FILE
DAM/130/000(1997)

Ryan Dave

From: David Murray [David.Murray@burnettwater.com.au]
Sent: Monday, 8 September 2003 9:01 AM
To: Allen Peter; Flatley Glenn
Cc: Ryan Dave
Subject: RE: Burnett R Dam

NTAR 17_Response.doc Letter BW NTAR 30 04.09.03.doc...

Peter/Glenn

I attach the response from both teams to your list of questions. If you wish to discuss this further please call.

Thanks again for your timely response to our "application".

Dave Murray

-----Original Message-----
From: Allen Peter [mailto:Peter.Allen@nrm.qld.gov.au]
Sent: Tuesday, 2 September 2003 9:45 AM
To: Flatley Glenn
Cc: David Murray; Ryan Dave
Subject: FW: Burnett R Dam

Glen,

I have attached several questions for Burnett Water over their development applications for Burnett River Dam. Could you please incorporate them in an information request to Burnett water?

I suspect the answers to some of them will be that they will be addressed (and reported on) in the final design report.

As you can see, I have also sent a copy to Dave Murray for his information.

If you have any questions, give me a call.

Peter Allen
Director, Dam Safety (Water Supply)
Water Industry Compliance
12th Floor Mineral House
(Work phone) 3224 7636,
(Mobile) 0418 728 755
Fax 3224 7999
email peter.allen@nrm.qld.gov.au

> -----Original Message-----
> From: Ryan Dave
> Sent: Monday, 1 September 2003 5:34 PM
> To: Allen Peter
> Subject: Burnett R Dam
>
> > <<QuestionsForBurnettAlliance.doc>>

TEAM 1
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8 September 2003

Burnett Water Pty Ltd
Level 3
307 Queen Street
BRISBANE QLD 4001

Attention: Frank Sutton

Dear Frank,

Burnett River Dam – NTAR No 30

Our response to NTAR No. 30 is as follows:

Hydrology

Team 1 has adopted the Failure Impact Assessment Report prepared by SunWater. Team 1 did not undertake additional dambreak or consequence assessment during the Stage 2 process. As part of the final design phase, the failure impact assessment study will be extended and refined, forming the basis of the Acceptable Flood Capacity of the dam and feeding into the Emergency Action Plan for the dam.

Team 1 recognises that the Failure Impact Assessment produced by SunWater only extended to Wallaville, approximately 40km downstream of the dam. To determine the Acceptable Flood Capacity for the dam, the Population at Risk (PAR) needs to be calculated for the Flood Affected Zone (ANCOLD Guidelines on Assessment of the Consequence of Dam Failure, May 2000). According to the ANCOLD Guidelines on Risk Assessment (1994) the Flood Affected Zone is defined as that zone of flooding where the changes in depth and velocity of flooding due to dambreak are such that there is potential for **incremental** loss of life. As indicated in the SunWater Failure Impact Assessment Report, the incremental PAR for a sunny day failure is 235 for the 1:100 AEP flood and zero for the PMF.

Team 1 believes that a large portion of the population downstream of Wallaville will be underwater prior to the PMF and therefore the incremental PAR will be significantly less than the total population within the flood plain. Based on this assessment Team 1 does not believe the PAR would be greater than 1000, but would be in the range of 100-1000. This will need to be confirmed with a more detailed assessment during the detailed design phase.

Based on a PAR between 100 and 1000, the Burnett River Dam is given a High A Hazard Rating based on Table 3 in the ANCOLD Guidelines on Assessment of Consequence of Dam Failure, May 2000. According to Table 8.1 of the ANCOLD Guidelines on Selection of Acceptable Flood Capacity for Dams (March 2000), the Fallback Flood Capacity for a High A dam is the PMP Design Flood. Team 1's design can safely pass the PMP Design Flood of 94,861 m^3/s, without putting the dam at risk of failure.

Spillway Hydraulics

The lake level values quoted in Section 4.3 – Team 1's Initial Arrangements Investigated, were for initial spillway arrangements only. During this initial investigation of options stage we had an ogee shaped crest for the Secondary Spillway and a primary spillway, which was 300m long. The final arrangement adopted had:

- A broad crested shape for the secondary spillway to enable access to the outlet works.

- A 315m long primary spillway, with 7 x 45m wide monolith blocks, which worked well with the proposed conveyor system for RCC delivery.

The rating curve developed for our final arrangement has been verified with a 3-dimensional, physical hydraulic model (Refer to Section 4 – Volume 2).

The flood routing results presented in Section 3.2.6 of Volume 2 are the results for the final spillway arrangement. The results presented in this section are as follows:

AEP	*Description*	*Inflow*	*Outflow*	*Max Lake Level*
1:10,000	N/A	50,328 m³/s	49,285 m³/s	EL 82.60m
1:30,000	PMP Design Flood	94,861 m³/s	93,457 m³/s	EL 87.46m
N/A	PMF	106,863 m³/s	104,377 m³/s	EL 88.45m

As indicated in Section 5.2.1, the Extreme loading for the dam stability assessment was the PMP Design Flood, which was a headwater level of EL 87.5m. Based on the stability results, this is clearly not the worst load case and the dam would be stable for the PMF, with a headwater level of EL 88.5m. As indicated in Team 1's submission, we believe one of the key advantages of our design is that it has a certain level of immunity to changes in the design flood (ie. "PMP Immune").

Foundation

Team 1 has carried out a significant amount of investigation and analysis to arrive at the position of incorporating the basalt flow on the right abutment into the foundation. The analysis carried out included:

- Block stability analysis both during construction and for the completed dam. The analysis undertaken has demonstrated that the proposed design is stable. The basalt flow on the western side of the diversion channel has included a number of anchors to provide adequate stability.
- Seepage analysis to determine potential leakage, which indicated that the seepage is small and acceptable for a dam of this height.
- Finite element analysis to determine differential settlement. This analysis indicated that the differential settlement was negligible.

Therefore it is Team 1's opinion that the sub-basaltic alluvium will not adversely impact on the serviceability and stability of the dam.

Team 1 has made a number of design provisions to accommodate the potential compressibility of the sub-basaltic alluvium, namely:

- Location of a monolith joint at the nose of the basalt flow, which will allow small amounts of differential settlement. The membrane is folded over itself at this monolith joint to allow for some movement.
- The alluvium under the nose of the basalt flow will be excavated to the depth of its thickness and replaced with backfill concrete.
- The alluvium seam exposed by the diversion excavation cut, although very thin at this point, will be treated as indicated on the drawings.
- The design also has grouting specified in this area which will increase the stiffness of the alluvium.

The stability analysis on the basalt block was based on a water level of EL 72m and a factor of safety of 2.0. As indicated in the dam stability analysis (Section 5 of Volume 2), the worst load case for sliding stability was the less extreme floods. The reason for this is that the tailwater associated with the more extreme events provided a restoring force. Therefore a headwater level of EL 72m, without any tailwater, was considered to be a likely worst case scenario in terms of sliding stability of the rock block. As part of the detailed design, the stability of the basalt block will be assessed for a range of reservoir levels. For more extreme flood events a factor of safety less than 2.0 would be acceptable.

The uplift pressure under the basalt block was assumed to vary linearly from full uplift head at the upstream point to zero at the downstream point. The linear reduction in uplift was confirmed by the seepage analysis that was carried out.

zometers have been specified to be installed in the sub-basaltic alluvium, which will provide a basis for validating our design assumptions. In the unlikely event that the uplift pressures measure higher than assumed, it would be relatively easy to drill drain holes that are angled from the downstream toe to intercept the alluvium material.

Seismic Loading

The Seismology Research Centre prepared a Review of Seismicity Report for the Burnett River Dam (September 2002). This report was used as the basis for obtaining Peak Ground Acceleration (PGA) for various Annual Exceedence Probability (AEP) events.

The ANCOLD Guidelines for Design of Dams for Earthquake recommends that the design earthquake load (MDE) be determined by consideration of the hazard category of the dam, and is selected as an earthquake for a given AEP. The ANCOLD Guideline do not specifically give an AEP for which a certain hazard dam should be designed for, however, Table 4 provided in these guidelines does provide some guidance. For example the Canadian Dam Safety Association recommends a MDE with an AEP of 1 in 5,000 to 1:10,000 for very high consequence dams. Based on the guidance provided in these guidelines, Team 1 adopted a 1:10,000 AEP earthquake for the MDE.

The dam, appurtenant structures and equipment should remain functional and damage from the occurrence of earthquake shaking not exceeding the Operating Basis Earthquake (OBE) should be easily repairable. It is fairly common for an OBE of 1:500 AEP being adopted for a dam, which is what Team 1 has adopted for Burnett River Dam.

You can achieve high PGAs for low magnitude earthquakes if it is a shallow near field earthquake. However, it is observed that earthquakes smaller than about magnitude 5 rarely cause any damage. Therefore for dams it is common practice to only consider PGAs resulting from magnitude 5 and above. Based on the Review of Seimicity Report, the PGAs for the 1:10,000 AEP and 1:500 AEP earthquakes, with magnitude 5 and above, are 0.2g and 0.035g respectively.

Team 1's response to NTAR No. 18 should also be read in relation to seismic loading.

Load Combinations

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

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.

Tailwater Level

Our adopted tailwater levels in the stability analysis were as follows:

- 1:1,000 AEP flood, HWL 78m, TWL 60m
- 1:10,000 AEP flood, HWL 83m, TWL 70m
- PMP Design Flood, HWL 87.5m, TWL 78.0m

Team 1 recognises that the tailwater level for these extreme flood events is not certain, as the tailwater levels adopted are based on extrapolating a computer model, which was calibrated against smaller historical floods. For this reason Team 1 undertook a sensitivity analysis of the tailwater level on the stability analysis. This analysis indicated that all criteria would be meet even if the tailwater level were 3m lower than that adopted.

No analysis has been carried out in relation to the tailwater level in association with hydrodynamic effects of the plunging jet. However, our proposed design had both a 1:100 scale 3-dimensional model and a 1:75 scale 2-dimensional model for us to measure and assess the hydraulic conditions of our design. In the 2-dimensional model we had pressure tapings installed so that we could measure the pressure profile on the spillway crest. We also had five (5) pressure tapings installed on the downstream face and dissipator apron to measure the hydrodynamic effects. Two pressure tapings on the vertical face of the steps (one at EL 54 and the other at EL 32), three pressure tapings in the dissipator slab (2m, 10m and 18m downstream of the bottom step). The tailwater pressures measured on the 2-dimensional hydraulic model are given in Table 12.3, Section 12 – Volume 2. The HWLs closest to the design loads, that we have pressure measurements for, is EL 77.7 and EL 84. The measured values from the hydraulic model study are summarised in the table below:

HWL	***Adopted TWL***	***Measured Tailwater Level based on Pressures Measured on Hydraulic Model***				
		@ EL54	***@ EL32***	***2m d/s***	***10m d/s***	***18m d/s***
EL 77.7	Approx. 60m	EL 55.2m	EL 63.7m	EL 64.8m	EL 59.2m	EL 65.3m
EL 84.0	Approx. 70m	EL 66.3m	EL 75.0m	EL 74.7m	EL 75.3m	EL 76.1m

Based on these readings, the tailwater pressures assumed in the stability analysis appear to be reasonable. The pressure distribution on the downstream face can be further refined with increased pressure tapings on the hydraulic model.

Outlet Works

Hydro Tasmania used its experience from an Operators viewpoint to ensure that issues relating to operations, maintenance and OH&S were incorporated into the Stage 2 design. In addition to this, Team 1 had a meeting with one of SunWater's operators to walk through our design and receive feedback in relations to operations, maintenance, and OH&S. Following this meeting some modifications were made to our design, to address issues raised.

As part of the Final Design Phase, Team 1 will be carrying out a HAZOP study to ensure that our final design is acceptable from an operations, maintenance and OH&S viewpoint.

The following provides some clarification of the arrangement of the 2.2m dia. valve and outlet works in general

1. The 2.2m diameter valve in the irrigation outlet pipe is almost redundant. Its purpose is to provide closure against flow in the event of catastrophic failure of the irrigation outlet conduit between the concrete plug and the irrigation valves isolation/guard valves.
2. The guard gate at the inlet to the irrigation conduit is of relatively light design and is located where it is for isolation of the full length of the irrigation conduit. It does not have closure into flow duty so that it and its lifting gear

requirements are minimal. The intent would be to have it stored out of the water and out of the way of normal operations within the tower and only used for inspection of the guard valve and the conduit downstream.

3. The air vent has been shown to indicate that consideration has been given to this requirement rather than sizing it for construction. Its sizing is for the benefit of *emergency* operation only of the fixed wheel gate. Its importance for minimizing vibration, cavitation and minimizing increase in gate downpull should be considered in this context. It would be the subject of more rigorous design during the design and construction phase.

4. Other comments:

   - The irrigation outlet conduit makes use of the diversion works as per Team 1 concept. The arrangement of the pipe conduit within that offers separate and isolatable irrigation and environmental releases outlets. Piped conduit for the full length provides:

     (i) an accessway (via this) into the downstream fishway culvert as it means that the culvert in which it runs is not flooded.

     (ii) a solution for attaching/ anchoring the irrigation valves and pipework in the longitudinal direction.

   - The outlet works have been arranged to minimize tower operational requirements to accommodate different storage levels (with the utilization of shutters instead of baulks) and to keep regular maintenance possible within the tower rather than gates/ shutters being taken away from site.

   - The reference made to review during the final design by Operations and Maintenance Engineers was carried out. There appeared to be general satisfaction with the concepts being submitted and comments made at that time were subsequently considered and addressed during the follow-up phase.

Should you require any further information, please do not hesitate to call.

Regards,

Mark Hamilton
Project Manager

Att: Copy of Burnett Water fax – NTAR 30