7. Technical Causes of Breach

7.1 Response of Overpumping Protective Systems on December 14, 2005

As noted above, both units were in the pumping mode in the early morning of December 14, 2005. At 04:39, Unit #2 was shut down automatically at an indicated upper reservoir water level of Elev. 1591.6. At 05:15, Unit #1 was shut down manually by the Bagnell Dam control center operator in accordance with instructions from St. Louis control center to shutdown just shy of where it would shut down automatically (Elev. 1594). At that time, the reservoir level reading was Elev. 1593.7. The automatic shut down of the first pump and the non-automatic shut down of the second pump is consistent with level information from the pressure transducers and the automatic shut down elevations described above.

Since the reservoir overtopped and the top of the parapet wall at its lowest point is at Elev. 1597, it is clear that the actual water level exceeded the indicated Elev. 1593.7 and that the pressure transducer signals were in error. No shutdown or alarm was produced from the conductivity probe backup system on December 14, 2005.

7.2 Upper Reservoir Water Level Monitoring and Control System as Found

Following the reservoir failure, the pressure transducers were removed from their protective pipe and re-calibrated. The pressure transducers in service on December 13-14, 2005 are identified as TX2 and TX3. TX1 had been removed from service earlier. The complete calibration test report by Siemens is contained in Appendix A of the Rizzo Report.

Figure 7-1 shows plots of ma output versus PSIG for TX2 and TX3 compared to a reference (ideal) transducer. Both TX2 and TX3 have linear response to pressure but TX2's ma output represents about a 7.86 feet higher indication than the reference curve while TX3's ma output represents about 0.85 feet higher indication than the reference curve. Figure 6 on page 20 of 76 in Appendix A of the Rizzo report shows that the as found PLC logic includes a subtraction of 9.38 feet from the TX2 pressure indication and a subtraction of 2.4 feet from the TX3 pressure indication. The basis for these adjustment values is not stated in Appendix A of the Rizzo report.

If the pressure transducers were located at the design elevation of 1500, these PLC subtractions in the pressure indications would be greater than they should have been based on the post-breach transducer calibrations and would have resulted in level readings about 1.5 feet lower than they should have been. However, if the pressure transducers were located above elevation 1500, the PLC subtraction values may have been selected to adjust the level readings to match the actual reservoir level. As such, the subtraction values would have adjusted the level readings for both the transducer offsets as well as actual elevation of the transducers.

Figure 7-2 shows plots of ma output versus temperature for TX2 and TX3 at a constant pressure of 40 PSIG (high upper reservoir level). While TX3 shows little response to temperature change, TX2 shows an unusual ma output shift between 5 degrees and 20 degrees. At temperatures below 5 degrees, TX2 indicates the pressure to be about 7.11 feet higher than that above 20 degrees for an actual constant pressure of 40 PSIG.

On December 13-14, 2005 the water temperature was in the 5 degree range. Since the upper reservoir level was calculated as the average of TX2 and TX3 on this date, the TX2 temperature shift output would have resulted in an indicated level of 3.56 feet higher than actual assuming that TX2 had been adjusted to match the actual level when the water temperature was above 20 degrees. By itself, the temperature response of TX2 as the water cooled would have indicated higher water levels and produced pump shutdowns at lower actual upper reservoir elevations for the same setpoint shutdown elevations.

Prior to removal of pressure transducer TX1 from service on September 27, 2005; the influence of temperature shift response in TX2 on the water level indication would have been less since it represented only one of three readings used in the average. After removal of TX1 from service, TX2 represented one of two readings used in the averaging process. Accordingly, the water level indication error due to water temperature changes would have been greater after September 27, 2005.

In response to FERC Question No. 29d:, AmerenUE responded in part “It appears that TX2 did not exhibit the 0.5 ma shift until tested at the GE facility under extreme and abrupt temperature changes.” In any case, such a temperature shift response in cold water would have resulted in a higher water level indication rather than a lower indication.

A visual examination of the pressure transducer protective pipes, Figures 7-3 through 7-5, shows that the protective pipes had moved from their straight alignment in the lower elevation of the reservoir. Since the transducer cables remained fixed at their instrument box on the parapet wall (Figure 7-6), any movement of the protective pipes from their initial straight alignment would produce an upward movement of the pressure transducer and a corresponding negative error in the water level reading. That is, the reported water level would be less than the actual level.

To avoid penetrations of the liner material and the creation of possible leakage paths, the protective pipes were supported on plastic plates that were connected by eye bolts to two stainless steel guide cables. The cables were secured only at the bottom and top of the reservoir. Figures 7-7 and 7-8 show these support systems as found after the breach event.

An internal e-mail from September 27, 2005, written two days after Hurricane Rita, stated “This morning Jeff and I went up to the upper reservoir when the controls indicated we were at 1596 elev. There were no waves on the surface but we could see a couple of wet areas on the west side of the reservoir parapet walls. We pulled the vehicle up to these wet areas and climbed on top of the vehicle to see the water level. We were surprised to see the level within four inches of the top of the wall. It was above the top batten strip holding the vinyl on. This level is at least six inches higher than what I remember from when we first came back from the controls upgrade last fall. Jeff looked at the level xmtrs when we got back to the plant and found one of the three reading a foot higher than the other two. When he took that one xmtr out of the average we now read about 1596.2. I still feel we are about another .4 feet higher than that. Jeff then added a .4 adjustment to the two remaining xmtr average making the current level now read 1596.6. We’ll check on what this does to the actual level the next several mornings.”

Figures 7-9 through 7-11 show upper reservoir water level readings taken during and prior to the Hurricane Rita event.

Figure 7-12 (09/27/2005) shows the disabling of one upper reservoir pressure transducer and one lower reservoir pressure transducer and the addition of the 0.4 feet offset in the upper reservoir level indication.

Another internal e-mail also indicates that the protective pipe movement was observed as early as October 7, 2005 and that the pump shutdown set point was lowered from Elev. 1596 to Elev. 1594 "__so that we won't pump over the reservoir walls."

Figure 7-13 shows water level readings from December 2, 2005. Until the second pump turned on for the second time, the water level fluctuations are relatively small and may be due to surface wave action or small movement of the pressure transducers within the protective pipe. However, after restart of the second pump, these level reading fluctuations increased dramatically and no longer have a stable periodicity.

Figure 7-14 shows a continuation of water level readings from December 2, 2005. Once the level rose above about Elev. 1563, the large fluctuations decrease significantly and are very small when the water level was falling during generation later in the day. This pattern of water level fluctuations is found on most days after December 2, 2005. This evidence suggests that the pump discharge pattern created substantial forces acting on the protective pipes and/or the support cables when the water level is lower and that these forces diminish as the flow discharge pattern shifts upward at higher water levels. The evidence also suggests that the generation mode flow pattern into the intake is more stable and produces much less disturbance to the protective pipes. This is consistent with the much higher exit losses associated with discharge into an open reservoir compared to entrance losses for the same geometry.

The actual forces acting on the protective pipes and/or the support cables during pumping may have resulted from the flow around them. Flow over the protective pipes and cables may also have produced Von Karman vortex shedding. Such vortices would produce alternate forces toward the reservoir wall and away from the reservoir wall. Forces away from the reservoir wall would reduce the normal force between the pipe support plates and the reservoir liner. This reduced normal force might have allowed slipping of the support plate and pipes along the reservoir liner.

The graphs of upper reservoir water level for December 1st through December 13, 2005 show relatively stable indications during generation with one or both units, standstill and pumping with only one unit. However, once a second pump starts, the water level indications are generally more erratic. This tends to confirm that the higher flow from two pumps is providing the force moving the pressure transducers protective pipe.

A review of two pump operations during 2005 shows that the upper reservoir water level indications are reasonably stable until early August. Figures 7-15 through 7-22 are examples of these levels from the pressure transducers. Beginning in early August, the water level plots begin to show the erratic behavior that increased until December 14, 2005.

Figure 7-23 shows an interesting pattern of water level readings for December 10, 2005 with both units off followed by both units generating. We don't know if these level fluctuations are due to transducer movement or other causes. The left portion of the plot seems to be damping out until the disturbance around 14:24. The subsequent fluctuations appear to be building in amplitude until the two generators began operation.

Figure 7-24 shows the water level readings from the start of both pumps on December 13, 2005 through the reservoir failure on December 14, 2005. The –222 MW arrow shows the indicated water level when pump 2 completed its start sequence. The water level indication remained level for about 12 minutes rather than immediately beginning the more rapid rate of rise that it should have. At that level of Elev. 1550, two pumps were producing a level rise of about 10 feet per hour or about 2 feet in those 12 minutes. While there were smaller subsequent level indication fluctuations, they did not restore the level readings back to the trend line shown.

The most logical explanation is that during those twelve minutes the transducers were moving up at about the same rate as the water level, hence showing no level change during the interval. The line labeled "Level trend without offset" shows where the water level indications should have been without the offset. It should be noted that the level indication at the beginning of the plot is not necessarily accurate given the many indications of prior transducer movement and erratic readings. It is also possible that generating mode flows past the transducers may have tended to bring the protective pipes back to near their original positions resulting in some periodic level error corrections.

Figure 7-25 shows indicated upper reservoir water levels around the time of the breach on December 14, 2005. A trend line has been added to show the calculated rate of rise for one pump operation at the maximum reservoir level. Note that the measured water level rate of rise matches the calculated trend line very closely to within a few minutes of the rapid drop in level. This suggests that the breach occurred very quickly after shut down of the second pump.

With a 15 minute per foot rate of rise for one pump and a minimum parapet elevation of 1597 at panel 72, more than 15 minutes would have been required to raise the water level from Elev. 1597 to Elev. 1598 since overtopping would have been occurring at panel 72 and other locations. Figure 7-25 does not show such a long period of reduced rate of rise prior to the breach. Therefore, the water level could not have reached as high as Elev. 1598.

Figure 7-26 is an enlargement of Figure 7-25 with two trend lines added. The left trend line represents rising water level prior to overtopping and the right trend line represents a reduced rate of level rise associated with beginning of overtopping. The lines intersect at about 5:07 AM suggesting that the actual level was around

Elev. 1597 at the time. An adjusted water level scale is included on the right of the plot based on an Elevation of 1597 at 5:07 AM. This analysis is based on level indications at the south end of the reservoir and does not include delay times associated with distance to the overtopping locations.

Figure 7-27 is a plot of maximum daily water level indications for December 2005. The plot shows that level indications as high as that shown for December 14, 2005 were achieved on many earlier days. Since reservoir failure did not occur on those dates, it suggests that the level reading offset described above for December 13, 2005 is primarily responsible for the failure to shut down the last pump. As noted above, that offset resulted in the actual water level being at least two feet higher than the pressure transducers indicated.

The buildup in level indication variations during pumping and the smoother level indications during generation suggest that the protective pipes were displaced due to pumping flows and tended to straighten out from generation flows and perhaps their own weight. We cannot be certain that the protective pipes always straightened out fully after a generation operation, so there may have been a residual level error when the pumps started on the evening of December 13, 2005 and at other times as well.

During our interview process, we asked operators from Osage and the St. Louis control center to describe the displays available to them showing upper reservoir water level. All interviewees stated that they have digital information as well as graphical displays of water level versus time. We then asked if they had ever seen any unusual indications on the graphical displays and all but one stated that they had not seen unusual indications. One interviewee did respond as follows; "I have seen a time or two where we've had a level problem, it would freeze up momentarily, and we've had them call and reset and it popped right back. I've seen that maybe once or twice."

We conclude that the failure of the second pump to shutdown automatically based on water level indication was due to level errors resulting from accumulated movement of the pressure transducers within their protective pipes including the twelve minutes of two units pumping on December 13, 2005 during which no level increase was indicated by the pressure transducers. Since the water temperature was in the 5 degree range on this evening, any influence of the TX2 temperature response would have been in the opposite direction to physical raising of the pressure transducers.

7.3 Emergency Water Level Protection Backup System as Found

An internal e-mail dated October 7, 2005 stated "The Hi and Hi-Hi Warrick probes are 7" and 4" from the top of the wall respectively. So if on 9-27 the level was 4" below the wall the Hi level Warrick should have picked up." And "If you want to lower the Hi level probes we can do that but I think we chose the levels so that normal wave action wouldn't cause nuisance trips." Since the top of the wall at the location of the Warrick probes was determined to be at Elev. 1597.92 by AmerenUE in 2004 and 1598.0 by KdG after the breach in December 2005 the Hi-Hi probe could have ranged between Elev. 1597.59 and 1597.67; the Hi probes could have ranged from 1597.35 to 1597.42.

After the breach, the Hi and Hi-Hi conductivity probes were found to be 4" and 7" below the top of the wall as described in the above e-mail of October 7, 2005. As shown on Figure 7-28, this places the Hi-Hi probe above the top of Panel 95 (1597.39), in the breached area and above the top of Panel 72 (1596.99), the minimum elevation of any panel in the reservoir. We received no documents or interview responses indicating why or when the conductivity probes were raised to these elevations.

Since the conductivity probe system had operated correctly when tested at commissioning in the fall of 2004, we investigated the following possible reasons for failure to respond before the breach.

Estimates of the maximum reservoir water level achieved prior to the breach were made by several parties using the following methods:

- Elev. 1597.63 based on examination of dike crest for evidence of water spill (erosion).
- Elev. 1596.74 based on post breach observed vertical movement of transducer pipes.
- Elev. 1597.4 based on examination of pressure transducer data for reduction in rate of rise while pumping suggesting Elevation 1597 (panel 72).

Figure 7-29 shows areas of erosion around the upper reservoir perimeter. Estimates of the maximum reservoir water level were made by noting the parapet levels adjacent to these erosion areas.

AmerenUE measured a 14 foot lateral displacement of the transducer pipes over an arc length of 119 feet in the displaced pipe as found after the breach event. This results in a calculated vertical movement of about 3 feet for the enclosed transducers. Adding 3 feet to the maximum measured water level of 1593.74 gives an adjusted water level of 1596.74.

It should be noted that the as found displaced position of the transducer pipes does not necessarily represent the maximum position achieved prior to the breach event. In the days following the event, the transducer pipes gradually straightened out and moved back to near their original position. As such, the actual vertical movement of the pressure transducers was likely somewhat higher than the calculated 3 feet value.

Figure 7-30 shows a maximum water level of about Elev. 1597.4 based on indexing the pressure transducer record to Elev. 1597 when the rate of rise decreased during one pump operation.

Figure 7-28 is a summary of the results including the as found elevations of the Hi and Hi-Hi conductivity probes. The estimated level during breach is a shown as a range of levels dependent on method of calculation noted above. The maximum water level based on the as found displaced shape of the transducer pipes is excluded for the reason given above.

While some estimates of maximum water level are higher than the Hi probe elevation, none of the selected estimates reach the Hi-Hi probe elevation. These results are consistent with the fact than no probe alarms were recorded on December 14, 2005 since an alarm is only initiated from the Hi-Hi probe and not from the Hi probe.

While we consider the above to be the most likely explanation for failure of the conductivity probe system to initiate pump shutdown, we considered the following additional possibilities.

At our request, a series of tests was conducted to investigate the sensitivity of the probe system to the following conditions:

- Clear vs. turbid water.
- Water temperature variation.
- Relay supply voltage variation.
- Ice on probes.

The results demonstrated that the conductivity probes and relays performed satisfactorily for all test conditions.

However, the investigation documented a programming error in the Unit #2 pump shutdown logic. This PLC error, made on September 16, 2005, disabled the Unit #2 shutdown in response to operation of any conductivity probe (Lo, Lo-Lo, Hi, Hi-Hi). The Unit #1 shutdown logic did not include this error. Figure 7-31 shows the final as found shutdown logic.

Since Unit #2 was shutdown manually on December 14, 2005, the programming error was not a factor in the overtopping event. Based on the above test results, Unit #1 would have shutdown automatically if the Hi and Hi-Hi probes had remained wet for the required sixty seconds.

We conclude that the Hi and Hi-Hi conductivity probes were located too high to initiate pump shutdown and prevent overtopping of the upper reservoir. As noted above, the programming error in the Unit #2 shutdown logic was not a factor in the December 14, 2005 breach of the upper reservoir.

7.4 Overtopping of Embankment Dam

7.4.1 Sensitivity of Taum Sauk Dumped Rockfill Dam to Overtopping

It is well known in the Dam Engineering profession that overtopping of embankment dams is one of the most frequent causes of embankment dam failures. In 1972 Buffalo Creek Dam in West Virginia failed by overtopping and 118 persons were killed. The dam was built from mine wastes. In 1977 two earth dams on the same river in Brazil were overtopped and failed during a storm. In 1964 flow through a 200 ft high section of Hell Hole Dam in California, under construction, resulted in a failure and the dam had to be rebuilt. The downstream slope of the dumped rockfill was a 1.3:1 slope which had a dominant size (diameter for 50% passing), Leps 1973, of about 8-12 inches. In any case, the Hell Hole failure is an incident where the exiting of seepage on a 1.3:1 dumped rockfill slope resulted in erosion and instability of the slope.

Because all embankment dams are considered to be vulnerable to failure by overtopping, embankment dams usually have spillways and failures still result in some cases due to either inadequate spillway capacity or improper operation of spillway gates, caused by human error.

In the case of pumped storage projects, the Upper Reservoir in many cases is not connected to a river and the reservoir levels are determined solely by the controlled pumping and generating activities. A study of precedent indicates that based on the philosophy of the various owners and engineers that some of these projects have a spillway capacity equal to the pumping capacity and others have no spillway at all and rely on controlling the reservoir level and terminating the pumping at predetermined reservoir levels. The Taum Sauk Project was constructed without a spillway and thus was dependent on monitoring to control reservoir levels to prevent overtopping. It is interesting that in the middle 1960's that Taum Sauk and Cabin Creek were the only two pumped storage projects without spillways on the Upper Reservoir to pass errant pump overflows.

Although it should be assumed in design that all embankment dams will fail if overtopped, some rockfill dams are more sensitive to failure by overtopping than others depending on the steepness of the downstream slope, the compactness of the rockfill, and the percentages of sand and fines in the rockfill.

Based on the appearance of the breach slopes at the Taum Sauk rockfill embankment during the initial inspection of December 15, 2005, it was evident that the embankment in the area of the breach was not constructed as a normal rockfill embankment. At best it should be classified as a "dirty rockfill" in the breach area as is shown in Figure 6-4. The recent drilling and investigation program conducted by Paul C. Rizzo Associates (PCR) has also indicated that the Upper Reservoir Embankment materials contain much finer materials than expected for a rockfill embankment. The recent program conducted in January 2006 involved drilling (7) borings using a 6 inch sampler and sonic drilling techniques. Even after correcting and adjusting for the smaller samples, the inferred rockfill gradations indicated fines contents as high as 20% passing the # 200 sieve. Reference PCR Forensic Report Dated April 6, 2006.

Studies of the rockfill gradations at Taum Sauk by PCR have resulted in the Lower and Upper bound grain size distribution curves, as shown in Figure 7-5 of the PCR Report and given in Figure 7-32 in this report. It is shown in Figure 7-32 that for the upper bound sizes of rockfill at Taum Sauk that the dominant size (50% passing) is 4 inches and that for the lower bound sizes that the dominant size is about 3/8 inch. Thus the dominant size of rock fill at Taum Sauk is significantly smaller than the Hell Hole dominant size range of 8-12 inches, as discussed above; thus the rockfill at Taum Sauk would be considered to be more vulnerable to erosion than the Hell Hole rockfill. Panel Member Hendron had the opportunity to inspect the rockfill at the rebuilt Hell Hole Dam in 1966 and can attest that the gabbro rockfill at Hell Hole Dam was much stronger and of larger size than the Taum Sauk rockfill. The Hell Hole rock appeared not to have any materials passing the No. 200 sieve, whereas the range of curves shown in Figure 7-32 indicate that there was from 0-20% passing the No. 200 sieve and from 0 to 45% sand in the rockfill at Taum Sauk. Due to the steep downstream slope and the small dominant size range of the dumped rockfill at Taum Sauk it is the Panel's judgment that the Upper Reservoir embankment dam slopes in the area of the breach were composed of "dirty" rockfill and were very erodible as compared to other rockfill dams, especially other compacted rockfill dams. In fact the historical documentation of the project contains many comments by James Barry Cooke and others about the erosion of portions of the slopes due to rainfall.

It is noteworthy that Cabin Creek Dam was constructed as an upper reservoir dam for a pumped storage project in Colorado. This dam was completed about a year after Taum Sauk and consisted of granite rockfill compacted in two ft thick lifts with a maximum size of 2 ft. The rockfill did not have measurable amounts passing the #200 sieve and had a maximum percentage passing the 1-inch size of 10%. The downstream rockfill slope was 1.75:1. This dam was overtopped by over pumping but did not fail. It is no doubt in large part due to the fact that the dam was well compacted clean rockfill, as opposed to being dumped, and the downstream rockfill slope was somewhat flatter at 1.75:1 as compared to the dumped "dirty" rockfill slope of 1.3:1 at Taum Sauk.

The "dirty" rockfill found at Taum Sauk, with as much as 45% sand plus fines, was likely not free draining for the flows imposed by overtopping. Thus, the flows from overtopping could increase the phreatic levels beneath the parapet wall and within the downstream slope. In the case of a steep downstream slope of 1.3:1, the phreatic levels do not need to be increased very much to cause instability of many potential failure surfaces. The designs of steep sloped CFRD's are predicated on the assumption that the rockfill is free draining. The rockfill found at the Taum Sauk Breach may in fact not be free draining, and increases in piezometric levels caused by the overtopping flows could also have initiated stability failures of various portions of the slope and/or sliding and overturning of the parapet wall, as well as erosion.

The failure of the Gouhou Concrete Face Sand and Gravel Dam in China, on August 27, 1993, is pertinent to the Taum Sauk breach. Gouhou Dam had an upstream slope of 1.6:1 and a downstream slope of 1.5:1 and was a well compacted gravel which contained, on the average, about 40% sand. The top of the face slab was at Elev. 3277.35 meters where there was a joint between the horizontal footing of a parapet wall and the top of the face slab. The dam had been in service for more than 3 years but the reservoir level had never exceeded Elev. 3277.35 meters. An investigation of the failure found that the dam failed within about 24 hours after the water elevation exceeded 3277.35 meters. It was concluded from this study, in a paper by Zuyu Chen, October 1993, that the infiltration into the gravel-sand fill, from the face slab-parapet wall joint, increased the phreatic surfaces in the dam due to the fact that the gravel-sand fill was not free-draining and resulted in failure of the downstream slope. This particular failure is pertinent to the Taum Sauk case because it is an illustration of the mode of failure which can and did happen in the case due to leakage through a concrete face and of parapet wall-face joint into a less than free-draining embankment fill. This is one of the hazards of permitting a "dirty" rockfill; the Taum Sauk fill could have had as much as 45% sand sizes or smaller which of course was similar to the percentage of sand in the Gouhou embankment fill.

7.4.2 Effect of Storing Water on Parapet Wall

The effects of storing water against a parapet wall as a "normal" routine loading when the embankment is a dumped rockfill dam are to increase the number of potential modes of failure and to intensify or increase the probability of occurrence of other modes of failure which existed prior to the decision to store water against a parapet wall founded on the dam crest.

For example, the placement of a 10 ft-high parapet wall on the crest of a dumped rockfill dam before settlements are complete most likely will result in differential settlements along the wall; and, the downstream movements associated with the water loading on the dam face and upstream side of the wall will result in opening of the joints between the parapet wall panels. This opening of the parapet wall joints results in additional leakage through the wall joints which would not occur if the parapet wall were not used to contain operating reservoir levels. This leakage could decrease the stability of the slope upon penetration into a dirty rockfill or it could be the cause of surface erosion of the downstream slope surface.

In the case of overtopping of a 10-ft high parapet wall, the velocity of the water impinges on the dam crest with a velocity of about 25 ft/sec., which is enough to accelerate erosion at the toe of the wall and results in the water having an initial velocity down the downstream slope, which enhances the erosion capability of a given flow over the top of the wall.

In the most severe case, the overtopping water may erode the rockfill at the toe of the wall footing enough that the 60 ft wide parapet wall panel tips over and results in an immediate flow through the 60 ft wide opening of about 7,000 ft^3/sec. This large discharge is an immediately available source of erosive energy at the top of the slope; it is a source of erosive energy which would not be available if the wall were not used as a storage mechanism.

For the Taum Sauk Upper Reservoir, the probability of overtopping the parapet wall was high in the case of any instrument errors because the shut off elevation of 1596 was too close to the low point on the top of the wall of 1596.99 at Panel 72.

7.4.3 Foundation of Rockfill Dam

The foundation rock at the Upper Reservoir Dike, being the flattened top of Proffit Mountain, is generally fresh to slightly weathered, hard, moderately to abundantly jointed rhyolite. Joints are generally steeply dipping, open, and some were filled with clayey products of weathering such that seepage would occur without proper measures to seal the reservoir floor. During construction, the overburden was observed to vary from a few feet to as much as 65 feet thick (MWH, 2003). Several significant clay seams, gently dipping, and up to four inches in thickness were encountered. Under the dike, the seams were treated either by excavating and backfilling with concrete or covering with smaller-sized compacted rockfill. The upstream (or inside) 70 feet of the base of the dike was specified to be prepared such that not more than two-inches (average) of soil were left in place. A filter zone and several layers of compacted rock were placed over questionable areas where piping of the foundation might be possible. Outside the 70-foot zone, the weathered rock was left in place where its competence was judged equivalent to the rockfill. Low areas or depressions in the natural topography were filled with compacted rock. Drainage to the outer slopes was reportedly provided for all foundation areas.

During IPOC inspections at the site, a residual soil zone of weathered rhyolite could also be observed in the breach area; and one location is shown in Figure 6-4. The residual soil was observed to be clayey and it was judged to have an effective shear strength almost dictated by the clay portion of the soil. Exposed rhyolite bedrock is also observed in Figure 6-4 as well as the remnants of the lower face slab and plinth.

A closer view of the exposed rhyolite bedrock and residual soil is shown in Figure 6-5. This photo is taken looking east and the rather flat looking joint surface in the rhyolite dips toward the camera in a westerly direction. This discontinuity was observed in the field to dip nearly west at a dip of about 10^o. This discontinuity is described as Fracture Set 8 (FS-8) in the Rizzo Report and is reported to have a dip of 8^o and a dip azimuth of 270^o. As a result of the observation of the residual soil, the IPOC requested that samples of the residual soil be taken for direct shear testing.

The shear strengths reported in the Rizzo Forensic Report ranged from an “effective” angle of shearing resistance of 28° to 38°, with a best fit of 33°, when the data is interpreted with a cohesion value of 0. It is possible that this zone of residual soil of weathered rhyolite was present downstream of the 70 ft. wide stripped area and could control the overall stability of the embankment, rather than the angle of shearing resistance of the rockfill, as the angle of shearing resistance is less than the rockfill and the zone of residual soil dips down the hill parallel to the original topography. The low dipping joint surface shown in Figure 6-5 is important in that it serves to give a foundation discontinuity which daylights to the west side of the embankment and gives a foundation that in general dips downhill at about 8-10^o in the direction of the applied water forces. In addition some of these joint surfaces appear to have clay coatings.

Considering the downstream sloping topography of the embankment foundation of residual soil overburden and the significant clay coated joints within the foundation rock that also gently dip to the west, together with the steep embankment slopes, it is understandable that the stability of the embankment may have been marginally stable and vulnerable with the additional conditions imposed by overtopping. The surcharge conditions imposed by the water flowing over the parapet wall and over or through the embankment materials may have induced higher phreatic surfaces and caused sliding along the base as well as facilitated shallow slope movements during the progressive failure of the Upper Reservoir embankment.

7.5 Possible Failure Modes

7.5.1 General

The experience that the embankment and parapet wall survived maximum water levels between Elev. 1595 and 1596 many times between 1963 and 2004 with leakage out of the reservoir ranging from 10 to 100 cfs indicates that the dam was stable for the conditions present before the liner was installed in 2004. This observation indicates only that the Factors of Safety of the dam slopes, and the Factors of Safety of the wall against overturning and sliding were greater than 1.0 for various potential sliding surfaces for conditions prior to 2004. This does not mean that the actual Factors of Safety between 1963 and 2004 would meet 2006 standards or FERC Guidelines, but that is really only an academic discussion anyway because in this report we are mainly concerned with the technical reasons for breach on December 14, 2005.

After the fall of 2004, the geomembrane covered the face slab and reservoir face of the parapet wall which reduced the total leakage from the Upper Reservoir to about 5-10 cfs. Thus the possible local phreatic surfaces around the wall and its footing as well as phreatic surfaces within the dam should have been lower than they have ever been and the Factor of Safety of all modes of failure should have been higher than at any time in the history of the project for the 1596 reservoir levels without the effects of wall overtopping. The chronology of events strongly suggest that although the construction of the liner made the Upper Reservoir dam more stable, that the unreliable instrumentation system and the missetting of the Warrick Probes made overtopping possible. Moreover field observations after the breach indicated that overtopping did occur. Thus the modes of failure discussed below are only those associated with overtopping. The dam is assumed to have proved its stability before the overtopping event of December 14, 2005.

7.5.2 Discussion of Specific Modes of Failure

Mode a) The 1.3:1 slopes (37.5°) are very steep and when overtopping occurs it is very easy to get erosion down the slope surface and a local increase in phreatic surface parallel to the slope which can result in shallow progressive sloughing of the slope possibly from the toe upward until the sloughing begins to undermine the parapet wall which leads to sliding and overturning of the wall which then greatly increases the flow as one 60 ft. wide panel overturns or slides resulting in a very high flow which greatly accelerates the failure by immediately imposing a flow of 7,000 cfs on the slope.

Mode b) As overtopping initiates the process in a) above and the progressive sloughing takes place, the flow of water over the top of the 10 ft. high wall impinges at the dam crest at a velocity of 25 ft./sec. and begins locally undermining the wall footing in addition to the sloughing caused by thin layers becoming saturated and failing deeper with time. This shortens the time required to reach overturning or sliding of the wall. In addition to undermining the wall footing, this jet of water at 25 ft./sec. impinges on the upper finer rockfill and can locally transfer to a 10 ft. pressure head which can change the stability of the wall by changing the uplift pressures at the wall toe.

Mode c) It is possible that the local increase in the phreatic surface between the parapet wall and the upper part of the slope caused by the impinging jet of water can cause a local wedge just beneath the wall to deform and/or reach limiting equilibrium without the entire slope below becoming unstable. This is similar to the case considered by means of a FLAC analysis in the FERC Breach Report as shown in FERC Report Figure 9.5. This is one possible mechanism which is enhanced by the high parapet wall loading in excess of 10 ft. of water head. It is obvious that this mechanism can occur combined with a) and b) above.

Mode d) Another mode of failure can be deep wedges founded on a base of residual soil inclined downhill at about 10°. The various wedges could have steep backslopes as shown in Figures 8-22, 8-26, and 8-28 of the Rizzo Report and can be analyzed for varying phreatic levels on the residual soil base.

7.5.3 Comments

According to the stability analyses conducted by PCR and FERC potential failure mode a) is very likely and the progressive sloughing and erosion in a) can be accelerated, leading to sliding or overturning of the wall, when taking into account the local undermining of the wall by the velocity of the water jet impinging on the downstream side of the parapet wall footing as described in b) above. According to the PCR calculations the parapet wall is likely to fail by overturning if undermined by 3 ft. Mechanism c) described above seems possible and was indicated by a FLAC analysis conducted by FERC. The deep wedges of mode of failure d) were analyzed by PCR and required the phreatic surface near the toe to build up to about 30 ft. above the base of the toe of the dam. This mechanism is possible but the time for this deep phreatic surface to build up 30 ft. is somewhat problematic considering that the "dirty" rockfill will result in a high percentage of water runoff rather than deep infiltration.

It is the judgment of the IPOC that we most likely will not ever know the exact sequence of failure at the breach. It seems most likely that the failure mode was a combination of modes a), b) and c) described above. The participation of a deeper mode such a d) cannot be excluded however especially after any wall panel overturning results in a huge flow of water.