OVERVIEW OF THE TAUM SAUK PUMPED STORAGE POWER PLANT UPPER RESERVOIR FAILURE, REYNOLDS COUNTY, MO

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INTRODUCTION

In 1953 Union Electric began considering construction of a pumped storage facility to generate electrical power during peak usage periods, which was a relatively new concept at that time. The pumped storage scheme had the advantage of being able to operate at full power almost immediately, allowing the owner to postpone construction of a much larger steam plant by harnessing some of the company's off peak power. Construction of the lower and upper storage reservoirs was authorized for the Taum Sauk site in December 1959 (Gamble, 1960), and operations began in 1963.

Although other sites were considered, the St. Francois Mountains of southeast Missouri was selected. The rugged topographic relief provided the required head for the efficient operation of a pumped storage power plant and favorable geology was conducive to the construction of the needed reservoirs. The final selection of a location for the facility, named the Taum Sauk Pumped Storage Hydroelectric Power Plant, was chosen along the East Fork of the Black River and atop Proffit Mountain near Annapolis, MO in Reynolds County about 5 miles from Taum Sauk Mountain, the highest point in the state. Proffit Mountain is the 6th highest point in Missouri and provides around 800 feet of elevation differential between its peak and the valley of the East Fork of the Black River below.

GEOLOGIC SETTING

The St. Francois Mountain region is a geologically unique area forming Missouri's oldest geologic province. During the Precambrian time (>1,500 Ma) igneous granite rock formed due to volcanic eruptions and intrusions of magma (Unklesbay and Vineyard, 1992). Volcanoes erupted large quantities of ashy pyroclastic flows and rhyolitic lava. Thick layers of pyroclastic materials were deposited throughout the region as either air fall tuff or ash flow tuff. Residual heat from the eruptions often melted or "welded" the pyroclastic ash fragments together and cooled to form a steel-hard igneous rock known as welded tuff or ignimbrite. Most of the ash flow tuff present in the Proffit Mountain region is reddish in color and of felsic, or rhyolitic composition. Various rhyolites and tuffs have a cumulative thickness of several thousand feet in the St. Francois Mountains. Large bodies of reddish to grayish granite formed when magma cooled slowly within upper portions of the earth's crust.

After the decrease and eventual halt of volcanic activity during the Precambrian time, the area was subjected to the uplift of the Ozark dome (Unklesbay and Vineyard, 1992). This uplift exposed the igneous knobs and ridges common to the St. Francois Mountains of today. When the Cambrian seas began to rise, much the region was blanketed by water, leaving the igneous knobs and ridges as highpoints or islands. Deposition of sedimentary rocks during this time left thick layers of sandstones and dolomites on the sea floor and draped layers of the same material on the slopes of the igneous highpoints and knobs.

Regression of the Cambrian seas exposed the younger sedimentary deposits and the igneous highpoints. Erosion of the Cambrian strata cut new drainage patterns. In some places, these drainage patterns were cut through sedimentary deposits down to steep igneous ridges preserved in the subsurface. The modern drainage pattern formed without regard to the underlying Precambrian terrain, which resists the effects of weathering and erosion to a greater degree than the younger, softer sedimentary rocks. When rivers cut down into these ancient bedrock ridges, their flow is locally restricted, forming steep, closed in chutes and potholes called shut-ins. Johnson Shut-Ins on the East Fork of the Black River is an example of this type of feature and is located below the Upper Taum Sauk Reservoir. As with the most of the Ozark Plateau, the St. Francois Mountains were not glaciated during the Pleistocene. This preserved many ancient, deeply weathered zones of bedrock and soil which are present throughout the region.

The Taum Sauk Upper Reservoir is located atop Proffit Mountain, a highpoint that remained exposed during the rise of the Cambrian seas. The top of the mountain is comprised mainly of what is known as Taum Sauk Rhyolite. During construction of the Upper Reservoir, much of the mountain top was blasted off, with the material, mainly broken Taum Sauk Rhyolite, being used to construct the Upper Reservoir's rock-fill dike:

> The Taum Sauk Rhyolite is a red to dark maroon ash flow tuff containing up to 30 percent phenocrysts of alkali feldspar and quartz; fiamme may or may not be present. The formation is widely exposed in the Proffit-Wildcat-

Taum Sauk mountain area. Although its maximum thickness has not been established, it is greater than 3000 feet thick. The type section is in sec 15, T.33 N., R. 2 E.; Johnson Shut-Ins Quadrangle (Thompson, 1995).

CONSTRUCTION AND EARLY HISTORY

The Taum Sauk Pumped Storage Hydroelectric Power Plant was constructed from 1960-1962 and began operation in 1963. The lower reservoir was formed by constructing a 60 foot high concrete gravity dam along the East Fork of the Black River around 3 miles upstream of Lesterville, MO. The upper reservoir was sited on Proffit Mountain, approximately 800 feet above the lower reservoir and connected by a 7,000 foot long tunnel. The upper reservoir was created by excavating the crest of Proffit Mountain and using the muck to construct a kidney-shaped rockfill dike with a maximum height of approximately 90 feet. An as-built cross section is shown in Fig. 1. The water side of the embankment was lined with shotcrete and capped by a 10 foot high parapet wall. The total capacity of the upper reservoir was about 4,600 acre-feet.

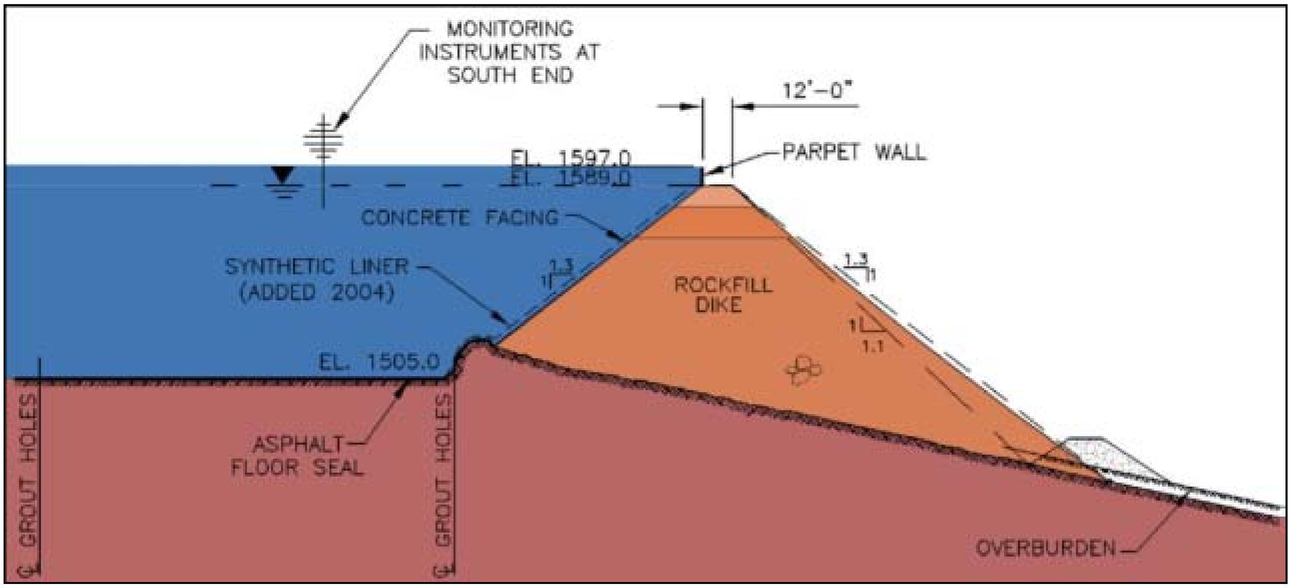

Fig. 1. – North side of the breached embankment shows the lower three quarters was end-dumped while the upper quarter was placed into two sequences of rolled filling. There does not appear to have been any significant effort made to mechanically compact the embankment because it was assumed to be a clean rockfill.

The majority of the upper reservoir's rockfill embankment appears to have been constructed through simple end dumping of the excavated material. The fill was allowed to tumble down the side of the embankment, lying near its natural angle of repose. The embankment materials were not mechanically compacted until the upper 16 feet of fill, which was compacted in four separate 4-foot thick lifts. According to FERC (2006), this was the last uncompacted concrete faced rockfill dam constructed in the United States. The boundaries between the different methods of fill placement were easily observed in the breached section of the embankment after the failure (see Fig. 2).

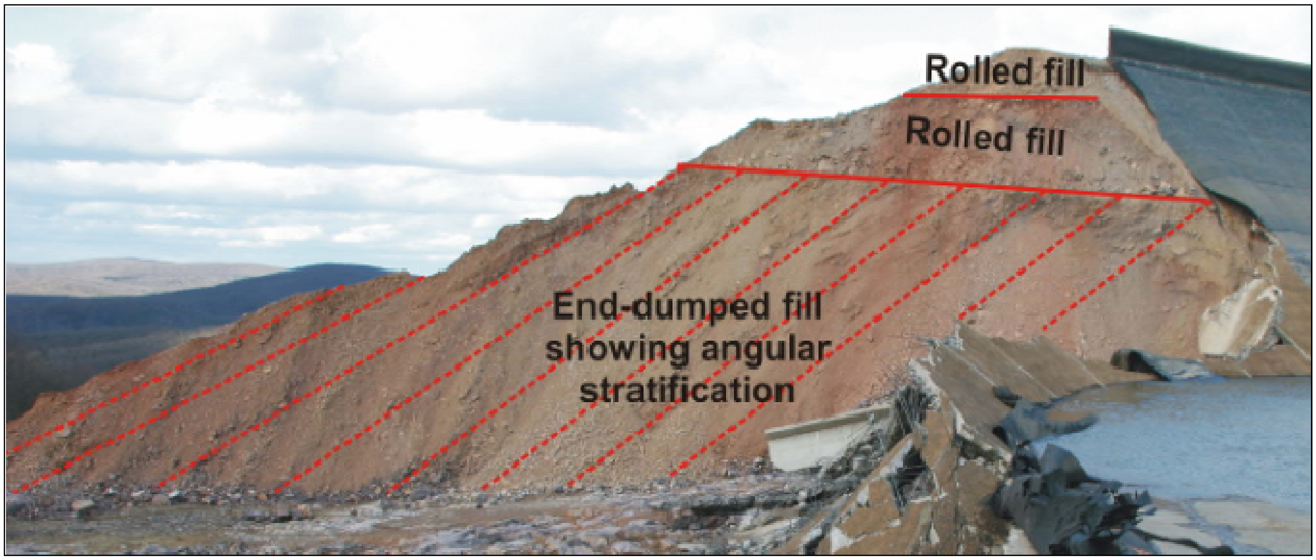

Fig. 2. – North side of the breached embankment shows the lower three quarters was end-dumped while the upper quarter was placed into two sequences of rolled filling. There does not appear to have been any significant effort made to mechanically compact the embankment because it was assumed to be a clean rockfill.

A 350 MW powerhouse/pumping station was located at the southern base of Proffitt Mountain, accessed by a channel, excavated into the native bedrock. This station was equipped with two reversible pump/turbine units of which one or both would operate in pumping or generation, depending on the power demand and available water in the reservoir. In 1999 the two units were upgraded to 440 MW capacity, which could lift up to 5,238 cfs into the upper reservoir under full head.

SETTLEMENT OF THE UPPER RESERVOIR EMBANKMENT

High rates of settlement were experienced at the upper reservoir during the first four and a half years of operation. Between 0.5 and 0.8 feet of settlement were experienced during this time, which correlates to 0.53% to 0.73% of the total embankment height, respectively. J. Barry Cooke (1915-2005), a member of the National Academy of Engineering, served as a consultant to Union Electric for the project, from the time of its design and many years thereafter. In 1967 Cooke summarized his observations about the reservoir's performance in a letter to Union Electric (Cooke, 1967). Cooke felt that the average settlement rate of over 0.1 ft/year was unprecedented when compared to other rockfill dams, but was acceptable for this proect. He concluded that "*The Taum Sauk Rhyolite Porphyry is an excellent high compressive strength rock that should have stabilized in its settlement. However, the formation contained frequent zones of soft weathered rock, all of which could not have been selectively wasted*" and that *"I believe that a fill of 100% competent rock would have stabilized and that the percentage of weathered rock in the Taum Sauk is the cause.*" An example of such a weathered zone is shown in Fig. 20.

Settlement continued up to the time of failure in December 2005, with differential settlements approaching two feet along the crest of the reservoir's parapet wall (see Fig 3). This differential settlement also led to a cracking of the concrete lining and continual problems with leakage. Leakage became so severe in 1963, only months after operations began, it necessitated shutdown of the facility and additional grouting of the reservoir floor in the northwest portion near where the reservoir failed in 2005 (FERC Independent Panel, 2006).

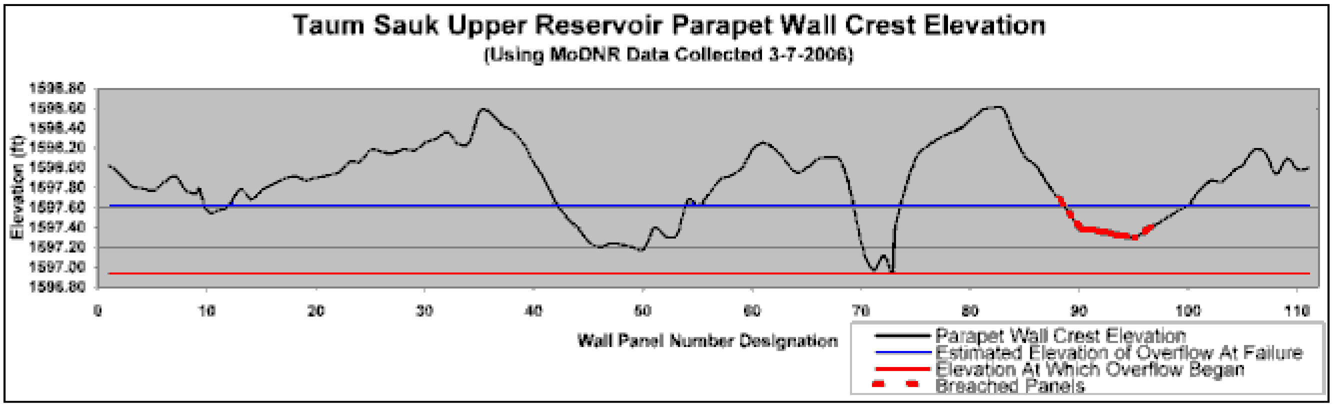

Fig. 3. – This figure illustrates the differential settlement along the crest of the upper reservoir parapet wall. Four segments of the parapet wall were almost two feet lower than their original elevation. The elevations of the breached panels (shown here in red) were estimated by AmerenUE after the failure and may not be accurate. (Data: MoDNR (2006))

INSTALLATION OF GEOMEMBRANE LINING

Leakage continued to increase each year, leading to concerns about the stability of the dike section and the efficiency of pumped storage operations. The upper reservoir was losing about two feet, or about 110 acre feet or water, to seepage each day (Tomich and Leiser, 2006). Some of this water was collected in small ponds and pumped back into the upper reservoir to retain efficiency.

Geosynthetics Inc. (GSI) was contracted to line the upper reservoir in 2004 at a cost of around $2.4 million in an attempt to reduce this leakage. GSI supervised the placement of 1.3 million square feet of 80 mil high density polyethylene (HPDE) textured geomembrane and geocomposite material. They also covered five rock outcroppings on the inboard side slopes with 80 mil textured linear low density polyethylene (LLDPE) material. After the lining project was completed, leakage from the reservoir was reduced dramatically (see Fig. 4). Leakage rates dropped by an order of magnitude, from an average of around 50 cfs to about 5 cfs and the overall efficiency of the facility reached approximately 70% (FERC, 2004; and FERC, 2005).

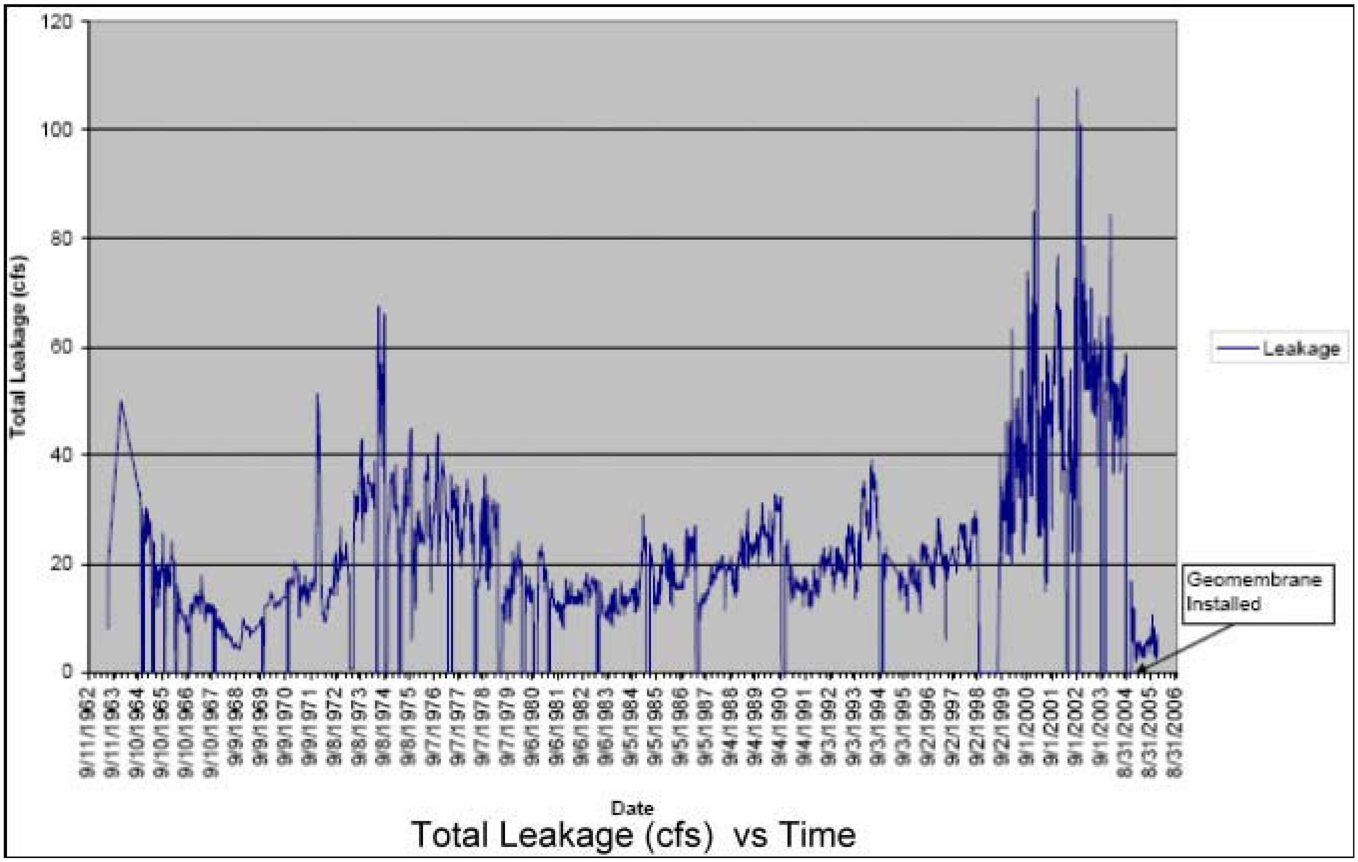

Fig. 4. – This figure from FERC's Independent Panel of Consultants Report (2006) shows a gradual upward trend in leakage rates until it was lined in 2004, dramatically reducing seepage.

DEREGULATION LED TO INCREASED UTILIZATION

The Taum Sauk facility was operated approximately 100 days a year prior to deregulation of electric power markets in the 1990's. Deregulation allowed utilities to sell power on the open or spot market at non-regulated rates to other utilities, increasing the value of power sold during periods of peak demand. This change in the markets made it profitable to run the facility around 300 days a year and AmerenUE provided financial incentives for executives based on the profitability of their power generation facilities (Leonard, 2006 and Leonard, 2007). Increased utilization likely influenced the decision to upgrade the pump/turbine units in 1999, which increased the efficiency and profitability of the plant.

RECOGNITION COINCIDENT WITH SERIOUS INSTRUMENTATION PROBLEMS

The Institute of Electrical and Electronics Engineers (IEEE) declared the Taum Sauk Plant an *Engineering Milestone* on Sept. 26, 2005. This recognition has only been bestowed on around 75 engineering project worldwide. The Taum Sauk project was recognized for:

- The plant was the largest in North America and one of the first of its type when it was constructed in 1963.
- The plant used the largest turbine generators/pumps in the nation when placed in operation.
- It was the first hydroelectric plant to be operated remotely (from St. Louis or the Bagnell Dam Power Plant), without humans onsite.
- Its ability to re-start the power grid in the event of a complete blackout, as coal and nuclear plants need external power to re-start.

Some AmerenUE employees visited the upper reservoir on Sunday September 25th, the day before the IEEE awards ceremony. They observed water pouring over the parapet wall along the northwest portion of the reservoir, in an incident they described as resembling "Niagara Falls." Operators acted quickly to manually shut down the pumps and turn on the generating units to lower the reservoir. Normally, workers wouldn't have been onsite on a Sunday morning.

Inspections after the September 25th overtopping revealed scour of the rockfill embankment up to 1 foot deep. Wind-whipped waves from the remnants of Hurricane Rita were initially assumed to have played a role in fomenting the overtopping, but the reservoir level was well above the normal freeboard maintained below the crest of the parapet wall.

Media reports suggest that a second, minor overtopping occurred on September. 27th, one day after the plant won the award and two days after the initial incident. On this occasion water levels were observed 4" from the top of the parapet wall and moisture on the land side of the parapet wall panels indicated minor overtopping had occurred that morning, however, it was not observed.

According to news paper accounts (Tomich, and Hand, 2006), AmerenUE's plant operator sent an e-mail to his supervisors on September 27th warning them about continued overtopping of the upper reservoir after the second overtopping incident. The operator stated that "*Overflowing the upper reservoir is obviously an absolute 'NO-NO,'" "The dam would severely erode and cause eventual failure of the dam…*" and "*If water continued to spill over the top of the wall, it could cause a section to collapse and then it would be all down hill from there — literally.*"

Divers were summoned and they ascertained that the new sensor conduits had become detached from their mountings along the sloping concrete face of the reservoir. Maximum water levels in the reservoir were re-programmed to reduce the operating level by two feet to provide a temporary "margin of error." Unfortunately, permanent repairs were postponed until regularly-maintenance the following spring (FERC Taum Sauk Investigation Team, 2006) to avoid an additional shutdown of the facility. Fig. 5 shows a Photo taken by AmerenUE on Oct 5th, 2005 when the reservoir pool was drawn down. This shows the sensor conduits have detached from their anchorages.

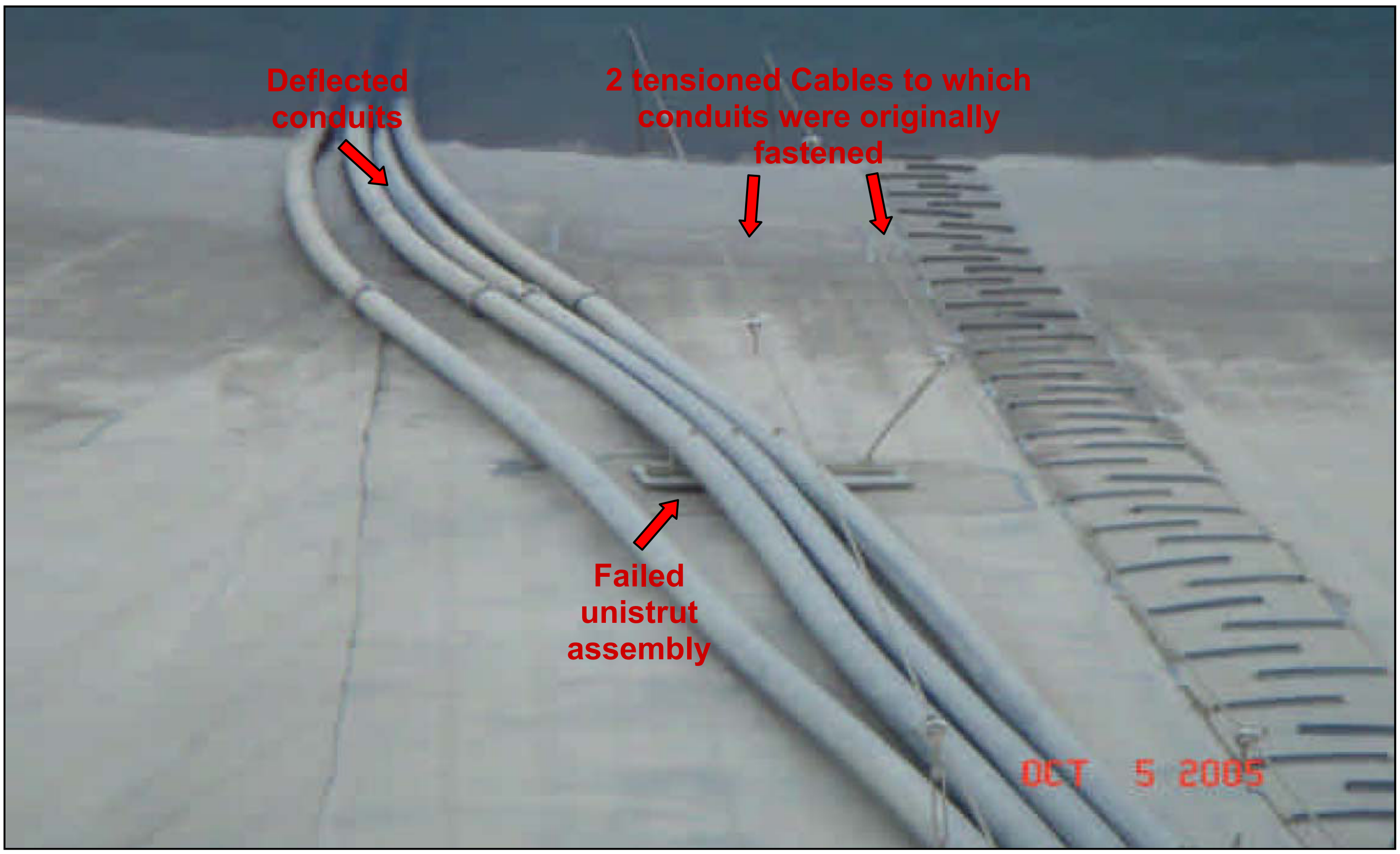

Fig. 5. – This photo taken on October 5, 2006 by AmerenUE shows the gross deflection of the reservoir's instrumentation array (photo from Rizzo Associates, 2006).

It was assumed that the two foot adjustment would prevent the reservoir pool from overflowing until repairs could be completed 6 to 8 months hence. The reservoir was also equipped with "fail safe" Warrick probes, which were intended to shut down the pumps automatically if the water level reached a pre-programmed elevation on the parapet wall. These probes used conductivity readings, which only activate when exposed to water in the reservoir. These probes were intended to be a "fail-safe backup" in case the stage sensors in the submerged conduits failed for some reason. If the Warrick probes activated, the pumping units were programmed to shut down immediately instead of the normal method of gradually ramping down the inflow.

THE FAILURE

The 6,562 ft long parapet wall was comprised of 111 panels, each about 60 feet long. Each wall panel was designated by a number, between 1 and 111. These numbers are referred to in the description that follows. See Fig. 6 for locations of overflow (shown in orange) and the eventual breach (shown in red).

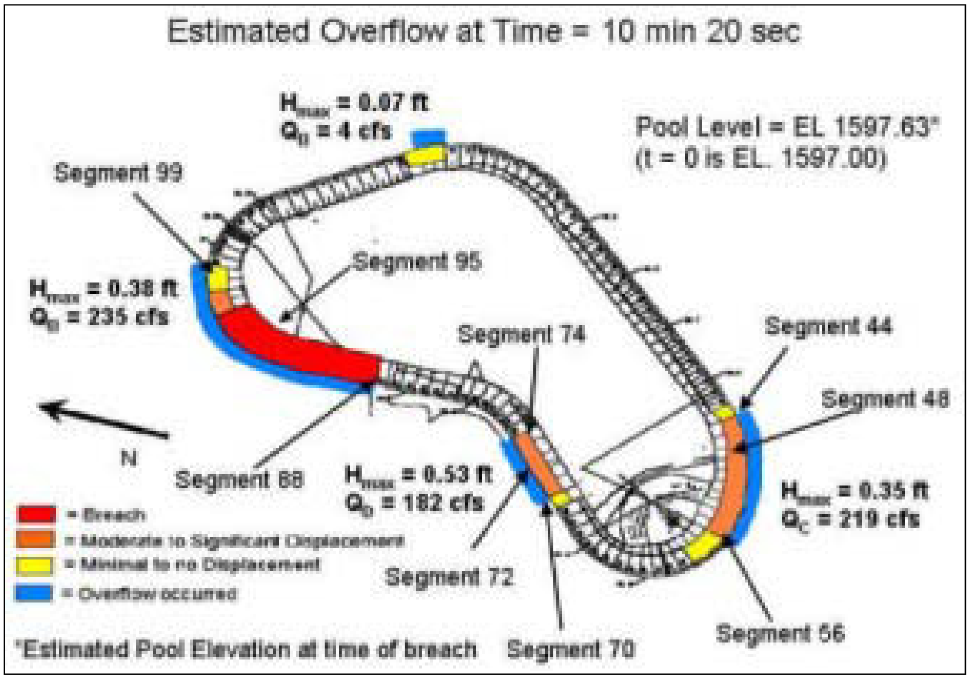

Fig. 6. – This graphic from Rizzo Associates, Inc. (2006) shows panel locations where overflow and failure occurred.

On Wednesday December 14th, 2005, the main reservoir stage sensors failed to shut down the pumps feeding water into the upper reservoir during the closing stages of its nightly filling. The "fail safe" Warrick probes affixed to the parapet wall were not activated. Water began pouring over the reservoir's parapet wall at the four locations where the wall had experienced the greatest settlement (shown in Fig. 6). Data recovered from the reservoir's control system and back-calculations indicate that the overflow likely initiated around 5:09 AM along the northwest portion of the reservoir (in the vicinity of wall panel 95). The parapet wall failed between panels 88 and 97, though physical evidence of overtopping was also observed beneath panels 100-103. One of the turbine pump units had shut down prior to overtopping (this unit was programmed to shut down 4-6 feet below the wall crest), but the second unit continued to run until the depth of overflow at panel 95 location was over 4.3 inches. This pumping unit was programmed to shut down when the reservoir level rose to within 2 feet of the wall crest. Water levels continued to rise until the second unit shut down. Based on back-calculations from a series of reports including Rizzo Associates, Inc. (2006), FERC's Independent Panel (2006), and FERC's Taum Sauk Investigation Team (2006), the reservoir was likely within seconds of failure by the time the second pump unit shut down.

The overflow resembled flow over a broad crested weir, with an extremely broad 'V-shape' over a distance of almost 900 feet. This condition would have trained more discharge towards the center of the settled section.

The discharge passing over the parapet wall initially spilled onto the wall's outboard footing, which was three feet wide. When the depth of overtopping exceeded 4.1 inches the lower nappe of the spillage would have begun spilling onto the unprotected embankment materials, scouring the rockfill beneath the wall. Post-failure observations at Panels 10-12, 43-56, and 69-74, (which all survived) suggest that the water quickly scoured deep plunge pools where it poured directly onto the unprotected rockfill. These materials were rapidly transported downslope and deposited in fans towards the toe of the embankment. The plunge pool deepened itself within a matter of minutes, displacing the largest clasts onto the rim of the plunge pool. Plunge pools normally excavate themselves to a depth of 1.5 times the free fall height on level ground, but the steep face of the rockfill dike (1.3:1, horizontal to vertical) may have exacerbated this situation, allowing an even deeper pool to develop. This statement is based on post-failure observations below Panels 71-72 (shown in Figure 7). The overtopping flow appears to have undermined three adjacent panels of the parapet wall in about six minutes, centered around panel 95, on the northwest side of the reservoir. Excerpts from a reconstructed failure sequence and complementary photos illustrative the failure mode (see Figs. 8-18.).

Fig. 7. – This photo taken below panels 71-72 show the deep plunge pool that developed and subsequent undercutting of the parapet wall. (Photo courtesy of David Hoffman).

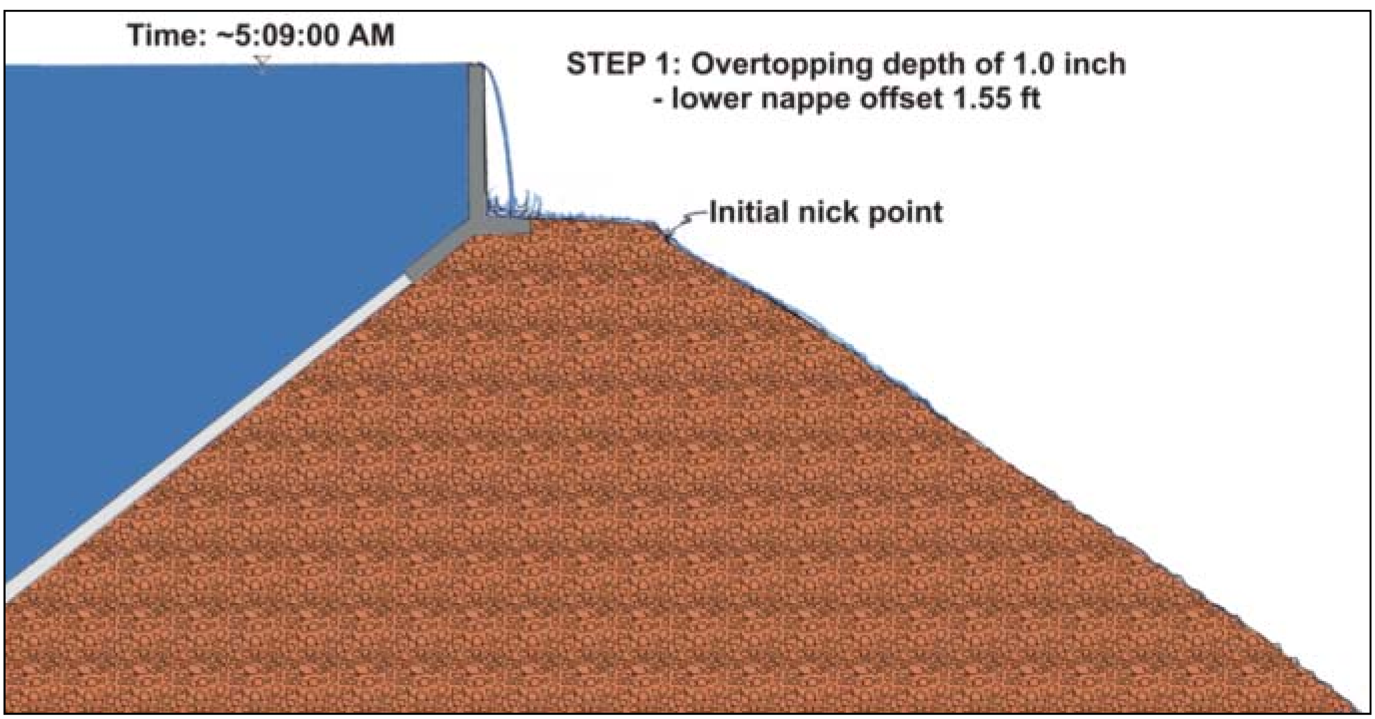

Fig 8. – The first figure from the reconstructed failure sequence shows the initiation of overtopping around 6 minutes prior to failure. Water is spilling onto the 3 foot wide parapet wall footing and just beginning to erode crest of the embankment.

Fig 9. – This photograph shows arrows representing the overflow nappe once it extended beyond the 3 foot wide wall footing and directly onto the underlying rockfill. The rate of scour and erosion increased dramatically once this occurred.

At 5:15 AM, only six minutes after overtopping is believed to have initiated, the 60-foot long segment near wall panel 95 toppled, unleashing 15 feet of flowing water over the remaining embankment and concrete liner (Figure 7).

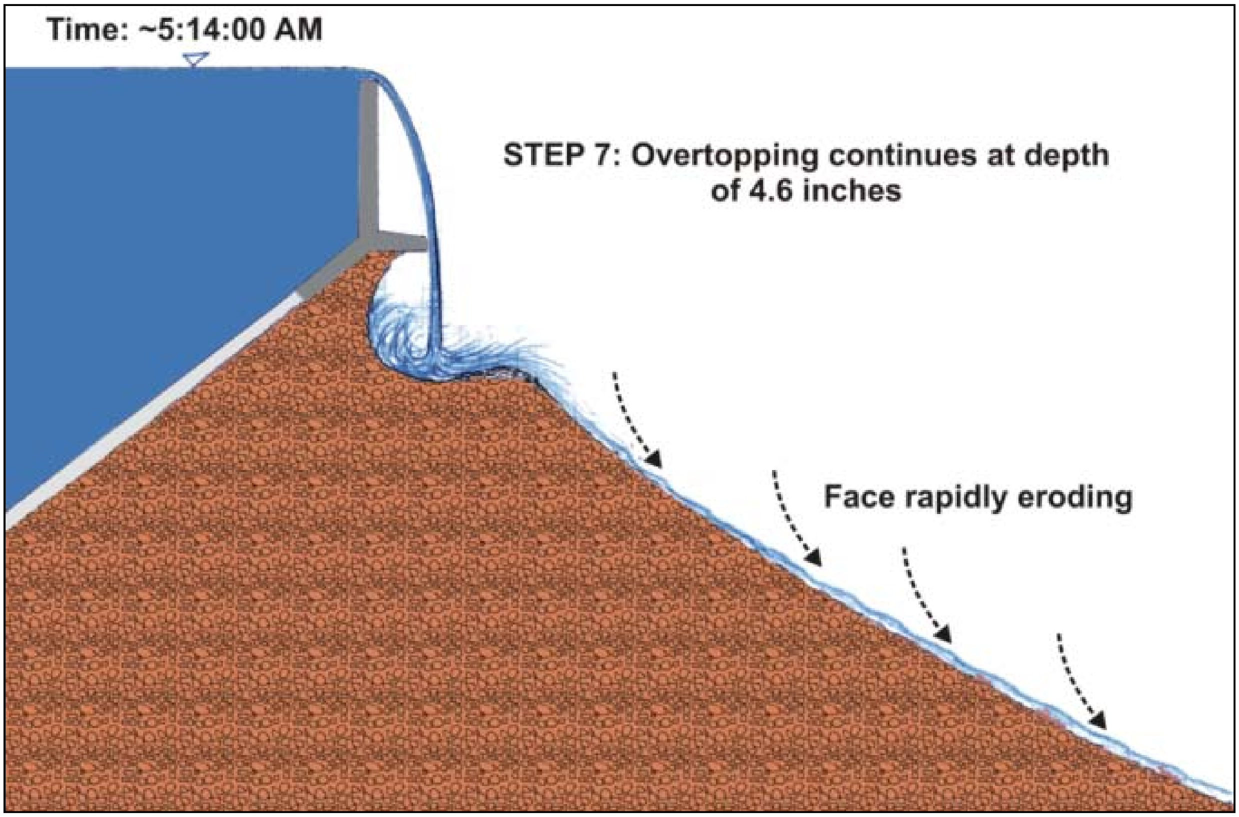

Fig 10. – This step of the reconstructed failure sequence illustrates the undermining of the parapet wall and rapid erosion of the embankment. The wall remained standing at this point due to the hydrostatic load on the inclined portion of the wall footing and 3-dimensional effects from surrounding wall panels.

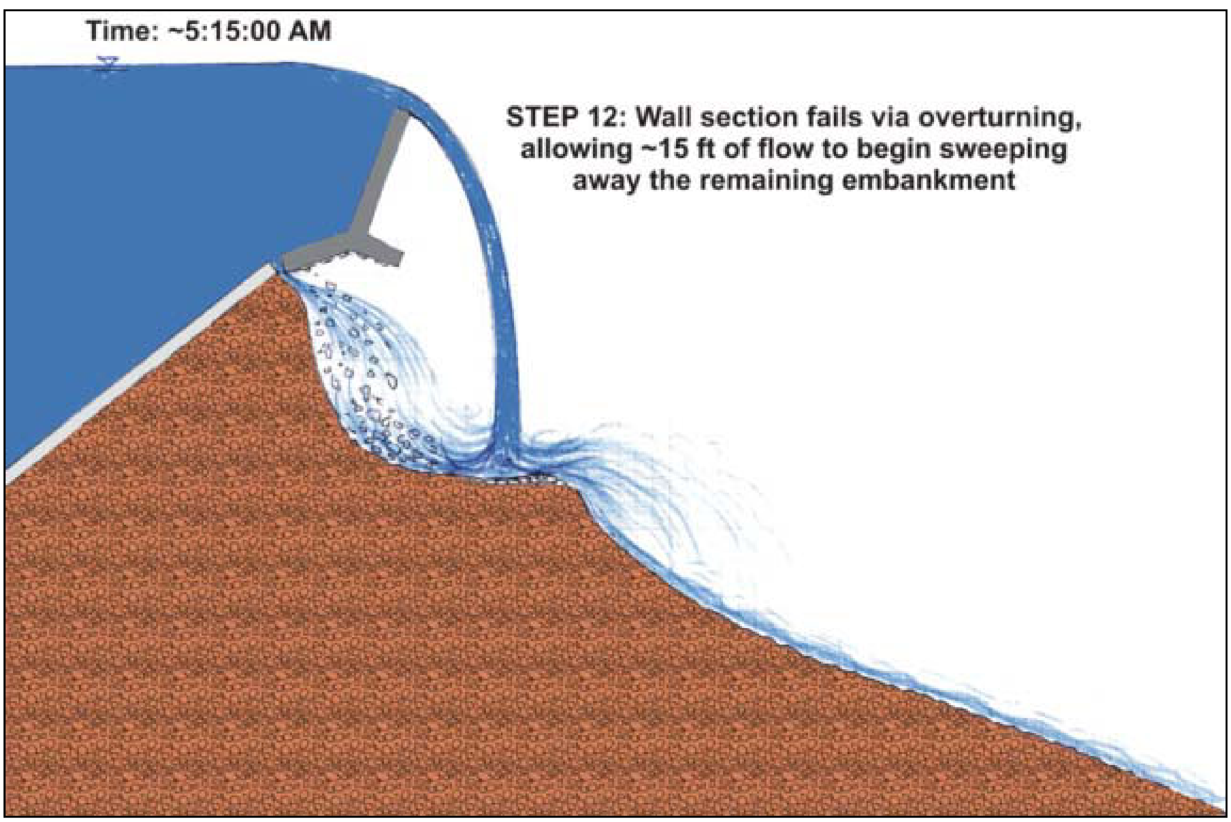

Fig. 11. – This step portrays how the parapet wall toppled in vicinity of panel 95, initiating the catastrophic failure of the reservoir by unleashing around 15 feet of flow over the remaining embankment.

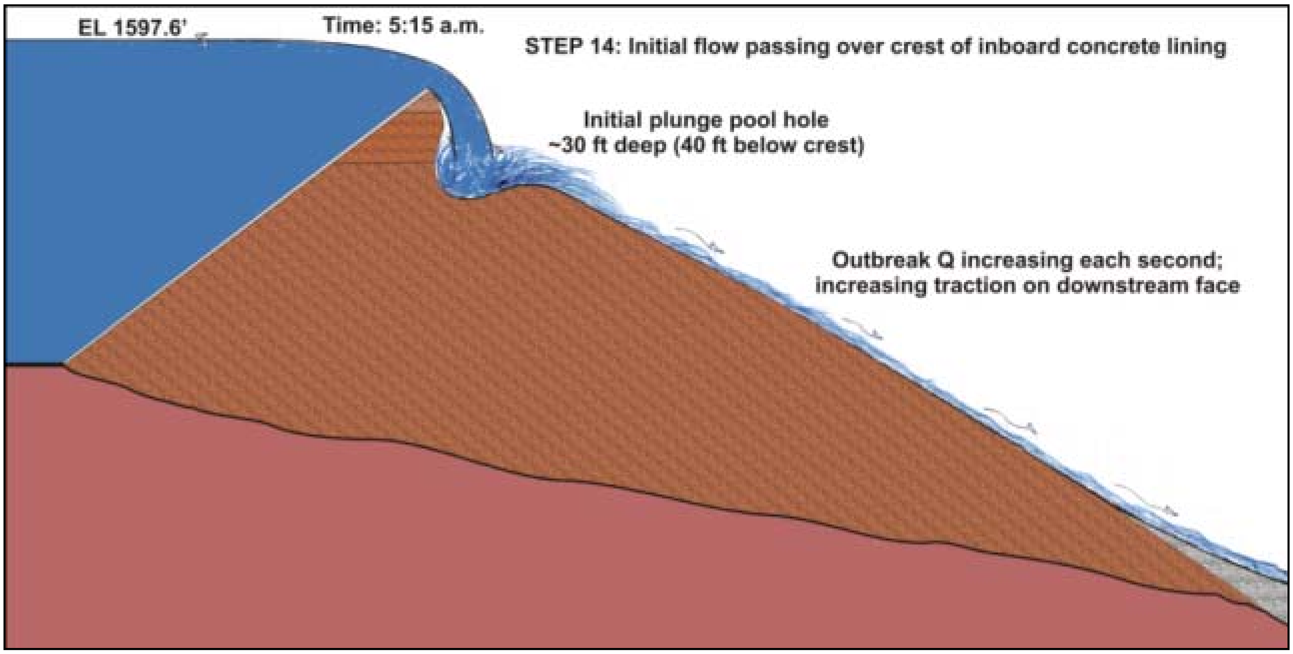

Fig. 12. – This figure illustrates ~15 feet of water overflowing the embankment immediately after the failure of the parapet. The reinforced concrete liner behaved as a thin-crested weir and allowed for the formation of a deep plunge pool.

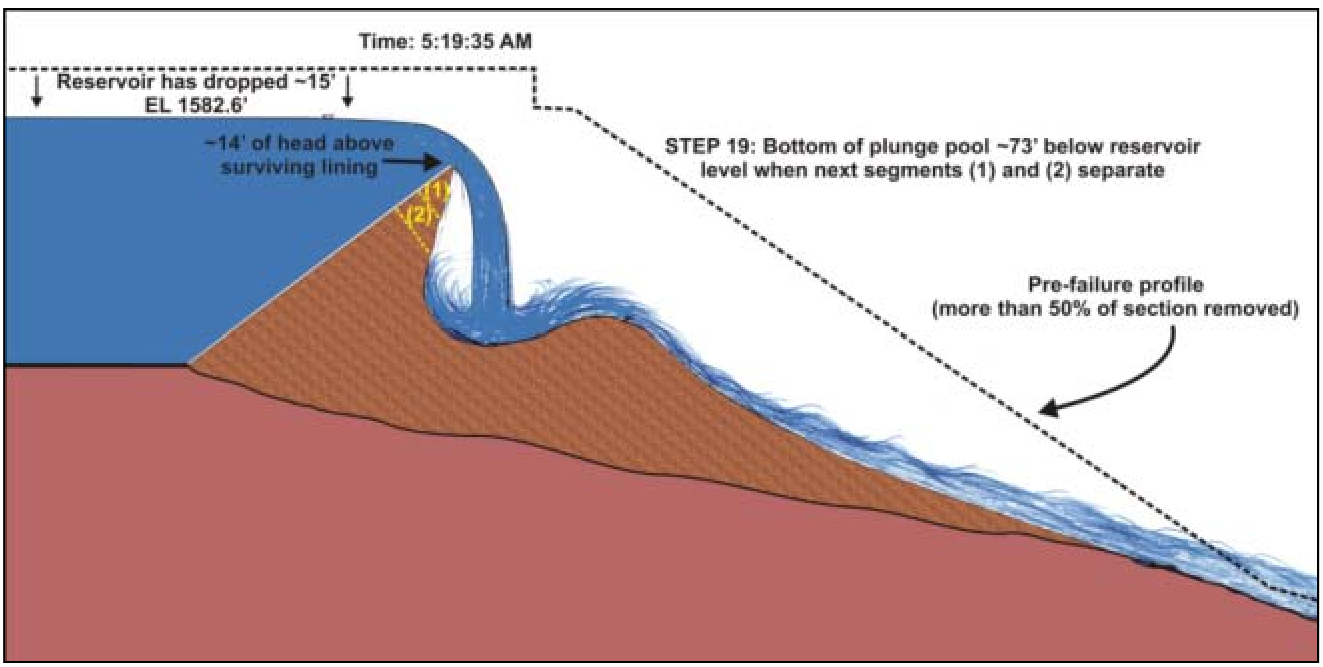

Fig. 13. – The embankment and concrete liner are progressively removed by the outflow. About one half the embankment remains at this stage.

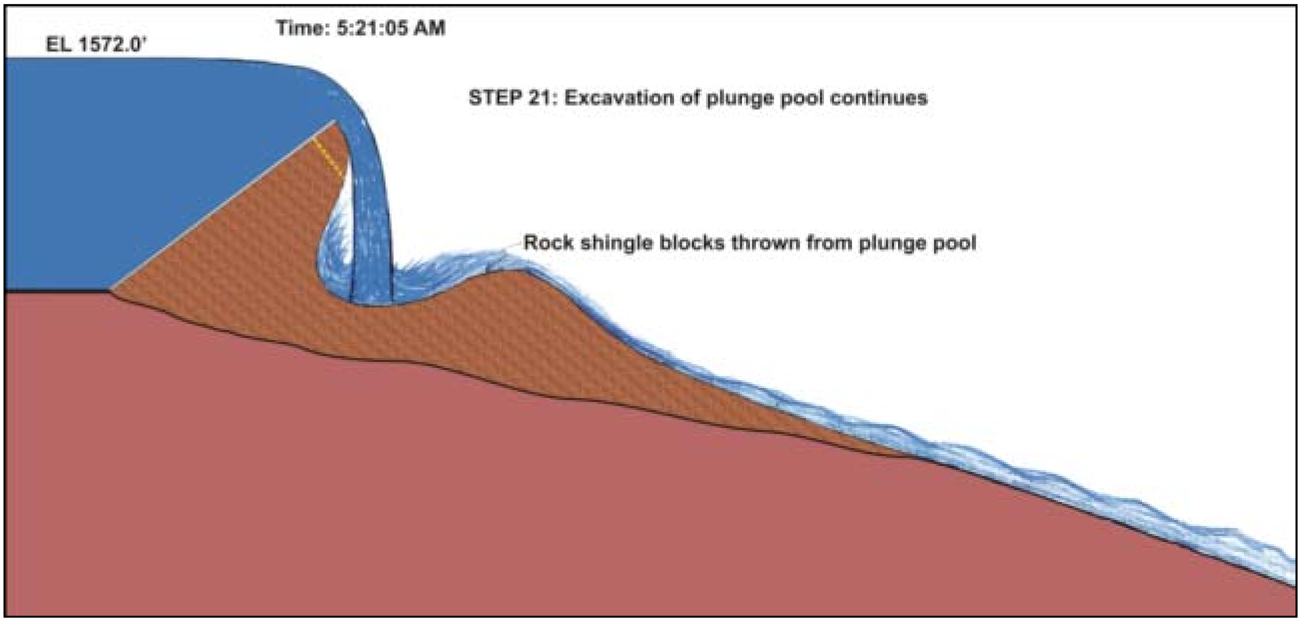

Fig. 14. – A continuation of the sequence show deepening of the plunge pool and ejection of large shingle blocks/boulders.

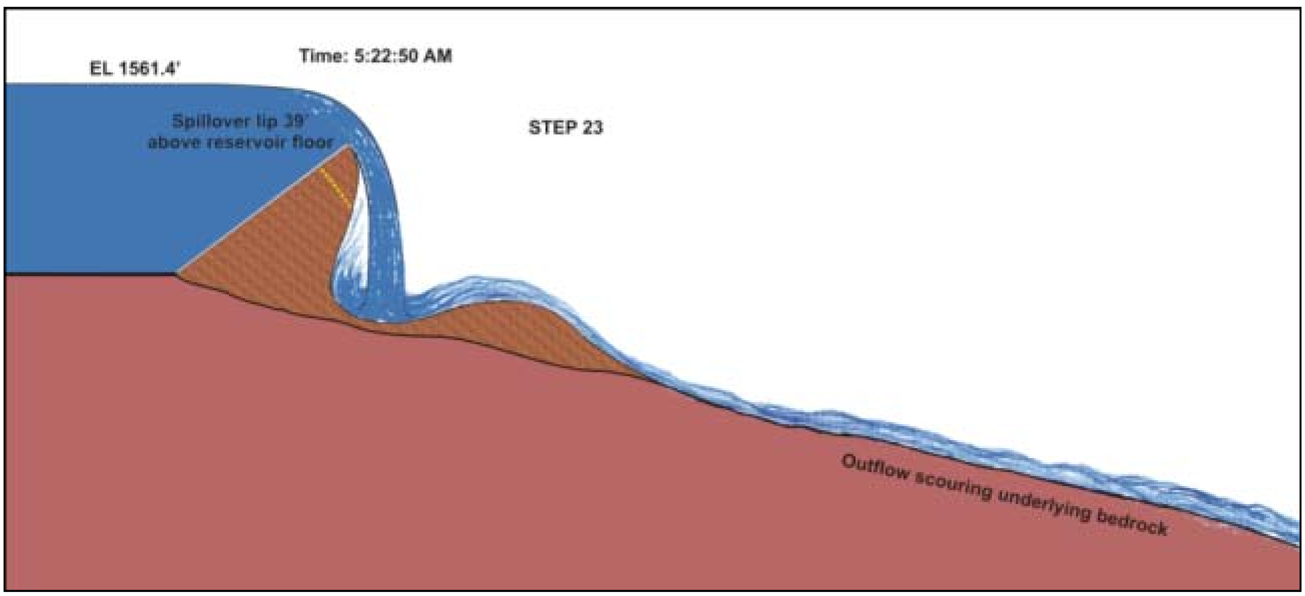

Fig. 15. – The plunge pool deepens towards the foundation interface and outflow has exposed much of the underlying bedrock, which is then scoured.

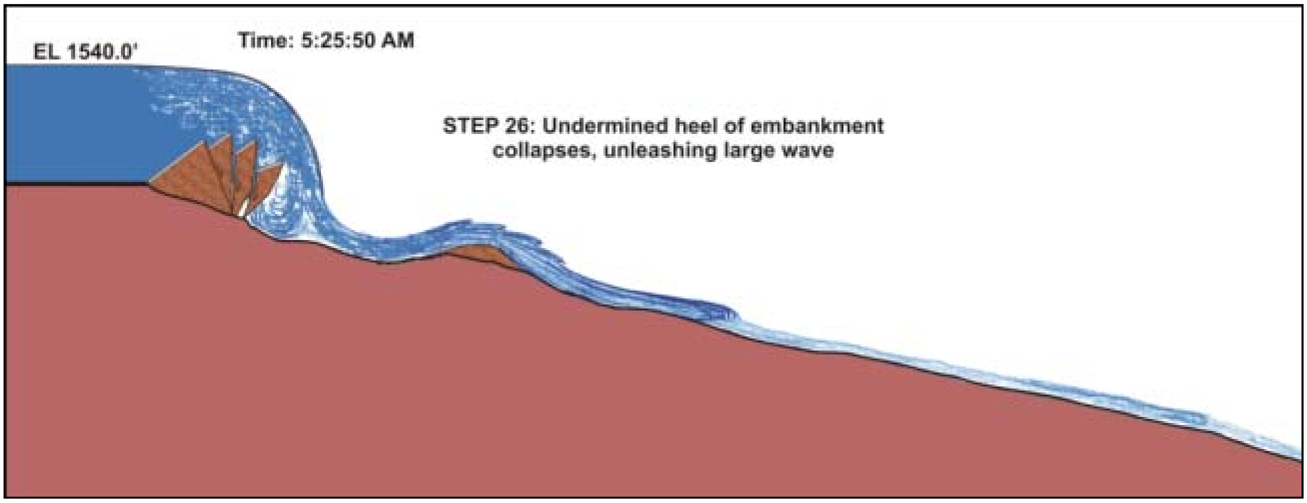

Fig. 16. – The embankment undergoes one final large collapse involving the concrete liner, sending one final surge down the slope.

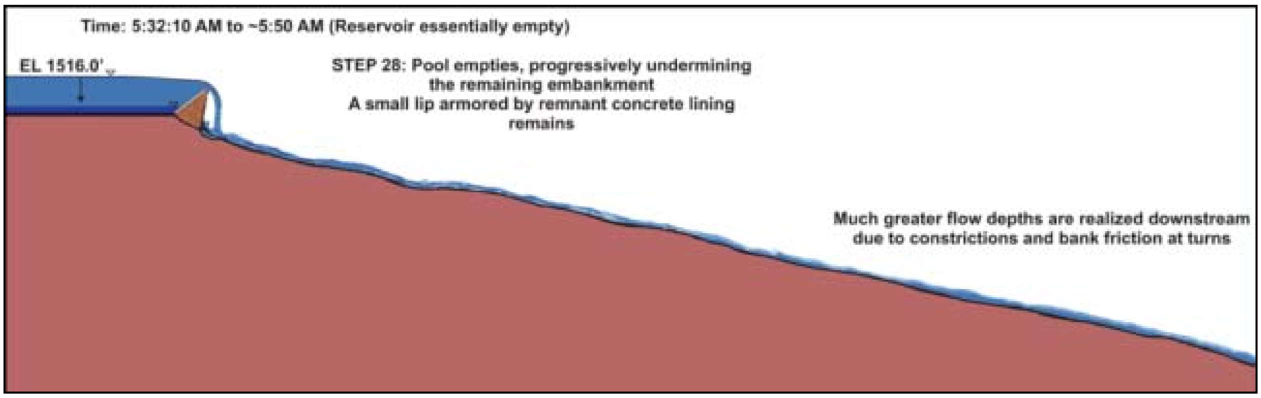

Fig. 17. – A small lip formed by the concrete liner armored the final last remnants of the embankment. Only a small amount of water remained in the reservoir.

Fig. 18. – The remnant lip as seen at the failure site.

The water surged down the upper slopes of Proffit Mountain, stripping the land of vegetation and soil into the underlying bedrock, and exposed the unique geology of the reservoir's foundation. The flow was highly turbid and included rockfill, concrete, rebar, and the geosynthetic liner along with soil/rock and hundreds of trees. The flow banked around curves with depths of up to 100 feet before entering the floodplain of the East Fork of the Black River, at Johnson's Shut-ins State Park. The park was heavily damaged and filled with debris. The flood waters passed through the narrow bedrock chute formed by the Shut-ins and continued downstream to AmerenUE's lower reservoir, where most of the discharge and debris were captured. Damage downstream was limited to increased turbidity of the Black River from silt and other fines which were carried over the Lower Taum Sauk Dam.

KEY FACTORS CONTRIBUTING TO THE FAILURE

"Dirty" Rockfill Embankment

After the reservoir failure, The Federal Energy Regulatory Commission (FERC) and the Missouri Department of Natural Resources, Dam and Reservoir Safety Division (MoDNR-GSRAD) conducted investigations into the failure. Although the embankment was intended to be clean rockfill (less than 5% passing the No. 200 sieve) an excessive amount of fine-grained material was visually recognized in the exposed dike one day after the failure (Leonard, 2005). Forensic analyses by FERC's Taum Sauk Investigation Team (2006), FERC's Independent Panel (2006), and by Rizzo Associates, Inc. (2006) confirmed a fines content of between zero and 20% in the exposed embankment bordering the breach (shown in Figs. 2 and 19). In addition, the percentage of sand sized material within the embankment was as great as 45% in certain locations, creating a far more erodible mixture than normally expected of clean rockfill (FERC Independent Panel, 2006). This problem was manifest by the unusual level of surfical erosion during the project's 42 year life, from rainfall-induced runoff (FERC Independent Panel, 2006). Cooke (1967) surmised that the dike could not have been clean rockfill because the observed settlement was almost an order of magnitude greater than those recorded on other concrete-faced rockfill dams (prior to 1967).

Fig. 19. - Close up of "dirty" rockfill exposed on lateral margins of the breach. The large clasts are 10-12 inches diameter.

Weathered Material in Embankment

The December 2005 outbreak flood exposed deeply weathered zones in the bedrock slopes of Proffit Mountain. A seam of weathered rhyolite was observed beneath the breached section of the embankment. The upper slopes of Proffit Mountain also contain a zone of deeply weathered diabase saprolite, which appears to be the remnant of a disintegrated dike or sill (see Fig. 20). Adjacent granites and rhyolites also exhibited a high degree of weathering, likely from hydrothermal alteration associated with intrusion of the diabase. The weathered diabase exhibits a soil-like texture, though retaining the rock's original fabric and fracture patterns. Core stones and remnant spheroidal weathering rinds are also visible.

Fig. 20. – A zone of deeply weathered bedrock (saprolite) on the upper slopes of Proffit Mountain appears to have formed due to the disintegration of a diabase dike or sill and nearby hydrothermally altered granites.

Insufficient Foundation Preparation

The project's design specifications called for the embankment's foundation to be stripped clean of all residuum and native soils before placement of the dumped rockfill. Remaining soils were to have been less than two inches thick and near saturation prior to fill placement.

Post-failure investigations included exploratory drilling through the embankment-foundation interface to ascertain the character of the contact. These borings revealed that up to 18 inches of residual soil were not stripped off prior to placement of the rock fill. Soils containing tree roots and other organic matter were also observed beneath the breached section of the embankment (see Fig. 21). This may have contributed to the increased settlement and shifting of the embankment in this area, due to the high fines content, poor drainage, and higher compressibility of the un-stripped residual soil cap, as well as the 'dirty' rockfill.

Fig. 21. – Residual soils, including tree roots and other organic matter, were observed left in place beneath the failed portion of the upper reservoir embankment.

Weathered rhyolite was also discovered beneath the breached section. This area had been over-excavated during construction, due to a "highly weathered zone" noticed at the time (FERC Taum Sauk Investigation Team, 2006). This northwest corner was the lowest area of the upper reservoir floor, except for a small area around the circular inlet used to fill and drain the pool. Additional filling was also required for this portion of the dike because of a natural depression in the flank of Proffit Mountain. These factors necessitated that the dike reach its maximum volumetric cross section in the northwest corner of the upper reservoir. This coincided with the area that also experienced some of the greatest settlement, over a zone about 900 feet long. The only location with greater settlement was at the juncture between panels 71-72, but this was a very narrow zone. The high percentage of fines in this portion of the dike would have exacerbated drainage and promoted settlement, due to hydrocompression (Rogers, 1998).

Issues Related to Liner Installation and Re-attachment of Instrumentation in 2004

The upper reservoir's monitoring system was anchored directly to the concrete lining prior to the fall of 2004, when it was replaced during installation of the HDPE geomembrane liner. The sensor network was comprised of four perforated HDPE conduits. The original design assumed two of the conduits would house pressure transducers; with another conduit serving as an extra; and a fourth conduit to be filled with concrete, to serve as ballast for all four, since they would be subjected to almost daily reservoir cycling. The original design specified that the instrument conduit array would be anchored to the new HDPE liner using welded HDPE straps. The contractor pointed out that this design could reduce the expected life of the liner by creating stress concentrations around the attachment points, and they suggested that the conduits should be attached to the concrete face beneath the liner (Rizzo Associates, Inc., 2006).

The alternative scheme used a pair of untensioned steel cables passing through eye bolts anchored to the concrete lining, above and below the HPDE lining. For unknown reasons, it was also decided to dispense with the concrete-filled ballast conduit, so this pipe was installed as another spare, and the entire array was bereft of any meaningful ballast.

The eye bolt scheme was then discarded in favor of turnbuckles, so the anchor cables could be tensioned and adjusted, after the array was in place and subjected to cyclic loading by the reservoir pool. Unfortunately, the turnbuckles were not locked after being tensioned, and they appear to have loosened themselves during cyclic loading engendered by the filling and emptying to the reservoir each day. The four sensor conduits were attached every 20 feet with hardware assemblies known as unistruts. A unistrut is a series of galvanized U-bolts fastened to a flat galvanized steel bracket. There were four U-bolts over the conduits at each unistrut anchor point. The cyclic uplift loads caused by near-daily reservoir filling and draining gradually loosened the instrumentation conduit array and the conduits worked themselves free of their unistrut anchors. The omission of ballast allowed much higher uplift forces to be realized by the sensor conduits, as air and water was trapped inside of them. The two anchor cables were unable to keep the array aligned because their turnbuckle anchors loosened.

As the turnbuckles failed, the unistruts were subjected to additional cyclic stresses and began coming apart. Once the four conduits were no longer attached to each other, they began to deform individually, instead of collectively. Since the individual stiffness of the conduits was less than their cumulative stiffness, the failure of the unitstrut assemblies played an important role leading to the eventual failure of the instrumentation system.

The base of the instrumentation array was also located approximately 120 feet from the 35 foot diameter glory hole inlet, which pumped water into the reservoir at a rate of 5,238 cfs. Vortices associated with this concentration of flow in and out of the inlet may have induced local currents and engendered traction forces on the lower unistrut anchors, in addition to the reservoir cycling. It is believed that sensor readings were erroneously low, between slightly more than 3 ft and as much as 4.2 ft, due to these problems (FERC Independent Panel, 2006).

<u>Elevation Datum/Staff Gage discrepancies introduced during liner installation</u>

The upper reservoir was designed to have two feet of freeboard between the water surface and the top of the parapet wall. A staff gauge installed on the inside of the concrete parapet wall at Panel 58 and sloping interior reservoir face during construction had settled approximately one foot over 42 years. The old gauging system was operated relative to this staff gage at Panel 58, so freeboard at the staff gage remained constant throughout the years, even as the gage settled. Unfortunately, differential settlement elsewhere around the reservoir was greater. The new gauging system was operated in terms of absolute elevation, which was one foot higher than the elevations stated on the staff gage. This resulted in a one foot reduction of absolute freeboard, and, thereby, lowered the margin of error against overtopping.

<u>Error in Location and Programming of “Fail Safe” Probes</u>

The upper reservoir’s “fail-safe” Warrick probes, which were intended to shut down inflow whenever the reservoir rose to within 2 feet of crest failed to activate after the overtopping initiated, and were unable to save the structure. These probes were located on the parapet wall at Panel 58, ABOVE the lowest points along the parapet wall at four other locations around the reservoir. FERC’s Taum Sauk Investigation Team (2006) states that the two Warrick probes were at elevations 1597.4 ft (Hi probe) and 1597.67 ft (Hi-Hi probe), respectively. The lowest point along reservoir’s parapet wall was at panel 72, with a crest elevation of 1597.0 ft., below both Warrick probes (FERC Independent Panel, 2006). Although panel 95 was destroyed in the failure, its crest had an estimated height 1597.25 ft. (Rizzo Associates, Inc., 2006), also below the Warrick probes. The auto-stop probes failed to activate during the two overtopping incidents in late September and this fact appears to have been overlooked prior to the Dec 14th failure. The Warrick probes were kept as high as possible to prevent false alarms and shut downs, due to wind-whipped waves, but were mistakenly placed well above the lowest points on the parapet wall. These could easily have been moved to lower elevations on the parapet wall at panel #58.

The probes were also programmed with a one minute delay before alarms would sound and the pumps would shut down (FERC Independent Panel, 2006 and FERC Taum Sauk Investigation Team, 2006). This delay would have allowed between 0.75 and 1.5 inches of additional water to be pumped into the reservoir, depending if one or both pumping units were running at the time. (FERC Taum Sauk Investigation Team, 2006).

The Warrick probes were also programmed to operate in series instead of in parallel, which only triggered the auto-stop system to activate if both probes had been activated for the programmed time (60 seconds). Although the Hi limit probe was located 4.92 inches above the lowest point along the crest of the parapet wall, it could have kept an additional 3.6 inches of water from flowing over the wall, provided that the probes had been programmed in parallel and with a 10 second delay (FERC Taum Sauk Investigation Team, 2006).

Although it did not play a role in this failure, an additional programming error was uncovered during the post-failure investigations. The Programmable Logic Controller at pumping unit 2 had been mistakenly programmed so that it couldn’t read input from either of the Warrick probes (FERC Taum Sauk Investigation Team, 2006). Pumping unit 2 had already shut down normally prior to water levels reaching the Hi limit probe, but unit 1 continued to pump, resulting in the failure. Had the Warrick probes been positioned below the lowest point(s) along the crest of the parapet wall and programmed with a proper delay, this error could have resulted in an identical failure had pumping unit 2 been set to shut down last.

Water may have reached as high as 1597.7 feet due to the combination of the excessively high probe elevation, the programming of the probes to operate in series, and the programmed one minute delay in their activation (FERC Taum Sauk Investigation Team, 2006).

Administrative Procedures

AmerenUE had no formalized oversight to oversee modifications to the reservoir's instrumentation and documentation on such changes was lacking to non-existent. There was also no formalized procedure to test such changes to insure they were properly implemented. Nor was there any documentation rationalizing the decision to program the probes in parallel or with a 60 second delay (FERC Taum Sauk Investigation Team, 2006).

The reservoir was routinely filled to within 1 foot of the parapet wall crest, providing an exceeding low margin of error, as compared to other pumped storage facilities in the United States (which usually operate with freeboards between 3 and 5 feet). This low margin of error was exacerbated by differential settlement of the parapet wall, which allowed four other zones to be about a foot lower than assumed by the plant operators (FERC Independent Panel, 2006). Visual oversight of the pumped storage operations were recommended by Cooke (1967) and initially implemented by Union Electric soon thereafter (Weldy, 1968). Sometime between 1968 and the failure in 2005, visual oversight was discarded as being an unnecessary precaution by the operators (probably, because there hadn't been any safety incidents of note until the Niagara Falls incidents in September 2005). The absence of visual inspections meant that the deterioration of freeboard (due to progressive creep displacement of the instrumentation conduits) was not noticed until the first overtopping incident on September 25, 2005. At this juncture the actual water levels should have been "ground truthed," or compared with the levels being reported by the reservoir's instrumentation (FERC Independent Panel, 2006). Instead, it was assumed that increasing the freeboard by three feet would provide an adequate margin of error to account for the instrumentation problems.

A retrospective review of the reservoir stage records suggests that something was awry with the instrumentation because it repeatedly shows water levels that do not make sense, based on the conditions prior to the failure. Some examples include: 1) the water level within the reservoir not rising when both pumping units were on; 2) the level rising 1 foot in 20 minutes with both pumping units on (it should have reported a 2.5 foot rise), and, 3) a 1.9 foot decrease in the reservoir level with both pumps operating. The system was not programmed to report or flag abnormal inflow rates to alert plant operators although it was recorded in the facility's computers (FERC Taum Sauk Investigation Team, 2006).

IMPACTS OF THE FAILURE

During the spring 2006 and 2007 legislative sessions, the Missouri governor and state legislature considered revising their dam safety act (initially adopted in 1977, but not funded until 1981) to improve inspection and maintenance of dams deemed to be a danger if they were to fail (e.g. lying above populated areas). Some legislators from rural counties and agricultural areas worried about increased costs associated with regulations so they voted against the bill, defeating the measure.

AmerenUE examined its internal policies and pledged to make changes in its operating and maintenance procedures to prevent future problems. A full-time dam safety officer has been hired to oversee all hydropower-related projects within the company. This official has been given the authority to shut down any hydropower facility due to safety concerns. His authority supersedes other decision makers in the company's chain of command.

AmerenUE is paying for 100% of the clean-up and repair at Johnson's Shut-ins State Park. FERC approved AmerenUE's plan to rebuild the upper reservoir and the utility recently completed the rebuild, successfully renewed its FERC permit, and resumed operations in the spring of 2010. The utility is involved in a lawsuit and investigation by the Missouri Attorney General's office. In November 2007 another suit was filed against AmerenUE by the Great Rivers Environmental Law Center alleging that FERC has failed to properly monitor the reconstruction project. Both of these suits have slowed the approval process. AmerenUE reconstructed the upper reservoir from scratch using a roller compacted concrete (RCC) embankment with a similar footprint, height and capacity to the original structure, creating the largest RCC dam in North America (see Fig. 22 for view of the rebuilt structure). Much of the aggregate material from the failed structure was washed, crushed, and resorted to make it suitable for reuse in the RCC mix. Sections of the foundation containing deeply weathered zones were over excavated and replaced with concrete and the footprint of the structure was altered slightly to avoid other such zones. The rebuilt structure includes a spillway discharging over the eastern side of Proffit Mountain in case of another overtopping. This location discharges onto uninhabited lands that the utility owns and controls and in a direction away from Johnson's Shut-in State Park. Additional measures will also be undertaken to reduce the risk of overtopping.

Fig. 21. – Overlooks within Johnson's Shut-ins State Park provide excellent views of the scour path formed during the failure and rebuilt structure. This photograph shows the rebuilt RCC upper reservoir during the spring of 2010, just prior to the resumption of operations at the facility.

RECORD FINE BY FERC

The Federal Energy Regulatory Commission fined AmerenUE $15 million; the largest fine ever accessed by the agency and 30 times larger than the previous record fine of $500,000. FERC assessed its record fine based on the following aspects (FERC, 2006):

- Failure to report the Sept. 25, 2005 overtopping to FERC
- Failure to report unusual instrument readings on Sept. 27th
- Failure to report the release of the transducer retention system
- Addition of 0.4 feet to the water level in the programmable logic controller to compensate for inaccurate readings
- Failure to repair the loose transducers
- Operation of the reservoir with insufficient freeboard
- Fail-safe probes moved to an elevation higher than the lowest point on the reservoir parapet wall
- System programmed to have a 1 minute delay in pump shutdown after activation of probes
- Probes reprogrammed to operate in series instead of in parallel
- Lowest of two probes not programmed to sound alarm when activated

All of the listed modifications to the facility required AmerenUE to notify FERC prior to such changes being implemented.

DISASTER COULD HAVE BEEN MUCH WORSE

The results from this failure could have been far worse had it occurred at a different time of year. Hundreds of unsuspecting campers would easily have perished in the state park campground had the failure occurred just six months later, on a busy summer weekend. The timing of the overtopping failure towards the end of its nightly filling cycle (around 5:15 AM) would have caught campers in their tents and recreational vehicles, and seasonal park staff in their nearby cabins. Fortunately, the campground was empty during the middle of December, resulting in no deaths and just five injuries.

PRIMARY FACTORS CAUSING THE FAILURE AND LESSONS LEARNED

Although multiple factors contributed to the disaster, these might never have culminated in a catastrophic failure had a conventional spillway system been included in the original design, or a subsequent retrofit. Everyone in the chain of corporate decision making seemed to assume that the three foot adhoc adjustment to the reservoir stage levels would easily account for any deficiencies for another 6+ months of operation, without quantifying the actual errors or verifying the failure mechanisms causing the erroneous readings. The two Niagara Falls incidents should have triggered more in-depth investigations and assessments of the problem, not to mention reports of the incidents and investigations to FERC. Any engineered system is capable of malfunctioning for an array of reasons, including aging and/or unforeseen circumstances. Many of the facility's shortcomings were adroitly pointed out in the first FERC peer review in 1967 (Cooke, 1967).

The impact of the differential settlement of the dike should have been appreciated by whoever was responsible for reservoir stage instrumentation. The dike is only as "high" as its lowest elevation; not the crest elevation where the instruments are located. Aging impacts are some of the most difficult to appreciate and/or anticipate, especially, if they have never been encountered previously by the personnel charged with making operational decisions.

The change orders allowing the instrument conduits to be affixed to the unistrut anchors without any ballast would not have fared well had they been subjected to external peer review. This is because pumped storage projects are subject to much more severe load cycling than conventional storage facilities. Last minute connection details often prove to be problematic. Hidden design and construction flaws can often cause unforeseen difficulties with operation and maintenance throughout the life of a reservoir.

The overflow incident on September 25th and 27th should have triggered an active monitoring program at the very least, to ascertain whether the problem was worsening with each cycle of filling.

In conclusion, the principal contributing factors appear to have been a series of errors in human judgment. It is estimated that only six minutes of malfunction fomented the catastrophe. Once the sensor problem was identified, a worker could have been hired to observe the reservoir level during the few critical minutes when the reservoir was topping off its nightly refill. Critical engineering systems with the ability to endanger life, property, and the environment should employ sufficient redundancy to survive the failure or malfunction of any single component, without suffering a catastrophic failure. The Warrick gages affixed to the parapet wall were intended to provide such a "fail safe" backup.

As in the case of most systems failures, this project could have benefited immeasurably from periodic external peer review by a mixed panel comprised of people with substantive experience with the operation of pumped storage projects; with particular reference to the instrumentation scheme.

As with almost all major engineering disasters, multiple factors contributed to the failure of the AmerenUE Taum Sauk Upper Reservoir. Although the project contained design and construction flaws for its entire operating life, it was able to operate for 42 years. Human factors played a key role in the failure. Had the facility been shut down and fully inspected after the faulty gage readings and overflow incidents were observed, and appropriate repairs conducted, the catastrophic failure would have likely been avoided.

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