10 March 2020
Witness Statement (2) by Ernest Schrader

As explained in my Witness Statement 1, due to personal issues this past month I was unable to develop a witness statement earlier, but I also was not previously specifically asked to prepare one. I have been responding, as best I can by email to specific questions as they were raised by PDI, and have had a couple of conference calls both with PDI and Hydro Tasmania. I have also provided Hydro Tasmania and PDI with various self-initiated emails containing supplemental pertinent information. I prepared and submitted Witness Statement 1 on 9 March 2020. It primarily addressed shear properties and shear testing. This follow-up statement addresses miscellaneous other issues, some of which a related to the shear issue. Again, I apologize for the last minute "cut and paste" nature of this statement. It has not been reviewed by Hydro Tasmania or anyone else, nor has anyone advised me regarding what to include or say. It, therefore, also has not been fitted to any special format.

My CV and RCC experience are summarized in my Witness Statement 1.

I am currently under an agreement with Hydro Tasmania to assist the PDI, but have not been interacting with their Engineers or management. Only lawyers, and then just for administrative concerns. At the start of the Paradise project I was engaged first as a consultant by Walker Construction. Later my invoices were sent to the BDA project manager and then to Hydro Tasmania.

I emphasize again that if, in fact, the lift surfaces are unbonded where they need to be bonded, and if the friction angle is lower than required for stability, we have a serious issue that must be dealt with and corrected. However, we first need to ascertain with credibility the issues of in-situ bond and friction at lift joints. As detailed in my Witness Report 1, based on industry standards and comparison to other projects with similar mixes and materials it is highly unlikely that the friction angle is low. It is more likely that it is high. This should not be left to chance, opinion, or questionable test results. It needs to be credibly determined for the in-situ RCC.

There still seems to be some misuse of the terms "high paste" and "low paste" RCC. In reality, both types of mixes typically have about the same paste content at about 22%- 24%. Past is everything passing 75 micros. That is water, tiny air bubbles, aggregate fines, admixtures, and aggregate fines. Lean mixes simply use aggregate fines to supplement the portion of paste that is not made up of cementitious materials.

Mr. Dolan has expressed concern that the RCC lifts were not well compacted, that the lower portion (or more) of the lifts is porous and loose, and that the broken nature of many cores demonstrate this. I believe that, as typically happens with lean mix RCC, the raveling and broken nature at the bottom of lifts as seen in cores is a result of the coring operation and the mix not being able to withstand the rigors of drilling and handling. If a hard angular coarse aggregate is dislodged from the matrix as the drill crosses a lift line, it then tends to rotate around in the core barrel and damage the RCC. This causes more aggregates to break loose and exacerbates the situation. The same can happen in high cementitious content RCC, but to a greatly reduced extent due to the stronger nature of the matrix.

During construction thousands of density tests were made using the double probe nuclear gauge. Every lift was checked at multiple locations. At each location the density was determined from the surface downward at depth intervals of 50 mm. The average density at each test location was specified to reach a minimum of 96% of the theoretical air free density. This is where strength starts to drop with decreasing density. The average density for Paradise was 97.8% which is almost 99% of the maximum practical achievable density. The typical range of density at different depths in the lift varied from 96.9% to 98.2%. There were some instances of lower test density towards the bottom of some layers, but the material was re-rolled to acceptable density or removed and replaced. This was a very rare occurrence.

As discussed in my Witness Statement 1, the best way to resolve whether there is porosity in the RCC in-situ is to saw cut or excavate a trench into the RCC so that its real internal condition can be seen by all. Saw cutting and excavation typically cause minimal disturbance of lift joints in lean RCC, whereas coring definitely causes damage.

A “down the hole” camera can also be used to look inside the holes. This typically shows that the situation is better in the hole than in the core. The GHD report contends that the camera at Paradise showed similar results to the core, but this is not the case as anybody can see for themselves.

Inspection during construction was excellent, depending primarily on Jose Lopez and Roberto Montalvo. I emphasized to them the need to document any issues that arose, and to be sure they were always corrected. Apparently some people have seen photographs of what they consider to be examples of poor construction. For example, a place where there may have been segregation at the edge of an advancing lift or damage to the surface by trucks. However, these photos would have been primarily taken by QC personnel to document the issue, which would then have been corrected. An example is shown in the QC weekly report for 15-21 August. The photo below, taken from that report, shows damage to the lift surface by trucks.

PICTURE 2. Surface damage (RL 73.315) on access during placement Of 2nd layer to RL 73.625 - Nightshift

However, the following photo from the report shows the corrective action that was taken.

PICTURE 3. Damaged surface is blown and
Completely covered with bedding mix

There has been conjecture that bedding mix was not used, or not used very much. However, the project records show that more than 8,000 cubic meters of bedding was used. When this is averaged out over all the lifts, it amounts to something on the order of about 10 meters (or more) of bedding width on each lift for the entire length of the dam. It clearly was used and it was inspected to be sure that it was used at every location as required.

The question has been raised as to whether the bedding mix constitutes a different material with different properties compared to the RCC. This raised an issue of possible strain incompatibility. This is an astute question. It could occur if mortar was used for bedding and it was placed much too thick, resulting in a layer of harden mortar with different properties between layers. However, concrete bedding with coarse aggregate, as used at Paradise, tends to squeeze up into the RCC as it is compacted and become part of it. Also the bedding mix was deliberately made with similar aggregates to the RCC and with properties that are compatible with the RCC.

The issue of strain incompatibility is related to the stiffness (modulus of elasticity) of the material. That is, how much does it deform under load. The RCC at Paradise was deliberately designed to have low modulus and be able to accommodate deformations. It could be thought of as a "rubber" concrete instead of a brittle "glass" concrete. It also had a very non-linear stress-strain behavior. That is, as the load increased the ability of the concrete to deform increased. This is a unique advantage of lean mix RCC, and is sometimes seen to a lesser extent in high cementitious content RCC.

When a "rigid body" stress analysis is done, as it typically is for small and medium height dams and some high dams, it assumes that the dam is an infinitely stiff mass. If a more detailed finite element (FE) analysis is done it typically assumes a relatively stiff modulus of elasticity, say on the order of 20+ GPA for all of the concrete at all levels of stress. Both of these analyses will result in tensile and shear stresses at the upstream face that are greater than actually occur compared to if a lower modulus and a reduction in modulus with increasing stress are taken into account. The result is lower actual tensile stresses and less (or no) cracking compared to an analysis that does not consider this condition.

Extensive stress-strain testing was done during design and construction of Paradise, with the resulting modulus of elasticity at various levels of load being reported (at 25%, 50%, 75%, and 100% of ultimate load). When the actual modulus of elasticity for the site specific mix and the levels of stress or load at different locations in the dam is taken into account in an FE analysis (which may need to be done iteratively), the results will show that, as an area in the dam starts to become highly stressed, it "softens" and "re-distributes" stress from areas of high stress to areas of lower stress. The peak stress is consequently reduced. Extensive testing during design and construction showed that at Paradise the modulus of elasticity was about 20 GPa at 25% of ultimate load but that it reduced to 16 GPa at 50% of ultimate load, 11 GPa at 75% of ultimate load, and only 4 GPa at 100% of ultimate load. In order to be correct the cracking analysis should take this strain softening into account. It is one of the major advantages of lean RCC. Two published papers that deal with this issue specifically for RCC mixes are attached for the benefit of Commissioner Carter.

The GHD analysis does not seem to have considered this beneficial property. Instead, it simply assumed no bonding and horizontal cracking at the upstream face, then calculated how far back into the dam that crack would extend as a discontinuity until it reached an area where compressive stresses would keep the crack from migrating further. A thorough analysis, considering bond at the bedding mix and the strain softening properties of the RCC with its ability to redistribute stresses would probably result in no cracking. However, this is a very sophisticated analysis that not typically done, at least not for a small to medium sized dam.

Along with less brittleness lean RCC also has higher "creep" properties. Over time this tends to reduce stresses that result from a forced deformation such as differential movement in the foundation or stress due to thermal cooling of the mass.

Although it is not part of my involvement at Paradise dam, I have extensive experience with damage to spillways, outlets, and downstream erosion at dams. I have previously provided the PDI with two articles from the American Concrete Institute. One of them is *Erosion of Concrete in Hydraulic Structures.* The Orth one is a *Compendium of Case Histories*. I was principal author of these documents. The reality is that erosion and damage due to high flows is quite common in dams that have significant floods or discharges. Erosion of stilling basins, aprons, and plunge pools as well as the areas downstream of them happens. When I assembled the compendium I contacted dams around the world. The ones that had not experienced damage were the ones that never had a major flood or discharge.

Uplift within a dam reduces stability. It is routinely taken to be some percentage of the full reservoir head at the upstream face, decreasing to 1/3 of that where internal drains are utilized, and then decreasing to the tail water pressure at the toe of the dam. The GHD report apparently extended this uplift all the way to the end of the apron, thereby reducing stability. I have never seen this done before. If uplift under the stilling basin is a concern it usually is accommodated by drilling drain holes through it. However, I am not the designer at Paradise and will leave this issue to the designer, Richard Herweynen for him to address.

The upstream membrane and its integral face drain system seems to be working as designed, providing basically 50% uplift reduction at the upstream face, or more in some cases. As discussed later one piezometer showed higher pressure. Usually membranes result in reductions more on the order of 90% -100%. It may be that the face drains are becoming clogged and need cleaning. As I recall, the designer, Richard Herweynen, provided for this to be able to be done. I do not know if anyone has ever done it. It may result in improved uplift reduction and greater stability.

RCC was placed to full height of the dam at the right abutment before placing the lower part of the dam. It was just sitting there with no reservoir. Yet, hydrostatic pressure developed between the membrane and RCC that was sufficient to push the panels off the dam. One possible scenario is that the pressure came from groundwater that was experienced in the right abutment during construction. The other possibility is that rainwater or cure water permeated from the top of the dam down into the dam between the membrane and RCC. If the bedding mix near the upstream face acted to bond and seal the RCC lift joint as intended, the water would build up pressure until it pushed off the panels. Both of these possibilities should be explored, especially to see if either of them can be contributing to artificially inflated piezometer pressures.

If there really is water in lift joints at this pressure, and if the downstream part of the dam without bedding is indeed not bonded, we should be seeing substantial seepage coming out of the dam along lift lines at the downstream face. However, the downstream face is quite dry. Consideration should be given to drilling a hole and installing a new piezometer near the one that is reading high. It may be an erroneous reading.

My Witness Statement 1 primarily concerned shear strength at lift joints. It failed to include an important aspect of the friction angle. As GHD has pointed out (but not used) the isolated small surface area of a core can be relatively flat and smooth. However, the overall lift surface that must slide is undulating typically will have roller ridges. This unevenness adds to sliding resistance when a large lift area is considered compared to a small isolated smooth surface at an individual core location. This could add 1 to 3 degrees to the friction angle. It may be practical to use criteria for undulating rock and other surface to justify an increased friction angle. An increase of 1 to 3 degrees could make the difference between acceptable and unacceptable sliding stability.

Dr. Ernest Schrader