Date	Location	Address
2 – 6 March 2020	Supreme/District Court Room, Bundaberg Courthouse	44 Quay Street, Bundaberg
9 – 13 March 2020	Court Room 4, Level 1, Brisbane Magistrates Court	363 George Street, Brisbane
16 – 20 March 2020	Court Room 17, Level 4, Brisbane Magistrates Court	363 George Street, Brisbane
6 – 7 April 2020	Virtual	Online

JOHN H BYRNE AO RFD
Chairperson and Commissioner
6 April 2020

Document Management Protocol

Purpose of this Protocol

1.1 This Protocol sets out the means and format in which electronic documents are to be produced to the Paradise Dam Commission of Inquiry (**Commission**).

1.2 To facilitate the expeditious conduct of the Inquiry, the Commission intends, as much as possible, to receive, manage and consider, materials in electronic form.

1.3 The Protocol should be read in conjunction with Practice Guidelines 1 and 2, which are available on the Commission's website at www.paradisedaminquiry.qld.gov.au.

1.4 Where the Commission thinks it appropriate, this Protocol may be varied, changed or replaced at any time.

1.5 Pursuant to this Protocol, a person is expected not to convert electronic documents to hard copy for the purposes of providing documents to the Commission. Unless otherwise agreed with the Commission, a person is expected to convert hard copy documents to electronic form for the purposes of production to the Commission in accordance with this Protocol.

1.6 The Protocol applies to:

(a) all witness statements (including exhibits to witness statements); and

(b) unless otherwise specified by the Commission, all other information, relevant documents and submissions referred to in Practice Guidelines 1 and 2.

General Principles

2. Identification of documents

2.1 Document identifiers (Document IDs) and page numbers will be unique to each page and will be the primary means by which documents will be referenced.

2.2 A person will identify documents for the purpose of production using unique Document ID. A Document ID will be in the following format:

PPP.BBB.FFF.NNNN

where:

Level	
PPP	The producing party code is a three alpha code unique to each producing. The Commission will liaise with producing parties and advise the producing party code to be used by each party.

Level	Description
BBB	The box number identifies a specific physical archive box or email mailbox or any other physical or virtual container. The box number is padded with zeros to consistently result in a 3 digit structure.
FFF	The folder number identifies a unique folder number allocated by each party in its own document collection. The folder number is padded with zeros to consistently result in a 3 digit structure.
NNNN	This refers to each individual page of each document. The page number is padded with zeros to consistently result in a 4 digit structure.

An example of the Document ID structure is XYZ.001.001.0001

where:

XYZ	Party Code
001	Unique box number allocated by person
001	Unique container number allocated by person
0001	Sequential page or document number

Note: If a different number is required, please contact the Commission to discuss.

2.3 It is understood and accepted that Document IDs may not be consecutive as a result of the removal of irrelevant documents during review. Host and attachment documents must, however, be identified and be given consecutive Document IDs.

2.4 A document filename is to be adopted according to its corresponding Document ID upon electronic production.

Document Management

3. Document metadata

3.1 Wherever possible, a person is to rely on the automatically identified metadata of electronic documents. Automatically identified metadata should be used when:

(a) searching for documents;

(b) itemising documents in a list;

(c) producing documents in accordance with the Production Specification at Schedule

3.2 A person should take reasonable steps to ensure that all appropriate document metadata is not modified or corrupted during collection and preparation of electronic documents for review and production.

3.3 The Commission accepts that complete document metadata may not be available for all electronic documents. A person should attempt to provide complete metadata where practicable.

4. De-duplication of documents

4.1 A person must take reasonable steps to ensure that duplicate documents are removed from the exchanged material (de-duplication).

4.2 Duplication will be considered at a document group level. That is, all documents within a group comprising a host document and its attachments, will be treated as a duplicate only if the entire group of documents is duplicated elsewhere.

5. Exclusion of unusable file types

5.1 Files with no user-generated content, such as system files and executable files, are to be excluded from the disclosure process (to the extent possible).

5.2 Temporary internet files and cookies are to be excluded from the disclosure process.

Document Production

6. Production of documents to the Commission

6.1 All documents will:

(a) be accompanied by an excel spreadsheet as detailed at Schedule 1;

(b) be provided in electronic format in accordance with paragraphs 7, 8 and 9;

(c) include all requested metadata and files responsive to the production or tranche in their entirety.

7. Document format and naming

7.1 All documents will be provided as fully text-searchable images as multi-page PDF/A files.

7.2 Electronic documents that do not lend themselves to conversion to PDF (for example, complex spreadsheets or databases) may be provided to the Commission as native electronic documents or in another form as agreed by the producing party and the Commission.

7.3 Each file provided by a producing party to the Commission will be stored in the folder structure that matches the Document ID structure. Further information is contained in Schedule 2 to this Protocol.

7.4 A unique page number label in the format described in paragraph 2.2 will be electronically stamped on the top right hand corner of each page of every document. Such page numbering can be readily achieved using commercial off the shelf products such as Adobe Acrobat Professional or Nitro PDF, however, any similar method will suffice.

7.5 The page number assigned to the first page of a document will be the Document ID for that document.

8. Format for witness statements and submissions

8.1 To enable hyperlinking to exhibits referred to within witness statements or submissions:

(a) witness statements and submissions should be provided as both –

i. Microsoft Word documents; and

ii. fully text-searchable images as multi-page PDF/A files;

(b) where a document is referred to in a submission or witness statement, the reference must be to the Document ID for the document; and

(c) each reference to an exhibit's Document ID should be made enclosed in double square brackets, for example [[ABC.001.001.0345]].

9. Completeness of documents

9.1 Where documents are produced, all parts of the document should be produced. For example, for an email chain the final instance of that chain, showing all parts of that chain, is to be produced along with every attachment.

10. Production media

10.1 Documents and accompanying metadata should be provided to the Commission on a solid state universal serial bus storage (USB stick) or a portable hard drive or read-only optical media (e.g. CD-ROM, DVD-ROM), and delivered to the Commission at Level 23, 50 Ann St, Brisbane.

11. Data security

11.1 Producing parties will take reasonable steps to ensure that the data is useable and is not infected by malicious software.

11.2 If data is found to be corrupted, infected by malicious software or is otherwise unusable, the producing party will, within 2 working days of receipt of a written request from the Commission, provide a copy of the data that is not corrupted, infected by malicious software or otherwise unusable (as the case may be).

Schedule 1 – Production specification

Excel index

1.1 All documents to be produced will be itemised in an excel index containing the following information for each document, where available:

(a) Document ID (see paragraph 2.2 of the Protocol);

(b) host Document ID (see below "**Document hosts and attachment relationships"**);

(c) document date;

(d) document type (see tab "**DocType List"** in the sample spreadsheet referred to in paragraph 1.2 of this Schedule);

(e) document title;

(f) author;

(g) author organisation;

(h) recipient;

(i) recipient organisation;

(j) confidential – yes/no/part and, if partly confidential, identifying the relevant part (refer to Practice Guideline No. 1 at paragraph 29(b)(i));

(k) confidential – scope (refer to Practice Guideline No. 1 at paragraph 29(b)(ii));

(l) confidential – grounds (refer to Practice Guideline No. 1 at paragraph 29(b)(iii)).

1.2 A sample spreadsheet is available from the Commission website www.paradisedaminquiry.qld.gov.au.

Document hosts and attachment relationships

1.3 Every document that is attached to another document will be called an attached document.

1.4 Attached documents will have the Document ID of their host document in the metadata field called 'Host Document ID'.

1.5 Host documents and attached documents are jointly referred to as a 'Document Group'.

1.6 In a Document Group, the host document will be immediately followed by each attached document in the order of their Document IDs.

1.7 Annexures, attachments and schedules to an agreement, report, legal document or minutes of a meeting may be described as separate attached documents associated with the relevant host document.

Schedule 2 – Folder structure and naming of files

2.1 This schedule specifies how electronic documents and images are to be located and named for the purposes of production to the Commission.

2.2 The folder containing all documents will be named either '\Documents\' or '\Images\'.

2.3 Documents produced as searchable images will be named 'Document ID.pdf'. Only the final full stop between the Document ID and the file extension will be used (e.g. 'ABC0010020312.pdf').

2.4 Documents produced as native electronic documents will be named 'DocumentID.xxx(x)' where 'xxx(x)' is the original default file extension typically assigned to source native electronic files of that type (for example, 'ABC0010020312.docx').

2.5 Folders containing documents will be structured in accordance with the Document ID hierarchy. For example, the document produced as a searchable image called 'ABC0010020312.pdf' would be located in the folder called 'Documents\ABC\001\002\'. That document will appear in the directory listing as 'Documents\ABC\OO1\002\ABC0010020312.pdf'. Where this same document has been produced as a Word document, it would be called 'ABCOO10020312.doc' and will be located in the folder called 'Documents\ABC\001\002\'. It will appear in the directory listing as 'Documents\ABC\001\002\ABC0010020312.doc'.

Appendix 8 – Concurrent evidence session – protocol and agenda

I Preamble

A. The Commissioners of the Paradise Dam Commission of Inquiry are directed by paragraph 3(a) of the Terms of Reference to inquire into, among other things, the root cause of structural and stability issues identified in engineering and technical studies between 30 January 2013 and 30 November 2019.

B. One such structural and stability issue is the sliding stability of the Paradise Dam (**the Dam**) as assessed by GHD Pty Ltd in GHD.005.0001 (**the GHD Stability Assessment**) and subsequently which suggest that the Dam does not meet the Guidelines of Australian National Committee on Large Dams (ANCOLD) (2013). More specifically, this issue involves, by reference to the Commission's revised 'Key Issues' dated 4 March 2020, as follows:

3.1 In terms of sliding stability:

e. adequacy of the bond between the roller-compacted concrete (RCC) lifts;

f. whether the consequences of using the particular 'lean' RCC mix adopted for the Dam limited the practicability of verifying shear strength parameters by *in situ* testing and, for that reason, necessitated greater reliance on quality management systems that recorded whether and to what extent specified construction methodologies and practices were adhered to than for mixes with higher cementitious content;

…the adequacy of testing and checking (and the standards against which such testing was undertaken) reliably to verify that the lift joints were of a quality likely to result in a dam about which there could be reasonable satisfaction of stability and structural integrity;

f. whether the Dam, as designed, 'essentially achieves stability with current friction values alone' (see, for example, SUN.010.002.0047) …;

g. what standards are properly to be applied in assessing the Dam's sliding stability.

C. One of the principal components of the GHD Stability Assessment is that the roller-compacted concrete (RCC) lifts within the Dam are below the 'Acceptance Criteria' in Table 6.1 of the ANCOLD Guidelines.

D. The question of whether these Acceptance Criteria are met is materially informed by assessments of the shear strength of those lifts and by an understanding of the characteristics of RCC, and especially 'lean mix' RCC.

E. The Commission intends to call experts with specialised knowledge of RCC, and, in particular, the assessment of its shear strength of RCC, and the circumstances in which it may be engineering good practice to undertake such assessments as well about its properties.

F. Pursuant to s 17 of the *Commissions of Inquiry Act* 1950 (Qld), the Commission considers that the means by which the issues set out below ought be inquired into includes by the giving of concurrent evidence by witnesses with specialised knowledge.

G. The expert witnesses proposed to be included in the concurrent evidence sessions are:

Session 1: RCC shear strength testing - 10 March 2020

a. Dr Ernest Schrader (in person)

b. Mr Timothy P Dolen (in person)

c. Mr Stephen Tatro (by video or phone link)

d. Dr Paul C Rizzo (by video or phone link)

e. Mr James Willey (in person)

Session 2: Dam construction with RCC – 11 March 2020

a. Dr Ernest Schrader (in person)

b. Mr Timothy P Dolen (in person)

H. The issues set out below are framed to facilitate the orderly and expeditious inquiry into the question of the nature and cogency of the assessment of the Dam's sliding stability undertaken by GHD as well as the root cause of the structural and stability issue.

I. The concurrent evidence session is to be conducted by Counsel Assisting so that the various experts, guided by the Commission, work to arrive (where possible) at a common resolution of the issues stated below.

II Issues to be addressed at concurrent evidence session

Session 1: RCC shear strength testing

1. Identify the shear strength test results and assessments undertaken on samples from Paradise Dam which are <u>not</u> the subject of criticism as between the witnesses.

2. On that basis, what is the difference between witnesses as to the stability of the Dam, and what, precisely, is that difference?

   Of the test results and assessments which *are* the subject of criticism, to what extent do those criticisms have merit in connection with samples being subjected to multi-stage testing, having regard to:

a. ASTM D5607, Uplift Pressures, Shear Strengths, and Tensile Strengths for Stability Analysis of Concrete Gravity Dams (EPRI 1992) and relevant guidance from the US Bureau of Reclamation;

b. the respective residual friction angles which were returned in the various stages of the testing undertaken;

c. the extent of displacement that the lift joints would undergo if the Dam were to fail in sliding.

3. With respect to the testing and assessment undertaken on samples of RCC from the Dam:

a. to what extent, if any, does the testing and assessment undertaken in 2006 and as reported in Commission Exhibits 59 and 60 and in the Report of Mr Montalvo dated 14 September 2006 assist in informing the sliding stability of the Dam;

b. what, if any, and to what degree, are the results of shear strength testing of samples undertaken in 2015 and in 2019 a reliable basis for an assessment of the Dam's sliding stability;

c. have a sufficient number of tests been undertaken to give shear strength parameters with reasonable certainty (for the purposes of the applying the Acceptance Criteria) so as to derive 'well-defined' shear strength parameters;

d. what degree of reliability can be attached to GHD's assessment of sliding stability so far as it is informed by the tests referred to in 4(a) and (b) above?

4. If a sufficient number of tests have <u>not</u> yet been undertaken to give shear strength parameters with reasonable certainty (within the meaning of the Acceptance Criteria) within the Dam:

a. what further such testing or assessment could practicably be undertaken to derive 'well-defined' shear strength parameters to inform an assessment of the Dam's sliding stability;

b. how ought that testing be undertaken and by whom?

5. Is it reasonably practicable, all things considered, or necessary, to undertake further shear strength tests and assessments?

Session 2: RCC qualities and assessment

1. The properties and comparative advantages and disadvantages of low (LCRCC), medium and high cementitious mix RCC in dam construction.

2. The consequences for dam construction when using LCRCC, including as to:

   a. the dimensions of a dam and its lift joint surface areas;

   b. whether reliance ought be placed, in terms of the dam's sliding stability, on friction alone;

   c. the achievement of bonded lifts;

   d. the means available to test for shear strength;

   e. construction methods and methodologies;

   f. the extent to which good engineering practice requires the use of bedding mix between lifts.

3. Whether the Lift Joint Quality Index:

   a. is properly to be used to establish or calculate values for cohesion and friction;

   b. if applied, is a substitute for shear strength testing, and if so, in what circumstances.

III Protocol

1. Witnesses will be provided with:

   a. a copy of this Agenda and Protocol;

   b. a brief of relevant documents (including any report provided by a witness to the Commission to date).

2. Counsel Assisting will conduct the concurrent evidence session in accordance with this Agenda and Protocol and be responsible for ensuring the fairness of the session and that each witness has a reasonable opportunity to speak on each issue, and to express their agreement or disagreement (and reasons why) with other witnesses.

3. Questioning of the witnesses will be by Counsel Assisting, except as stated below. Counsel Assisting will ask questions of witnesses and direct the giving of evidence and the sequence of it in accordance with the agenda above.

4. Any Counsel for any party given leave to appear will, no later than 24 hours before the commencement of the concurrent evidence session, provide to Counsel Assisting, any points they wish to be raised to explore or expose issues stated in the agenda above.

5. Counsel Assisting will consider any points notified in accordance with 4. above, but will not be bound to pursue such points with witnesses.

6. To the extent a party has notified an issue in accordance with 4. above, and if Counsel Assisting does not pursue the issue with witnesses, or does so inadequately in the view of a party, that party may, through its Counsel, seek leave of the Chairperson to ask questions of witnesses.

7. The Chairperson may grant leave to a party to ask questions of witnesses whether or not the procedure in paragraphs 4-6 above has been followed.

8. Questioning of the witnesses shall be conducted sequentially in terms of the issues set out below, with any application for leave to be raised and decided at the conclusion of the questioning by Counsel Assisting upon each individual issue.

9. Nothing in this Protocol precludes or curtails the usual right of a Counsel for a party to object to a question put by Counsel Assisting in the course of evidence.

10. Counsel Assisting will decide the order in which witnesses speak on a particular issue but will, as much as reasonably possible, rotate or change the order in which witnesses speak on different issues.

11. Witnesses who are physically present or present by video link at the concurrent evidence session may indicate their wish to speak by raising their hand. Witnesses appearing by telephone who wish to speak during the concurrent evidence session can indicate their desire to speak by saying their last name.

12. The concurrent evidence session No 1 will be conducted, so far as is possible, to commence at 8.30am AEST on Tuesday, 10 March 2020 and to conclude no later than 2.30pm AEST.

13. The concurrent evidence session No 2 will be conducted, so far as is possible, to commence at 8.30am AEST on Wednesday, 11 March 2020 and to conclude no later than 2.30pm AEST.

14. The concurrent evidence session is exempt from the requirements of Practice Guideline No 2 paragraph 8(d) to (f).

Counsel Assisting
9 March 2020

Appendix 9 – List of exhibits

Exhibit No	Description	Document ID
Tender list 1 – Exhibit A		
1.	Commissions of Inquiry Order (No. 1) 2019	PDI.024.0001
2.	Invitation to submit a registration of interest – January 2003	ALL.155.008.0001
3.	Quality Control Report – August and September 2005	ALC.002.001.0750
4.	Memoranda from R. Herweynen – 5 October 2005	SUN.126.001.0001
5.	NSW Public Works Report – 22 August 2013	DNR.002.8498
6.	Statement of Mr Stephen Dewar – 20 January 2019	DES.001.001.0001
7.	TRP Report No 1 – October 2013	IGE.017.0001
8.	TRP Report No 2 – January 2014	IGE.018.0001
9.	TRP Report No 3 – November 2014	IGE.019.0001
10.	TRP Report No 4 – 15 December 2015	IGE.020.0001
11.	TRP Report No 1 – 29 May 2019 and 23 September 2019	SUN.009.003.0613
12.	TRP Report No 2 – 23 September 2019	IGE.051.0001
13.	TRP Report No 3 – 9 December 2019	SUN.009.002.0001
14.	Report of Tatro Hinds 'Shear Strength Evaluation Comments' – 25 November 2019	TAT.001.0001
15.	GHD Memorandum – 5 September 2019	DNR.001.2363
16.	GHD Memorandum – 25 November 2019	GHD.005.0001
17.	Statutory Declaration of Mr Peter Allen – 28 January 2020	PTA.001.0001
18.	Alliance Agreement – 27 February 2004	SUN.009.002.0020
19.	Alliance Agreement – 23 May 2005	ALL.144.002.0389
20.	Specifications Part 1	DNR.004.4559
21.	Specifications Part 2 of 2 (contains RCC Specs)	DNR.003.8385
22.	Method Statement for RCC & Trial Mix Specifications – undated	DNR.010.8266
23.	Roller Compacted Concrete Specifications – undated	ALC.002.001.1176

Exhibit No	Description	Document ID
24.	Detail Design Report – June 2004	GHD.002.0001
25.	Final Detail Design Report – November 2005	DNR.001.0267
26.	A Report of the Inspector-General of Emergency Management – 19 December 2019	IGE.084.0001
27.	SunWater submission to the Inspector-General – 28 October 2019	IGE.076.0001
28.	Development Permit – 30 October 2003	DNR.003.7173
29.	Development Permit – 3 June 2004	DNR.003.7159
30.	Development Permit – 6 October 2005	DNR.003.7192
31.	Paradise 2004 select memos RC 20191112	SUN.009.002.0147
32.	Paradise 2005 select memos RC 2019111	SUN.009.002.0203
33.	ANCOLD Guidelines 1991	ACD.003.0001
34.	ANCOLD Guidelines 1992	ACD.002.0001
35.	ANCOLD Guidelines 2013	ACD.001.0001
36.	Memoranda from E. Schrader – 23 September 2004	SUN.010.002.0024
37.	Memoranda from E. Schrader – 2 August 2004	SUN.010.002.0047
38.	RCC Quality Control Report (Report No. 9)	SUN.110.003.0001
39.	Sunwater Due Diligence – Schrader-Montalvo Presentation	ALC.001.001.1874
40.	Letter Sunwater to DEWS – 3 May 2013	DNR.013.0571
41.	Paradise Dam Remedial Works Inspection	DNR.012.9331
42.	Sunwater Paradise Dam Safety Review Revised Report – 2016	DNR.002.3132
43.	Construction Drawing BDA-D-C-008 General Arrangement	SUN.024.001.0001
44.	Construction Drawing BDA-D-C-011 Primary spillway	DNR.008.6388
45.	Photograph – 10 November 2008 – Dam Wall from downstream view	SUN.024.001.0002
46.	Photograph – 10 November 2008 – from left abutment view	SUN.024.001.0003
47.	Photograph – 7 September 2017 – from right abutment downstream view	SUN.024.001.0004

Exhibit No	Description	Document ID
Tender list 2 – Exhibit B		
48.	Transcript of interview with Mr Daryl Brigden – 26 February 2020	TRA.510.025.0001
49.	Curriculum Vitae of Daryl Brigden	BRD.001.0002
50.	RCC trial section construction Report No 1	SUN.114.003.0001
51.	Memorandum from Dr Schrader 'Unformed Downstream Face of the RCC' – 4 August 2004	SUN.010.002.0355
52.	Memorandum from Dr Schrader 'RCC Density verification in situ' – 18 August 2004	DNR.011.1449
53.	SunWater Due Diligence Workshop notes – 11 & 12 August 2005	SUN.020.003.6637
54.	Burnett River Dam Fitness for Purpose Design Review – November 2004	SUN.016.014.1266
55.	Burnett River Dam Fitness for Purpose Review – December 2005	SWA.505.001.0014
Tender list 3 – Exhibit C		
56.	Statement of Mr James Willey – 21 February 2020	WYJ.001.003.0001
57.	Curriculum Vitae of James Willey	WYJ.001.002.0001
58.	CC Investigations Location Plan	WYJ.001.001.0001
59.	Spreadsheets from core logs – 2006	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

Exhibit No	Description	Document ID
		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
60.	Compilation of core photographs – 2006	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
61.	GHD Memorandum Review of RCC Shear Strengths – 28 February 2020	SUN.009.004.0037
62.	Dam forces diagram	PDI.067.0001
63.	James Willey circles Basalt Pimple on SUN.024.001.0002	WYJ.005.0001

Exhibit No	Description	Document ID
64.	James Willey circles Basalt Pimple and Skyslab connection on SUN.024.001.0002	WYJ.006.0001
65.	Updated GHD RCC Dam Investigation Locations	GHD.040.0001
66.	Report of Tatro Hinds 'Shear Strength Evaluation Comments' – 25 November 2019 (unredacted)	IGE.028.0001
Tender **list 4 – Exhibit D**		
67.	Statement of Mr Peter Foster – 19 February 2020	PTF.001.001.0001
68.	ICOLD Bulletin 126 – RCC Dams, State of the Art and Case Histories	ICO.001.0001
69.	ICOLD 2019 Update to Bulletin 126 – Roller Compacted Concrete Dams	ICO.002.0001
70.	USBR –Design Criteria for Concrete Arch and Gravity Dams	PDI.074.0001
71.	Statement of Mr Jonathon Reid –19 February 2020	JTR.001.001.0001
72.	Statement of Dr Steven Pells – 27 February 2020	ST001.0001
73.	Statement (unexecuted) of Mr Francisco Lopez	LOF.001.0001
	Annexure FL-1	LOF.002.0001
	Annexure FL-2	LOF.003.0001
	Email from K Spies, Clyde & Co, confirming truth and accuracy of unexecuted statement– 4 March 2020	LOF.004.0001
Tender list 5 – Exhibit E		
74.	Article by Herweynen, Griggs, Schrader and Starr 'Burnett RCC Dam Design – an innovative approach to site specific conditions'	PDI.035.0001
75.	Article by Lopez, Griggs, Montalvo, Herweynen and Schrader 'RCC Construction & Quality Control for Burnett Dam'	PDI.037.0001
76.	Statement of Mr John Young – 3 February 2020	YOJ.001.001.0001
77.	Paradise Dam Spillway Improvement Project, TRP No 2 (presentation) – 27-28 August 2019	ALC.002.001.1085
78.	Memo from R. Herweynen re: clean-up under spillway apron – 14 October 2004	SUN.116.002.0001

Exhibit No	Description	Document ID
79.	GHD Paradise Dam Preliminary Design Report (Draft) – July 2019	SUN.009.003.0001
80.	Burnett River Dam – Specification – Section 3.0 Surface Excavation and Earthworks	SUN.119.001.0028
81.	URS (Burnett Alliance) Stage 2 Proposal Volume 3A Technical Information – 29 July 2003	DNR.007.1087
82.	Golder Associates engagement letter – 24 December 2003	GOL.002.0001
83.	Statement of Mr William Michael (Mike) Marley – 2 March 2020	MAR.001.0001
84.	Design calculation files – Burnett Dam Alliance	DNR.005.3464
85.	Draft Geotechnical Design Report: Burnett River Dam, Vol 1	DNR.006.3286
86.	Dam Foundation Inspection (DFI) Register	SUN.116.004.0002
87.	Design calculation files – Burnett Dam Alliance	DNR.005.4145
88.	Design calculation files – Burnett Dam Alliance	DNR.005.4886
89.	Golder presentation to Sunwater regarding Geotechnical Design Verification, Geotechnical Mapping, Testing & Documentation during Construction	DNR.011.1864
90.	Peer Review of Foundation Adequacy – 2004	DNR.010.0918
91.	Statement of Mr Russell Paton – 4 March 2020	PAR.001.0001
92.	Burnett Dam - SunWater Due Diligence Workshop, Civil Group – 11 & 12 August 2015	SUN.016.010.4219
93.	Email from Richardson re: Sunwater Spillway 2D model study – 28 May 2003	SUN.018.026.5652
94.	Sunwater Spillway 2D model study – amended April 2003	SUN.018.026.5653
95.	Preliminary Comments on RCC Coring at Paradise Dam by Robert Montalvo	ALC.001.001.1683
96.	Burnett River Dam Preliminary Design Report Vol 1 – February 2003	DNR.003.7930
97.	Statement of Mr Jonathan Williams – 20 February 2020	WLJ.001.003.0001

Exhibit No	Description	Document ID
	Annexure - Curriculum Vitae of Jonathan Williams	WLJ.001.002.0001
98.	ASTM D5607 – Standard Test Method for Performing Laboratory Direct Shear Strength Tests of Rock Specimens Under Constant Normal Force	WLJ.001.001.0001
99.	Curriculum Vitae of Robert Montalvo	MOR.001.0001
Tender list 6 – Exhibit F		
100.	Statement of Mr Glenn Tarbox – 3 March 2020	TAG.001.0001
	Annexure – Curriculum Vitae of Glenn Tarbox	SUN.503.001.0615
101.	RCC Quality Control Report, Report No. 3 – September 2004	SUN.110.002.0158
Tender list 7 – Exhibit G		
102.	Concurrent Evidence Session Agenda and Protocol	PDI.075.0001
	Shear Strength Testing: List of Documents	PDI.076.0001
	Dam Construction with RCC: List of Documents	PDI.077.0001
103.	Letter report by Paul Rizzo 'Recommended Testing Program Paradise Dam – 13 February 2020	RIZ.001.001.0001
	Curriculum Vitae of Paul Rizzo	RIZ.002.0001
104.	Report by Timothy Dolen 'Review of RCC lift joint properties and stability analysis and commentary' – 1 March 2020	GHD.006.0001
	Curriculum Vitae of Timothy Dolen	DOT.001.0001
105.	Article by Francis McLean & James Pierce 'Comparison of Joint Shear Strengths for Conventional and Roller-Compacted Concrete' – undated	DOT.002.0001
106.	Article by Roselle Drahushak-Crow & Timothy Dolen 'Evaluation of Cores from Two RCC Gravity Dams' – undated	DOT.003.0001
107.	Statement of Mr Stephen Tatro – 20 February 2020	TAT.002.0001
	Curriculum Vitae of Stephen Tatro	TAT.002.0001 at ST1.0001
108.	Presentation by Stephen Tatro 'Paradise Dam, Problem, Question and Solution'	TAT.003.0001

Exhibit No	Description	Document ID
109.	Statement (1) of Dr Ernest Schrader – 9 March 2020	SCE.019.0001
	Curriculum Vitae of Ernest Schrader	SCE.018.0001
Tender list 8 – Exhibit H		
110.	Friction (only) Calculation Instructions	PDI.078.0001
111.	Statement (2) of Dr Ernest Schrader – 10 March 2020	SCE.021.0001
112.	Article by Dr Ernest Schrader et al 'RCC knowledge: how specific test can help to evaluate the real behaviour of material and a better design of RCC dams', ICOLD Ottawa Paper 2019, Paper 542 – undated	ICO.003.0001
113.	Article by Dr Ernest Schrader and Dr Ahmed Rashed 'Benefits of Non-Linear Stress-Strain Property & Membranes for RCC Dam Stresses' – undated	SCE.020.0001
114.	Transcript of interview with Mr Bruce Embery – 21 February 2020	TRA.510.018.0001
115.	Burnett Dam Alliance Organisation Chart	SUN.175.006.0009
116.	Quality Management Plan	SUN.162.002.0019
117.	Placement of Roller Compacted Concrete, BDA-QA-CHK-0060 Rev 9 – 16 November 2004	DNR.020.014.4624
118.	Placement of Roller Compacted Concrete, BDA-QA-CHK-0060 Rev 10 – 8 March 2005	SUN.112.001.0277
119.	Placement of Roller Compacted Concrete, BDA-QA-CHK-0060 Rev 10 – 5 August 2005	SUN.112.002.0368
120.	Burnett Dam Alliance, Non-conformance Report – 11 March 2005	SUN.117.004.0270
121.	Burnett Dam Alliance, Non-conformance Report – 8 June 2005	SUN.117.004.0175
122.	Graph – Monolith H – Base at RL 32.4 Friction Only – 50% Uplift	PDI.083.0001
123.	Graph – Monolith H – Base at RL 32.4 Friction Only – 50% Uplift	PDI.084.0001
Tender list 9 – Exhibit I		
124.	Article by Dr Ernest Schrader 'Shear strength and lift joint quality of RCC' – 1999	PDI.040.0001

Exhibit No	Description	Document ID
125.	Article by Dr Ernest Schrader - 'Extension Shearing Testing for Saluda Dam Roller Compacted Concrete'	PDI.047.0001
126.	Transcript of Interview with Dr Ernest Schrader – 26 February 2020	TRA.510.023.0001
127.	Transcript of Interview with Dr Ernest Schrader – 10 March 2020	TRA.510.006.0001
128.	Email: Questions Regarding Design Parameters for Lift Joints	SUN.129.001.0133
129.	Note to File: Design Criteria – Dam Stability Analysis	DNR.005.4886 (at 5056)
130.	Supplemental Statement of Dr Ernest Schrader – 11 March 2020	SCE.022.0001
131.	Nuclear gauge density testing	DNR.005.6570 (at 6593)
132.	Density and nuclear gauge calibration	DNR.005.6570 (at 6594)
133.	Density continued	DNR.005.6570 (at 6595)
134.	Nuclear gauge accuracy	DNR.005.6570 (at 6596)
135.	Density and nuclear gauge continued	DNR.005.6570 (at 6598)
136.	Compaction/ density gauge calibration problem	DNR.005.6570 (at 6599)
137.	Nuclear gauge calibration block	DNR.005.6570 (at 6602)
138.	Carpi, misc. (email)	DNR.005.4886 (at 5143)
139.	Carpi membrane issues	DNR.005.4886 (at 5146)
140.	Plinth – excavation and RCC start	SUN.010.002.0200
141.	RH email: Questions re Design Parameters for Lift Joints	SUN.021.001.8406
142.	Blend sand – RCC aggregate	DNR.011.1234
143.	Goodnight in aggregate	DNR.011.1345
144.	Conveyor issues	DNR.011.1265
145.	Conveyor – mixer hopper and sampling	SUN.010.002.0200 (at 1274)
146.	Material property and thermal update	DNR.011.1361
147.	Example of poor pozzolan performance with lean RCC and basalt	DNR.010.8241
148.	Cofferdam comments for RCC dams	DNR.010.8237
149.	Density meter calibration issues	DNR.005.6570 (at 6587)
150.	Thermal couples	DNR.010.8245
151.	Interior peak temperatures, start RCC October 18 = Day 0	DNR.010.8245 (at 8247)
152.	Specification – permeability testing	DNR.010.8235

Exhibit No	Description	Document ID
153.	Specification – aggregate gradation	DNR.010.8233
154.	Density test requirements and clarification	DNR.010.8239 (at 8240)
155.	Cement and ash comparison to Theiss [sic] mix results	DNR.010.8236
156.	Material property and Thermal update	DNR.007.2295
157.	RCC mix water	DNR.011.1472
158.	RCC aggregate and mix verification tests	DNR.007.2295 (at 2356)
159.	Nuclear gauge Geiger counter	DNR.011.1413
160.	Brief comments and reminders for Jose Lopez	DNR.011.1238 (at 1240)
161.	Thermal update – for ES Schedule as of May 2004	DNR.011.1556 (at 1557)
162.	Thermal properties, analysis and detailed schedule	DNR.011.1585
163.	Email – 60kg mix thermal results	DNR.011.1201
164.	Updated table of material properties	SUN.010.002.0356
165.	Aggregate gradation and handling	DNR.011.1204
166.	Potential change of cement	SUN.010.002.0205
167.	Potential GERCC – Schrader comments to date	SUN.010.002.0208
168.	Misc. comments	DNR.011.1204 (at 1206)
169.	Waterstop protection	DNR.010.8221
170.	Bedding issues	DNR.011.1228
171.	Bedding issues	DNR.006.4207 (at 4288)
172.	Bedding issues supplement	DNR.011.1231
173.	Upstream face and options & trial placement monolith joint	SUN.010.002.0382
174.	Dental concrete	DNR.011.1284
175.	Use new aggregate in current RCC	SUN.010.002.0384
176.	Foundation seeps at dental and RCC	DNR.011.1338
177.	RCC related tests and frequency	DNR.011.1501
178.	Cement storage and silos	DNR.011.1246
179.	RCC density – testing – achieving density – gauge	DNR.011.1452
180.	GERCC Schrader summary comments	DNR.011.1340
181.	Bedding mix supplemental clarifications and recommendations	DNR.011.1232
182.	Bedding fillet at panels	DNR.011.1227

Exhibit No	Description	Document ID
183.	More on GERCC – Schrader response to Tim Griggs summary	DNR.011.1405
184.	Carpi membrane at monolith joints	DNR.011.1244
185.	Top of conduit to RCC contact	DNR.011.1614
186.	Monolith joint former	SUN.010.002.0187
187.	RCC moisture	DNR.011.1475
188.	Rolled down edge – upstream to downstream	DNR.011.1513
189.	Non-linear FEM stress analysis	DNR.011.1412
190.	Grout and treatment of pimple	DNR.014.3416 (at 3475)
191.	Email – cleanup and steps to apron	DNR.011.1336
192.	Stockpile management	DNR.011.1572
193.	Cofferdam – Access - Diversion	DNR.011.1252
194.	Conveyor covers	DNR.011.1263
195.	Strain gauge instrumentation	DNR.011.1575
196.	Monolith joint former	DNR.011.1398
197.	Misc. comments – production – membrane punctures – strain gauges	DNR.011.1381
198.	Misc. comments wet weather RCC	DNR.011.1379
199.	Conveyor modifications – chute and upstream area – and allowed time to compaction (retarder)	DNR.011.1271
200.	Spillway crest top-out	DNR.011.1570
201.	RCC to abutment contact	DNR.011.1511
202.	RCC lift surface cleaning	DNR.011.1459
203.	Email – set time and time to compaction	DNR.011.1565
204.	RCC time of compaction – again	SUN.010.002.0286
205.	RCC quality control testing	SUN.126.001.0167
206.	RCC quality control testing – additional comment	SUN.010.002.0275
207.	Email – low density top of spillway	DNR.011.1360
208.	RCC dam - thermal	DNR.011.1448
209.	RCC dam issues	DNR.011.1442
210.	RCC dam - Right abutment and apron	DNR.011.1444
211.	Grout contacts	DNR.011.1248
212.	Left abutment extra joint	SUN.010.002.0142 (at 0143)
213.	RCC cores	SCE.025.0001 (at 0937)
214.	Roller Compacted Concrete Cores and Coring – Mexico	SUN.126.002.0049 (at 0050)

Exhibit No	Description	Document ID
Tender list 10 – Exhibit J		
215.	Email chain: to Richard Herweynen from Ernest Schrader re: Coring of RCC – 17 May 2005	SCE.023.0001
216.	Email chain: to Mark Hamilton from Robert Montalvo re: FW: Paradise – dam RCC data sheet – 22 March 2006	SCE.024.0001
217.	Email: to Mark Hamilton from Richard Herweynen re: Burnett Cores – Ernie Comments – 3 March 2006	SCE.025.0001
218.	Email chain: to Richard Herweynen from Robert Montalvo re: Burnett Cores + Assessment before testing – 11 March 2006	SCE.026.0001
219.	Email chain: to Ernest Schrader et al from Robert Montalvo re: Burnett – Core tests –Ernie Follow-up – 1 January 2007	SCE.027.0001
220.	Email chain: to Ernest Schrader et al from Jose Lopez re: FW: Core Hole in RCC – Ernie to Richard – Re- Re-Send – 16 September 2006	SCE.028.0001
221.	Email chain: to Ernest Schrader et al from Robert Montalvo re: Paradise – dam – Cores – Ernie – 23 March 2006	SCE.029.0001
222.	Email chain: to Ernest Schrader et al from Jose Lopez re: Burnett – Core Tests- Ernie Follow-up – 4 January 2007	SCE.030.0001
223.	Email chain: to Robert Montalvo et al from Ernest Schrader re: Core Hole in RCC – Ernie Response – 13 September 2006	SCE.031.0001
224.	Email chain: to Richard Herweynen from Robert Montalvo re: Core Hole in RCC – Ernie to Richard – Re- Re-Send – 18 September 2006	SCE.032.0001
225.	Memo – More Comment Concerning Goodnight Bed for RCC by Ernest Schrader – 22 February 2004	HYT.001.0001
226.	Presentation – RCC Quality and High Productivity – November 2004	LOJ.001.0001
227.	Statement of Dr Shayan Maleki – 11 March 2020	MAL.002.0001
	Curriculum Vitae of Shayan Maleki	MAL.001.0001

Exhibit No	Description	Document ID
228.	SunWater Limited, Paradise Dam, Hydraulic Modelling Review and CFD Model Development	GHD.041.0001
229.	USBR - Design of Small Dams (3rd ed, 1987)	BOR.001.0001
230.	Paradise Dam Flood 2010/11 Damage Inspection and Civil Works Rectification Report	DNR.006.3156
231.	Team 1 Proposal Document – Design Report	DNR.007.0477
232.	Memo re: Results of Peer Review of Preliminary Design – 27 January 2004	DNR.010.0929
233.	Issues regarding potential for erosion downstream of Spillways (19 February 2004)	DNR.005.3464 (at 3619)
234.	Paradise Dam Facility Strategy & Options Analysis: Preliminary Business Case - Supporting Technical and Environmental Review (GHD, March 2018)	IGE.033.0001
235.	USBR - Hydraulic Design of Stilling Basins and Energy Dissipators	PDI.064.0001
236.	Sunwater Spillway 2D model study amended – April 2003	SUN.018.026.5653
237.	URS Paradise Dam Spillway Damage - Independent Technical Review – 9 October 2014	SWA.512.001.0578
Tender list 11 – Exhibit K		
238.	Transcript of Interview with Mr Eric Lesleighter – 21 February 2020	TRA.540.019.0001
239.	Email from Alexander McKinnon to Brock Morgan re: Lesleighter hydraulic scales – 29 February 2020	LEE.002.0001
240.	Memo from Brett Collins to Richard Herweynen and Andreas Neumaier – Issues regarding potential erosion downstream of Spillways – 19 February 2004	DNR.020.019.3164
241.	Statement of Mr Christopher Dann – 9 March 2020	DAC.001.0001
242.	Burnett Alliance Dam Design Expert Review Panel Report – 27 June 2003	DNR.003.8615
Tender list 12 – Exhibit L		
243.	Memo – Preliminary Assessment of Erodibility of Goodnight Beds under Primary Spillway Dissipator Apron – 24 November 2003	GOL.005.0001

Exhibit No	Description	Document ID
244.	Statement 1 of Mr Richard Herweynen – 12 March 2020	HER.001.0001
245.	Statement 2 of Mr Richard Herweynen – 12 March 2020	HER.002.0001
246.	Curriculum Vitae of Richard Herweynen	HYT.600.006.0001
247.	Transcript of Interview with Mr Richard Herweynen – 13 February 2020	TRA.510.007.0001
248.	US Army Corps of Engineers, Engineering and Design, Gravity Dam Design, Engineer Manual – 30 June 1995	HER.003.0001
249.	Burnett River Dam – Invitation to submit a Registration of Interest – January 2003	HYT.006.004.2793
250.	Burnett River Dam - Stage 1 – Request for Proposals – March 2003	SWA.500.001.2366
251.	Burnett River Dam Alliance – Vol 1 Response to Selection Criteria – Stage 1 – Proposal – 11 April 2003	HYT.510.004.0001
252.	Burnett River Dam – Stage 2 – Request for Proposals – May 2003	SWA.500.001.2068
253.	Burnett River Dam - Consortium Agreement – 1 August 2003	HYT.520.005.0001
254.	Burnett River Dam Consortium Agreement – 23 May 2005	HYT.522.002.0019
255.	Burnett River Dam Alliance – Design Management Plan	HYT.514.006.0293
256.	Professional Service Contract – Project No 114629 – Contract No 901384	HYT.505.004.0147
257.	Professional Service Contract – Project No 114629 – Contract No 901383	HYT.510.003.0049
258.	Email to Maurice Rimes from Richard Herweynen, re: Burnett Dam – Subconsultancy for RCC Expertise – 12 January 2004	HYT.510.003.0165
259.	Burnett River Dam - Review of Detail Design Report, Mappo Consulting – 6 August 2004	HYT.519.003.0008
260.	Emails referred to in paragraph 80 of Statement 1 of Richard Herweynen – 12 March 2020	DNR.013.7263
261.	Burnett River Dam – Inspection Onsite – 14 July 2004	HYT.519.003.0021

Exhibit No	Description	Document ID
262.	Email to Tim Griggs et al from Richard Herweynen re: Secondary Spillway Dissipator Apron Width – 10 December 2003	DNR.020.019.2052
263.	Note to file - Conventional Concrete in Dam – Richard Herweynen – 18 November 2003	DNR.020.019.2656
264.	Note to file - RCC Dissipator Slab for Spillway 2003 – Richard Herweynen – 10 November 2003	DNR.020.019.1888
265.	Presentation 14 August BDA Stage 2 – 14 August 2003	HYT.502.006.0003
266.	Note to file - Cement for RCC Mix 2004 – Richard Herweynen – 28 January 2004	DNR.020.019.1037
267.	Email to Sue Cumow from Richard Herweynen, re: Progress Report on RCC Testing 2003 – 4 November 2003	DNR.010.0323
268.	Email to Richard Herweynen from David Brett re: Cementitious Material Report 2003 – 12 November 2003	DNR.010.0317
269.	Memo - Cement and Ash Comparison to Thiess Mix Results – 25 January 2004	HYT.509.004.0105
270.	Stage 3 Stability Analysis	DNR.020.015.1040
271.	RCC Aggregate and Mix Verification Tests 2004	DNR.011.1436
272.	File note: Verification RCC mix Program – Richard Herweynen – 8 April 2004	DNR.020.019.1005
273.	Report 30.15.3.19 – R1 - Proposal for RCC Trial Embankment 2003 – 25 November 2003	DNR.020.016.5720
274.	Emails referred to in paragraph 177 of Statement 1 of Richard Herweynen – 12 March 2020	DNR.013.7361
275.	Placement of RCC BDA QA Check 0060 - Rev 10 - Lot No: RCCS – 232 – 17 February 2005	SUN.113.005.0202
276.	NCR 82 BDA RCCR 040 2004 – 5 August 2004	SUN.021.001.6730
277.	Letter to John Hunt from Jose Lopez and Robert Montalvo – Stockpile Aggregate Handling – 19 June 2004	DNR.020.019.1068
278.	Draft Agenda for Update Meeting with Regulator 2004	DNR.020.019.2606

Exhibit No	Description	Document ID
279.	Memo - Design Sign-off Prior to Impoundment – 5 October 2005	SUN.126.001.0001
280.	Comparison of results of a USBR Type I and Type II dissipator	HYT.006.004.5331
281.	Report of Burnett River Dam Design Inspection and Test Plan for the Basalt Pimple – August 2005	DNR.010.2038
282.	Burnett River Dam, RCC Sunwater Presentation – 21 September 2004	SUN.020.003.6480
283.	Transcript of Conclave with James Willey and Tim Griggs on 17 March 2020	PDI.090.0001
	Conclave Exhibit 1 – Photograph of diagram drawn on whiteboard by Mr Willey	
	Conclave Exhibit 2 – Photograph of diagram drawn on whiteboard by Mr Griggs	
	Conclave Exhibit 3 – Photograph of drawing or dam/foundation interface on whiteboard by Mr Griggs	
	Conclave Exhibit 4 – Letter from GHD to Commission, re: friction only calculations – 16 March 2020	
	Conclave Exhibit 5 – Placement of Roller Compacted Concrete DBA-QA-CHK-0060 Rev 9 Lot No RCC-109 – 16 November 2004 (DNR.002.014.4624 at .4629)	
	Conclave Exhibit 6 – Placement of Roller Compacted Concrete DBA-QA-CHK-0060 Rev 9 Lot No RCC5-656 – 12 October 2004 (SUN.114.001.0057)	
	Conclave Exhibit 7 – RL32.395 Lift Surface @ ~ Ch.361	
	Conclave Exhibit 8 – Page 16 of article 'Shear Strength and lift joint quality of RCC' – E.K. Schrader -1999 (PDI.040.0001 at .0016)	
284.	Assumptions in the Analysis of Sliding Stability for Conclave on 17 March 2020	PDI.089.0001
285.	Email chain to R. Herweynen from E. Schrader – re: RFI on RCC Density Testing – 1 September 2004	HYT.002.0001
286.	Statement of Mr Timothy Griggs – 12 March 2020	GRT.001.0001

Exhibit No	Description	Document ID
287.	Curriculum vitae of Timothy Griggs	HYT.600.005.0001
288.	Transcript of Interview with Mr Timothy Griggs – 14 February 2020	TRA.510.009.0001
289.	Memo from Richard Herweynen to Tim Griggs –'Tim Griggs' role on site' – 13 March 2004	DNR.020.019.2601
290.	Flow charts for foundation acceptance	SUN.120.001.0045
291.	As-built survey drawings	HYT.600.005.0008
292.	Dam foundation inspection and test plant – 22 February 2005	SUN.120.001.0102
293.	Photograph of bedding mix placement – 17 November 2004	HYT.600.003.0001
294.	Spreadsheet summary of bedding mix calculations – 15 December 2004	HYT.600.005.0007
295.	Review by Tim Griggs of bedding mix location	HYT.600.008.0001
296.	Results of friction only calculation sought by Commission	HYT.600.008.0003
297.	Design Management Plan (during design phase)	SUN.162.002.0149
298.	Supplemental Statement 2 of Dr Ernest Schrader – 16 March 2020	SCE.033.0001
299.	Placement of Roller Compacted Concrete DBA-QA-CHK-0060 Rev 7 Lot No RCC-031 – 25 September 2004	SUN.114.001.0273
Tender list 13 – Exhibit M		
300.	ICOLD Bulletin 135 - Geomembrane sealing systems for dams	HYT.003.0001
301.	GHD - SunWater Limited, Paradise Dam Spillway Improvement – Preliminary Design Update of Comprehensive Risk Assessment – January 2019	GHD.021.0001
302.	Transcript of Interview with Mr Andreas Neumaier – 24 February 2020	TRA.510.021.0001
303.	Peer Review April 2004 file	DNR.020.021.6529
304.	Burnett Dam Alliance, Expert and Peer Review Workshop, RCC – Mix Design and Laboratory – 19 January 2004	GHD.043.0001
305.	Report on Peer Review Workshops	SUN.018.005.2929
306.	Memo, Design Sign-Off for Practical Completion – 25 November 2005	DNR.005.0584 (at .0828)

Exhibit No	Description	Document ID
307.	Article – 'Using Sloped Layers to Improve RCC Dam Construction" – HRW Magazine July 2003 – Brian A Forbes	SME.001.0001
308.	Letter Johnson Winter & Slattery in response to Requirement to Produce documents relating to 'Construction Report – 13 March 2020, along with RCC Trial Section Construction Report No. 1 July 2004 – SUN.114.003.0001	MCM.016.0001
	Draft – Plan for Preparation of Construction Report	MCM.009.0001
	Spreadsheet – Overview of Dam Safety Condition	MCM.010.0001
	RCC Trial Section Construction	MCM.011.0001
	Draft – Plan for Preparation of Construction Report	MCM.012.0001
	Draft – For Comment Only – Construction Report	MCM.014.0001
	Document Delivery Matrix	MCM.015.0001
309.	Transcript of Interview with Mr Mark Hamilton – 24 February 2020	TRA.510.022.0001
310.	Placement of RCC BDA QA Check 0060 - Rev 10 - Lot No: RCCS – 231 – 16 February 2005	SUN.021.006.1405
311.	Placement of RCC BDA QA Check 0060 - Rev 10 - Lot No: RCCS – 297 – 29 March 2005	SUN.021.006.2078
312.	Project Alliance Board Meeting Minutes No 7 – 13 February 2004	SUN.018.005.2485
313.	Project Alliance Board Meeting Minutes No 11 – 16 April 2004	SUN.018.005.0938
314.	Project Alliance Board Meeting Minutes No 14 – 22 July 2004	SUN.018.004.8033
315.	Project Alliance Board Meeting Minutes No 19 – 21 December 2004	SUN.018.006.4485
316.	Project Manager's Overview PowerPoint – October 2004	SUN.279.003.0005
317.	Schedule Performance Presentation	SUN.018.006.5066
318.	Project Manager's Overview PowerPoint – December 2005	SUN.020.001.1741
319.	Memo – Miscellaneous Comments, Production – Membrane Punctures – Strain gauges – 16 November 2004	DNR.011.1382

Exhibit No	Description	Document ID
320.	Statement of Mr Chris Nielsen – 17 March 2020	NIC.001.0001
321.	Letter from Johnson Winter & Slattery, in response to letter requesting documents referred to in evidence of Bruce Embery – 13 March 2020	MCM.018.0001
322.	Email Bruce Embery to Samantha Amos – re: Additional Documents mentioned in Evidence given on 11 March	MCM.017.0001
323.	Guidelines on Acceptable Flood Capacity for Water Dams – December 2019	HYT.600.008.0005
324.	Statement of Mr Jose Lopez – 21 March 2020	LOJ.003.0001
325.	Design Assumptions – GHD response – 20 March 2020	GHD.045.0001
326.	Review of piezometer readings by Tim Griggs – 19 March 2020	HYT.004.0001
327.	Email chain: to Frank Sutton, from Ernest Schrader re: Rotec – Schrader Comments Late but URGENT – Steve Read also – 26 November 2003	SUN.010.002.0292
328.	Statutory Declaration of Mr William Curlewis – 19 March 2020	CUW.001.0001
329.	Email from Denise Obst to Commission re: Additional Calculations from James Willey – 20 March 2020	GHD.046.0001
330.	Paradise Dam Stability Calculation by Tim Griggs – 19 March 2020	HYT.005.0001
331.	Discussion Paper – Governance: regulatory scheme and institutional structure – 13 March 2020	PDI.092.0001
332.	Email to Commission from Ernest Schrader re: Schrader – PDI, Stability Analysis for Friction – 20 March 2020	SCE.034.0001

Appendix 10 – Paradise township

The town of Paradise, founded in 1888, stretched for more than a kilometre along the southern bank of the Burnett River. It was to have a short life, abandoned less than a decade later in 1896. Paradise was settled to mine gold, and no European structures or settlement had previously been there. At its peak, some 600 people dwelt there.[1]

The town owed its existence to a gold reef on the eastern bank of the Burnett River, extending from Paradise Creek to Finney's Creek. The discovery of the gold deposit by brothers James and Thomas Allen in 1888 led to the official proclamation of the Paradise Goldfield in November 1890.

Some say the town had a Victorian order and deliberateness: the town survey carried out in 1891 shows a regular, grid-like organisation of its streets and allotments. There was a school, and major buildings along its main thoroughfare Allen Street (named after the town's European founders and the discoverers of gold there). The town had numerous hotels, a community hall, post office, police station, courthouse (the top floor of which is now the Biggenden Historical Museum), at least two butchers, three carpenters, and seven mining companies, as well as a lemonade factory, a sawmill, and any number of grocers, tobacconists, drapers, dressmakers, dentists, shoemakers and stationers.[2]

Gold yields declined rapidly in the late 1890s. The town had ceased to exist by the early 20th century. Most of the buildings were removed to other settlements in the surrounding district, including Biggenden, Mount Shamrock and Mount Perry. The town reserve was not de-gazetted until 1939 and remnants of the town were excavated and salvaged by the University of Queensland before that occurred.[3] Structures and artefacts found at the site were transferred to Biggenden. They were studied in some detail by historians and archaeologists at the University of Queensland, including Ms Kate Quirk.[4]

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1 At its height, Paradise boasted a population of over 600: Queensland Registrar-General, *1891-1900 Statistics of the colony of Queensland*, Government Printer, Brisbane, 13.

2 The preceding paragraphs contain information drawn from the work of Kate Quirk, 'The Colonial Goldfields: Visions and Revisions' (2008) 26 *Australian Historical Archaeology* 13 <http://www.asha.org.au/pdf/australasian_historical_archaeology/26_04_Quirk.pdf> accessed 20 December 2019, and Jonathan Pragnell and Kate Quirk, 'Children in Paradise: Growing Up on the Australian Goldfields', (2009) 43 *Historical Archaeology*, 38.

3 Prangnell, J. M., Cheshire, L. A., Quirk, K. A., *Paradise: Life on a Queensland Goldfield* (2005); Jonathan Pragnell and Kate Quirk, 'Children in Paradise: Growing Up on the Australian Goldfields', (2009) 43 *Historical Archaeology*, 38.

4 Quirk, K., 'The Victorians in 'Paradise': Gentility as Social Strategy in the Archaeology of Colonial Australia', PhD Thesis, University of Queensland, December 2007.

Why this small field was proclaimed 'Paradise' is unclear. Some sources suggest that the name comes from a nearby pastoral run, but no evidence of such a run has ever been found.[5] Another account relates the naming of the town to a shepherd's hut and garden which existed before proclamation of the goldfield. The extent and beauty of this garden was such that when visitors came to the Degilbo station, the owner would take them to view the site, to which they would exclaim '*Oh, what a Paradise*'.[6]

5 Prangnell, J. M., Cheshire, L. A., Quirk, K. A**,** *Paradise: Life on a Queensland Goldfield* (2005), 8.

6 The Queensland Christian Witness and Methodist Journal, 1 July 1891, 7.

Glossary of terms

Term	Meaning
Abutment	The side of a river valley against which a dam is constructed. Right and left abutments are those viewed when looking downstream.
Adopted Middle Thread Distance	The distance, in kilometres, measured along the middle of a watercourse that a specific point lies from the mouth of that watercourse (or from junction with another watercourse).
Alluvium	Sediment deposited by flowing water, such as in a riverbed.
Annual exceedance probability (AEP)	The probability that a given rainfall total accumulated over a given duration will be exceeded in any one year, causing a flood event, with the probability being expressed as a percentage.
Appurtenant structure	Supplementary or ancillary features of a dam such as outlets, spillways, powerplants and tunnels.
Apron	Concrete lining downstream of a spillway which offers protection against erosion of the underlying natural materials.
Aquifer	An underground layer of rock, sediment or soil whose voids are filled or saturated with water.
Australian Height Datum (AHD)	The official national vertical datum for Australia which refers to Australian Height Datum 1971 (AHD71: Australian mainland) and Australian Height Datum (Tasmania) 1983 (AHD-TAS83). The datum surface passes through the approximate mean sea level realised between 1966 and 1968 at tide gauges around the Australian coastline.
Axis of dam	The vertical plane or curved surface, chosen by a designer, appearing as a line, in plan or in cross-section, to which the horizontal dimensions of the dam are referenced.
Basalt Pimple	The term used to describe a very large piece of basalt located in the Burnett River and left in place as an inclusion in the Paradise Dam.
Breach	An opening through a dam that allows reservoir draining; controlled breach is an intentionally constructed opening; and uncontrolled breach is the unintended failure of the dam.
Cofferdam	An enclosure built within or across a body of water, such as a dam, to allow the enclosed area to be pumped out, creating a dry environment allowing construction or repair work to be carried out.
Commissioning	The process of managing all activities required to verify and document the compliance, performance, functionality and transitioning to operation of new, renewed or modified assets.

Term	Meaning
Curtain grouting	The process of pressure grouting deep holes under a dam or in an abutment to form a watertight barrier and effectively seal seams, fissures, fault zones, or fill cavities in the foundation or abutment.
Dyke	(In Geology): A tabular body of igneous rock (generally formed from magma intrusions) that cuts across a structure of adjacent rocks or cuts through massive rocks.
Earth dam	An embankment dam in which more than 50% of the total volume is formed of compacted earth layers.
Embankment dam	Any dam constructed of excavated natural materials, such as both earthfill and rockfill dams, or of industrial waste materials, such as a tailings dam.
Failure mode	A potential failure mode is a plausible process for dam failure resulting from an existing inadequacy or defect which can lead to an uncontrolled release of the reservoir.
Flood of record	Flooding that equals or exceeds the highest stage or water discharge at a given site during the period of record keeping.
Flyash	The finely divided residue resulting from the combustion of ground or powdered coal.
Gravity dam	A dam resisting the pressure of impounded water through its own weight, and which by its weight alone is great enough to prevent it from sliding or tipping over.
Grout	A fluid paste material injected into soil, rock, concrete or other construction material to seal openings and to lower the permeability and/or provide additional structural strength.
Headwater	The water level at the upstream face of a dam. The head is the point on a watercourse up to which it has been artificially changed by an impoundment.
Heel	The junction of the upstream face of a dam with the foundation surface.
Hydrology	The branch of science concerned with the properties, distribution and circulation of the Earth's water, and especially its movement in relation to land, both on and below the surface and in the atmosphere.
Impoundment	The filling of a dam resulting in a body of water commonly known as a reservoir.
Kilopascal	A unit of stress or pressure, where 1 kPa equals 1000 Pa or 1000 times a force of 1 newton per square metre (that is 1 Pa equals 1 kilogram per metre per second squared).
Large dam	A dam with a height of 15 metres or greater from lowest foundation to crest, or a dam between 5 metres and 15 metres in height impounding more than 3 million cubic metres of water.

Term	Meaning
Lithology	Description of the physical characteristics of rock units visible at outcrop, in hand or core samples, or with low magnification microscopy. Physical characteristics include colour, texture, grain size, and composition.
Megalitre	A unit of measurement of volume: 1 million litres.
Monolith	An individual column of RCC or CVC constructed to form part of the spillway and the abutments of a gravity dam.
Ogee spillway	An overflow spillway, which in cross section shows ogee form of curve, i.e., separate convex and concave section, like the letter 'S'.
Peak flow	The maximum instantaneous discharge that occurs during a flood. It is coincident with the peak of a flood hydrograph.
Piezometer	An instrument for measuring the pressure of a liquid or gas.
Probable maximum flood	Flood resulting from probable maximum precipitation coupled with the worst catchment conditions that can realistically be expected.
Probable maximum precipitation	The greatest depth of precipitation for a given duration that is physically possible over a given size storm area at a particular geographical location during a certain time of the year.
Pug mill	A machine used to mix construction materials such as concrete.
Reservoir	Body of water impounded or potentially impounded by a dam.
Reservoir surface area	The area covered by a reservoir when filled to a specified level.
Riprap	A layer of large stones, broken rock, or precast blocks placed on a slope as a protection against wave action, erosion, or scour.
Risk assessment	The process of deciding whether existing risks are tolerable and present risk control measures are adequate and, if not, whether alternative risk control measures are justified. Risk assessment incorporates the risk analysis and risk evaluation phases.
Rockfill dam	An embankment dam in which more than 50% of the total volume is comprised of compacted or dumped cobbles, boulders, rock fragments, or quarried rock generally larger than 7.5 centimetres in size.
Scour	Erosion damage to structural and/or geological materials caused by fast-moving water. Scour effects increase with increased flow velocity and turbulence and with increasing erodibility of soil or rock.
Shear strength test	A test measuring a material's ability to resist forces that cause the material to slide against itself. The test determines the maximum shear stress that the material can withstand before failure occurs.

Term	Meaning
Shotcrete	A construction technique whereby concrete or mortar is conveyed through a hose and pneumatically projected at high velocity onto a surface, usually a vertical or overhead surface.
Slag	A glass-like material, usually a by-product produced when a metal has been separated (i.e. smelted) from its ore.
Slump (of concrete)	The result of a test designed to assess the consistency of concrete, i.e., the workability of fresh concrete before it sets.
Spillway	A structure over or through which flood flows are discharged. If the rate of flow is controlled by mechanical means, such as gates, it is considered a controlled spillway. If the geometry of the spillway is the only control, it is considered an uncontrolled spillway.
Spillway capacity	The maximum spillway outflow that a dam can safely pass with the reservoir at its maximum level.
Spillway channel	An open channel or closed conduit conveying water downstream from the spillway inlet.
Spillway crest	The lowest level at which water can flow over or through the spillway.
Tailwater	Body of water immediately downstream of the dam at any point in time.
Toe	The junction of the downstream face of a dam with the ground surface.
Toe drain	A system of pipes and/or pervious material along the downstream toe of a dam used to collect seepage from the foundation and embankment and convey it to a free outlet.
Training wall	A wall built to confine or guide the flow of water.
VeBe test	A test allowing for the evaluation of freshly mixed concrete, measuring its workability by determining the characteristics of mobility and compactibility. The test measures the relative effort required to change the mass of the concrete from the conical shape to the cylindrical shape by undergoing vibration process. The measurement is by time, in seconds.
Void-free matrix	A mixture of concrete that is free of any air trapped in the mixture which would cause a void once concrete hardens.
Weir	Low dam across a stream to raise the upstream water level; a notch of regular form through which water flows.
Windrow	A row of material, such as mown hay or deposited concrete.

List of acronyms

Acronym	Meaning
ACI	American Concrete Institute
AEP	Annual exceedance probability
AHD	Australian height datum
ANCOLD	Australian National Committee on Large Dams
BDA	Burnett Dam Alliance
CFD	Computational fluid dynamics
CVC	Conventional concrete
DEWS	Department of Energy and Water Supply
DNRME	Department of Natural Resources, Mines and Energy
DSDMIP	Department of State Development, Manufacturing, Infrastructure and Planning
EIS	Environmental Impact Statement
EL	Elevated level
EPRI	Electric Power Research Institute
FERC	US Federal Energy Regulatory Commission
FIA	Failure Impact Assessment
FoS	Factor of safety
FSL	Full supply level
GOC	Government Owned Corporation
HCRCC	High Cementitious Content Roller Compacted Concrete
ICOLD	International Committee on Large Dams
IGEM	Inspector-General Emergency Management
IPA	Integrated Planning Act
ITP	Inspection and Test Plan
kPa	Kilopascal
LCRCC	Low Cementitious Content Roller Compacted Concrete
LJQI	Lift Joint Quality Index
MCRCC	Medium Cementitious Content Roller Compacted Concrete
ML	Megalitre
MPa	Megapascal
NCR	Non Conformance Report
PMF	Probable Maximum Flood
PMPDF	Probable maximum precipitation design flood
QA	Quality Assurance

Acronym	Meaning
QC	Quality Control
RCC	Roller compacted concrete
RPEQ	Registered Professional Engineer of Queensland
TRP	Technical Review Panel
USACE	US Army Corps of Engineers
USBR	US Bureau of Reclamation

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Commission of Inquiry
PARADISE DAM

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