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EVALUATION OF THE TAUM SAUK UPPER RESERVOIR FAILURE

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ABSTRACT

The Taum Sauk Upper Reservoir was built in the early 1960's as a water storage reservoir for hydro-electric energy production. The reservoir was created by blasting rock from the top of Proffit Mountain. The rock debris generated was then used to construct a kidney shaped earthen embankment atop the mountain. The reservoir is located in Reynolds County near the town of Lesterville, Missouri. The Upper Reservoir was approximately 95 feet in depth and covered a surface area of roughly 55 acres. The reservoir had the capacity to hold nearly 1.5 billion gallons of water. The Upper reservoir underwent a catastrophic failure on the morning of December 14th, 2005 releasing most of its stored water down the northwest side of Proffit Mountain. An approximate 700-feet wide breach occurred along the northwest radius of the rock-fill dike, causing severe damage to state park property. No fatalities resulted from the failure events. This paper evaluates different failure mechanisms of the reservoir based on three distinct failure hypotheses. Conclusions are then made based on the analyses and discussion of each mechanism, categorized in terms of the likely contribution to the Taum Sauk Upper Reservoir failure.

INTRODUCTION

During the early morning hours of December 14, 2005, the Taum Sauk Upper Reservoir rock-fill dike underwent a catastrophic failure, releasing more than a billion gallons of water down the northwest side of Proffit Mountain. The release of water downstream ripped the land of existing vegetation and soil cover and the resulting wall of water demolished one residence directly in the flood path. Severe flash flooding was experienced on the East Fork of the Black River, including Johnson's Shut-Ins geological features and Johnson's Shut-Ins State Park campground. Figures 1 and 2 illustrate the consequences of the water release from the Upper Reservoir. Figure 1 displays downstream sedimentation and a portion of the downstream flow path. Figure 2 displays the residence that was demolished by the flash flood waters.

The breach occurred along the northwest radius of the Upper Reservoir over a distance of about 680 feet at the crest and through the entire height of the dike. Figure 3 illustrates the breach area of the rock-fill dike. Water began overtopping the reservoir on the morning of the failure. Overtopping at specific locations around the perimeter of the Upper Reservoir and possible failure mechanisms associated with the failure events after initiation of overtopping, are within the scope of this study. The cause of overtopping is of importance but is not within the scope of this evaluation.

The Taum Sauk power plant is located in Reynolds County, Missouri, approximately 90 miles southwest of St. Louis, Missouri, in the Johnson's Shut-Ins Quadrangle. The power plant facilities are approximately 4.5 miles northwest of Lesterville, Missouri, and approximately 1 mile east of Highway N.

Fig. 1. The upper reservoir breach and a portion of the flow path (photo courtesy of J. Spooner, 2005)

On December 15, 2005, select NHMI members embarked on a preliminary site reconnaissance in response to the Taum Sauk Upper Reservoir failure. Access to the site was limited; however one member of the team was granted access to the Upper Reservoir area and allowed to document what he witnessed. Other team members were involved with mapping the general flood water extent in the lower regions of the flow regime near Missouri State Highway N and the entrance to Johnson's Shut-Ins State Park. All members witnessed first hand the effects that the flood waters had on the surrounding landscape and also realized the destructive power of such an event. Figures 1 through 3 are photographs taken during the initial site reconnaissance. Additional details related to the site reconnaissance were published by Witt and Hoffman (2005).

Fig. 2. Remnants of the Park Ranger's residence (Hoffman, 2005).

Fig. 3. View of breach looking northwest across reservoir (Hoffman, 2005).

Several efforts were made to collect data for this study. The USGS Mid Continent Geographic Science Center (MCGSC) contracted an aerial survey on December 16, 2005. The aerial survey used Light Detection and Ranging (LiDAR) to capture the site topography after the failure events. The LiDAR data was made available to the University of Missouri – Rolla for this evaluation. Additionally, the MoDNR conducted an elevation survey to measure the remaining concrete parapet wall panels. This survey was referenced to an existing benchmark to compare data with previous surveys. These elevation data were made available for this evaluation and are presented in Appendix B of Gehring (2006). Although information and data were not easily accessible, an adequate amount was collected and key assumptions were made that allowed the progression of this evaluation.

The scope of this evaluation involved the post failure reconnaissance visit to the Upper Reservoir system and analytical studies based on data published by others. It included development of several failure mechanism hypotheses, analyses of the stability of the structural and geotechnical components under the assumes loading conditions, discussion of the relevance of each proposed failure mechanism, and classification of each proposed failure mechanism considering their respective probability of occurrence. In order to understand this grand failure case history a thorough description of the geological, geotechnical and structural conditions at the site are necessary.

BACKGROUND

Geologic Setting

The St. Francois Mountains region is a unique area consisting of Missouri's oldest landscape. During the Precambrian time igneous granite rock formed as a molten magma crystallized deep within the earth's surface. Volcanoes closer to the surface also began to erupt large quantities of pyroclastic flows and rhyolitic lava (Unklesbay and Vineyard, 1992). Thick layers of pyroclastic materials were deposited throughout the region as either air fall or ash flow tuff. Residual heat from the eruptions often melted or "welded" the pyroclastic ash fragments together and cooled to form a hard igneous rock known as welded tuff. 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. Many large bodies of reddish to grayish granite are included within this material.

After the decrease and eventual halt of volcanic activity during the Precambrian time, the area was subjected to the Ozark dome uplift (Unklesbay and Vineyard, 1992). The uplift, as well as erosion, formed the igneous knobs and ridges common to the St. Francois Mountains of today. When the Cambrian seas began to rise, much of the region became blanketed by water, leaving the igneous knobs and ridges as highpoints or islands. Deposition of sedimentary rocks during this period left thick layers of sandstone and carbonate sediments on the sea floor that draped the slopes of the igneous highpoints and knobs.

Regression of the Cambrian seas exposed the younger sedimentary deposits and the igneous highpoints. Erosion and weathering of the Cambrian rocks cut distinct drainage patterns through the sedimentary deposits. Present day drainage patterns preferentially cut through sedimentary deposits down to underlying steep granite ridges. These ridges resist the effects of weathering and erosion more 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 called shut-ins. Johnson's Shut-Ins on the East Fork of the Black River is an example of this type of feature, and is located below the Taum Sauk Upper Reservoir. As with the most of the Ozark Plateau, the St. Francois Mountains were not glaciated during the Pleistocene. Lack of glaciation preserved many ancient, deeply weathered zones of bedrock and soil, which are locally present throughout the region.

Taum Sauk Upper Reservoir

The following is a summary of the components that comprise the Taum Sauk Upper Reservoir. The information was obtained from a report provided by AmerenUE and made available soon after the failure (Rizzo, 2006). This evaluation focuses on the upper reservoir at the top of Proffit Mountain. Figure 4 illustrates an aerial view of the Upper Reservoir with locations of various components discussed below. The upper reservoir dike is about 6,500 feet long and closes to form a kidney-shaped reservoir. The concrete-faced dumped rockfill dam (CFRD) had a maximum height in the range of 85 feet above the base of the reservoir. The base of the reservoir is around elevation 1505 feet. At the top of the CFRD a 10-foot-high concrete parapet wall rests on the inside edge of the 12-foot wide crest. Survey data indicate that the parapet wall panels have settled over time and at some locations are as much as 2 feet lower than the design elevation.

A horseshoe shaped access tunnel exists through the northern side of the dike and provides access to the reservoir floor. The upstream face was fitted with a hinged steel bulkhead gate that opens into the reservoir. The inlet/outlet conduit consists of a 451 feet deep, 27.2 feet diameter vertical shaft shaped at the top as a typical 'morning glory.' The top 110-feet of the shaft is concrete lined and connects to a 4,765 feet long, 25 feet diameter, unlined horseshoe tunnel. The horseshoe tunnel ties into a steel lined horizontal tunnel roughly 1,807 feet long and 18.5 feet in diameter. A short penstock then splits the steel lined tunnel and directs flow to the pump-generating plant. (Rizzo, 2006)

Spillways are typically specified to protect dams, dikes, and reservoirs from the potential effects of overtopping water. The Taum Sauk Upper Reservoir was designed without a spillway system. One possible reason for not specifying a spillway system for the reservoir is the lack of topographic drainage area contribution to the total inflow into the Upper Reservoir. The reservoir's only contributions to inflow were from the intended pump-back procedures for filling and direct rainfall on the Upper Reservoir itself. This means that overtopping could occur from over-pumping/overfilling or excess rainfall with the latter being highly improbable. Furthermore, to prevent overtopping of the rock-fill dike by over-pumping, a system of redundant water level instruments was specified in the design.

Figure 4. Aerial view of Upper Reservoir.

The Taum Sauk Upper Reservoir was constructed in the early 1960's. It is important to note that design and construction practices today were not the state-of-the-practice in the early 1960's. The following is a partial list the specifications incorporated into the construction of the Upper Reservoir (Rizzo, 2006).

1. Rock was dumped and re-positioned by sluicing with water (jets) from monitors
2. Fines were removed by sluicing after rock was dumped into position
3. Foundation was prepared by removal of most deleterious materials by dozers. A note on the Drawings that applies to the 70 feet nearest the upstream toe reads as follows:

   "Strip to sound rock with not more than 2 inches (average) of dirt. This dirt to be thoroughly saturated before placing."

   "Remove topsoil and loose, unstable, altered material as far as possible with bulldozer"
4. Parapet walls were used to retain water on an "everyday" basis
5. Grout curtain was installed to a depth of about 20 feet. There is no evidence of the design basis.

No conclusions based on adequacies of construction practices and/or procedures are addressed herein, but it is important to note the dam remained functional throughout its 40 plus years of service.

Remediation and Upgrades

Initial filling of the Upper Reservoir occurred in 1963. Leakage from the Upper Reservoir was a re-occurring problem and concern since the initial filling. For example, at one time a sudden increase in seepage was experienced and emergency measures were taken to remediate the situation. The remediation required plugging two holes in the reservoir floor with concrete. Another episode of increased leakage occurred three days later resulting in complete shut down and mandatory repair. The repair required excavating a long trench down to bedrock and then backfilling the excavated trench with concrete. Throughout the following years a number of repairs were made focusing more on leakage through the horizontal and vertical joints in the concrete liner with an emphasis placed on the joints between the concrete liner and bedrock, the joint at the upstream toe of the parapet wall, and the joint between the concrete liner and the plinth (FERC,2006).

The Taum Sauk power plant began to be used more extensively after increasing plant efficiency in 1999. After increasing plant efficiency, higher rates of leakage were witnessed until the installation of a geosynthetic liner in 2004. The liner was placed over the original upstream concrete liner. The geosynthetic membrane on the interior side slopes was 80 mil thick High Density Polyethylene (HDPE). During the 12 months prior to the failure events and after the installation of the geosynthetic liner, observed leakage was significantly reduced (FERC, 2006).

HYPOTHESES OF FAILURE MECHANISMS

#1 - Rising Phreatic Surface and stability failure

The following summarizes failure mechanism #1: As overtopping of the Upper Reservoir rock-fill dike was initiated, water spilling over the concrete parapet wall infiltrated into the dike. The resultant increase of water within the dike subsequently caused the phreatic surface to rise within the rock-fill. The rising phreatic surface exerted excess pore pressures within the rock-fill material, consequently reducing the effective stresses acting on the bedrock and rock-fill interface. The reduced effective stresses lowered the shear strength of the rock-fill material. The reduction of effective stresses caused either a global slope stability or localized toe slope stability failure on the downstream slope of the Upper Reservoir.

If a localized toe stability failure occurred due to the aforementioned reduction of effective stresses, the global stability of the reservoir would be sufficiently weakened to the point of global slope stability failure. The capacity of the rock-fill dike to hold back the contained water, after either case of slope failure, would have been severely compromised. Complete failure due to a rising phreatic surface and reduced slope stability could have resulted. Figure 5 illustrates sketches of each stage of the proposed failure mechanism.

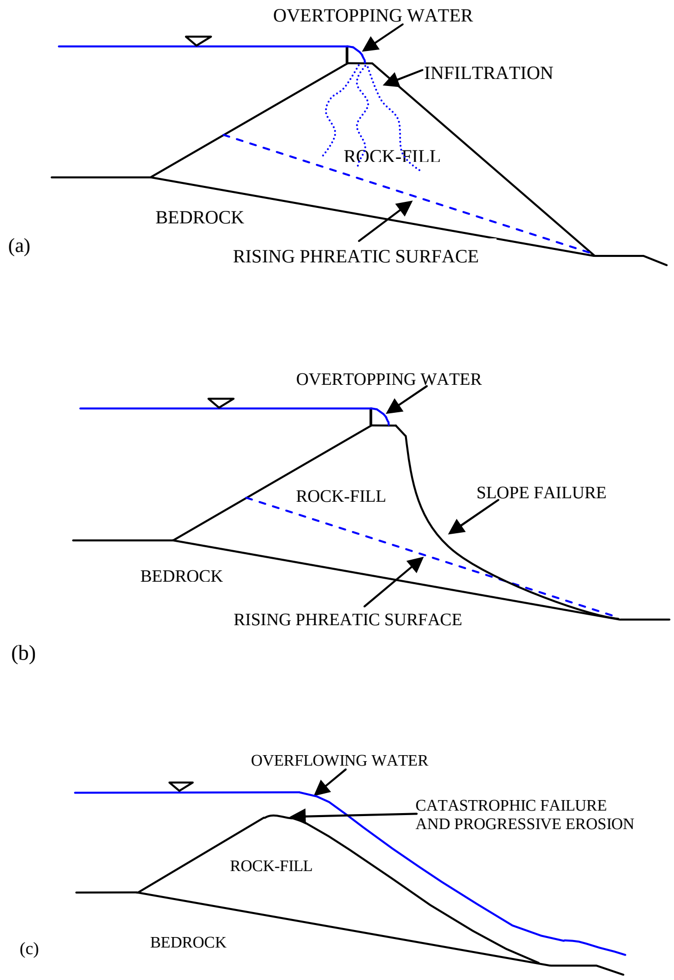

Fig. 5. Failure mechanism #1.
(a) rising phreatic surface, (b) slope instability, (c) catastrophic failure with progressive erosion (not to scale).

<u>#2 - Downstream Toe Erosion and stability failure</u>

The following summarizes failure mechanism #2: As overtopping of the Upper Reservoir was initiated, water spilling over the concrete parapet wall was allowed to flow down and along the downstream slope surface of the dike. If the water flowing along the surface reached the toe of the slope with a sufficient flow rate, a hydraulic jump could have been formed. Once a hydraulic jump formed, erosion could have been initiated at the downstream toe of the reservoir. This erosion process removed rock-fill material beginning at the downstream toe and progressively moved upward along the slope. The removal of material is similar to that of the localized downstream toe stability and could have affected the global slope stability of the dike. Once a sufficient amount of material was removed, the downstream global slope stability was severely compromised to the point of failure. The capacity of the rock-fill dike to hold back the contained water, after global slope failure, would have been severely compromised. Complete failure due to erosion at the toe and reduced slope stability could have resulted. Figure 6 illustrates sketches of each stage of the proposed failure mechanism.

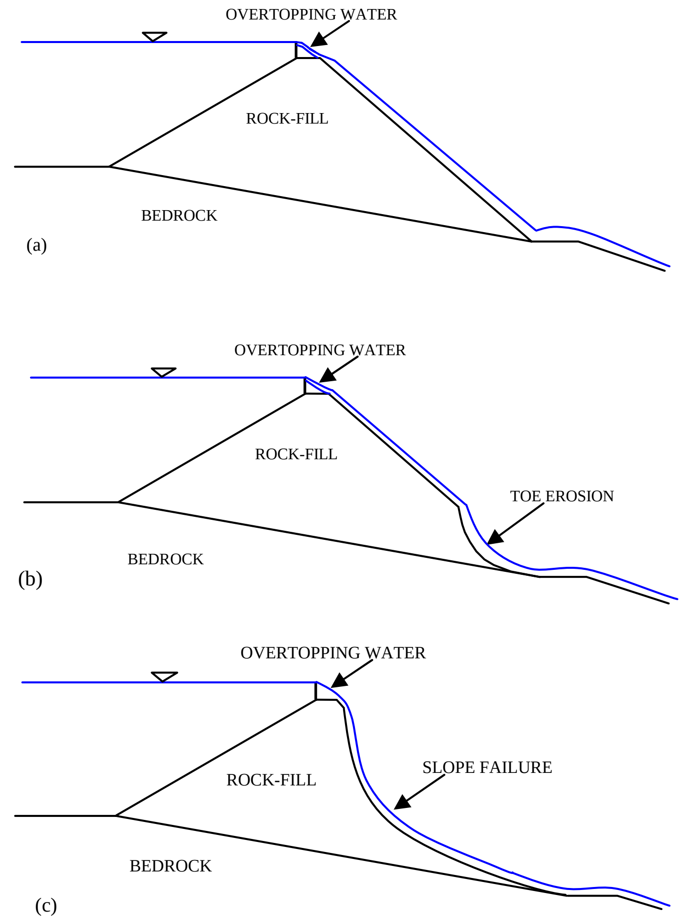

Fig. 6. Failure mechanism #2. (not to scale).
(a) water flowing across downstream slope, (b) progressive toe erosion, (c) resulting slope failure, (d) catastrophic failure with progressive erosion, same as Fig. 5(c).

#3 - Scour Undermining and parapet wall failure

The following summarizes failure mechanism #3: As overtopping of the reservoir was initiated, water spilling over the concrete parapet wall caused erosion and scouring at the downstream crest of the dike. This process began to undermine the concrete parapet wall foundation. As undermining and scouring progressed, the rock-fill material was transported down the slope. Once the undermining process removed a sufficient amount of rock-fill, the stability of the concrete parapet wall and the shear capacity of the rock-fill directly below the wall became severely compromised to the point of either wall failure and/or shear failure. After the aforementioned wall/crest failure(s), more water (10-feet) and consequently more flow was released in the area of the breach. The additional flow continued the erosion process down to the toe of the reservoir yielding the catastrophic failure. Figure 7 illustrates sketches of each stage of the proposed failure mechanism.

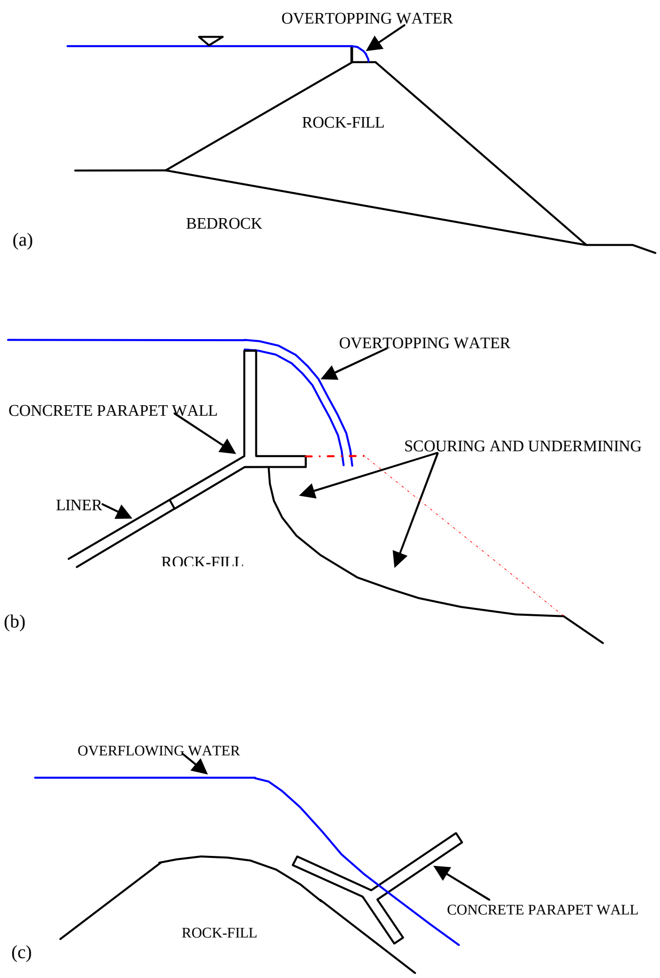

Fig. 7. Failure mechanism #3. (not to scale).
(a) overtopping water, (b) scouring and undermining, (c) parapet wall failure, (d) catastrophic failure with progressive erosion, same as Fig. 5(c)

DISCUSSION OF FAILURE HYPOTHESES RELATED TO ANALYSES

The analyses of each hypothesis made in this study were included in much detail in Gehring (2006). The following sections summarize the main points of the analyses and present related discussions.

Rising Phreatic Surface and stability failure

Assuming a free draining rock-fill, the possibility that a phreatic surface could rise to the extent of impending reservoir failure is unlikely. However, the presence of fines was noted in certain areas of the dike by Rizzo (2006). Given the noted fines content in the rock-fill, a phreatic surface rise was proposed as a failure mechanism. A rise in phreatic surface elevation within the dike would cause a decrease in effective stresses throughout the rock-fill mass underwater. The ability of the rock-fill material to dissipate excess pore pressures radially from the sections of overtopping makes it difficult to comprehend the rise in phreatic surface conditions needed to initiate global slope instability, localized toe slope instability, or both localized toe and global slope instability. However, the condition of a rising phreatic surface was analyzed in terms of localized toe and global slope stability.

The stability analyses show that for a condition with no excess pore pressures present within the dike, safety factors calculated using the Spencer method were 1.8 and 1.9 for localized toe and global slope stability, respectively. These factors are acceptable when compared to the minimum design recommendation safety factor of 1.5. When using the more conservative Bishop method the safety factor against global instability was close to 1.6 and the safety factor against localized toe slope instability was most nearly 1.5. Both factors are adequate when compared with design standards.

The phreatic surface was incrementally raised for the analyses. The safety factors began to drop, as expected, for both localized toe and global slope stability. A safety factor of nearly 1.5 was obtained for both slope stability conditions when the phreatic surface reached a theoretical elevation of roughly 50 feet above the upstream toe of the reservoir. The Spencer method was used for calculations. The calculated safety factors were still adequate when compared with design standards. The calculated safety factor for both slope stability conditions did not reach 1.0 until a theoretical rise in phreatic surface elevation was approximately 80 feet above the upstream toe of the reservoir. A safety factor of 1.0 corresponds to impending failure.

Moreover, before installation of the geosynthetic liner in 2004, information suggests that seepage from the Upper Reservoir became increasingly more costly as the life of the reservoir increased. It is noted that the reservoir remained stable before installation of the geosynthetic liner even with seepage pore pressures allowed to build up. This note suggests that the reservoir had experienced effects similar to a rising phreatic surface in the past. Although stability may have been in jeopardy during times of increased seepage, performance history suggests the reservoir could withstand some amount of excess pore water pressure build up.

Given the time frame of the failure events, it is unlikely that the phreatic surface elevation in the area of the breach reached an impending failure condition. From the aforementioned discussion and the performed analyses, a rising phreatic surface condition was not likely responsible for the rock-fill dike failure.

Downstream toe erosion and stability failure

Erosive and scour forces are difficult to quantify in terms of magnitude and extent. With erosion and scour, the contribution of flow characteristics causes changes in channel characteristics and changes in channel characteristics causes changes in flow characteristics. This reveals that the erosion and scouring processes are not independently controlled by flow characteristics nor independently controlled by channel characteristics. In other words, flow characteristics affect channel characteristics and channel characteristics affect flow characteristics. An attempt was made to identify the energy needed to initiate erosion at the downstream toe assuming that no changes in channel characteristics occurred on the downstream slope of the dike until the overtopping flow of water reached the downstream toe of the reservoir.

Through the analyses it was determined that a sufficient flow could have existed to mobilize granular material at the toe and initiate the erosion process. If this process occurred it would have severely weakened the rock-fill dike structure in terms of global slope stability. This effect would be similar to the effect that a localized toe failure would have on global slope stability. In areas where the overtopping flow was not sufficient to keep mobilized granular material suspended, deposition of the material along the downstream slope and along the downstream toe occurred. Evidence of an accumulation of granular material at the toe of the dike in non-failed, overtopped, areas suggests that a similar condition may have occurred in the breach segment before failure.

Analyses were conducted to quantify the possibility that overtopping water could initiate the erosion process along the downstream slope. The infinite slope analyses revealed that the downstream slope was marginally safe with respect to sloughing and planar failure and exhibited a safety factor of 1.1. Flow across the surface of the downstream slope or infiltration as water flowed down the slope could have reduced the safety factor and increased the effects of erosion, leading to either increased crest erosion, reduction of the embankment cross-section, and/or increased toe erosion.

The possibility that erosion could take place at the downstream toe of the Upper Reservoir existed given the condition that no material mobilization occurred at the downstream crest or along the downstream slope as water flowed downstream. The aforementioned discussion and post-failure evidence in other areas that experienced overtopping discount the likelihood of this process occurring. From the discussion and analyses, downstream toe erosion was likely not responsible for the rock-fill dike failure.

Scour undermining of parapet wall

The ability of the concrete parapet wall to withstand the applied loads (mainly water pressure) is paramount to the structural integrity of the entire Upper Reservoir rock-fill dike structure. If the parapet wall failed in any way, the sequential flow of water would have continued the erosion process from the top to the bottom of the rock-fill dike, resulting in catastrophic failure. Two mechanisms were proposed that could have lead to the instability of the parapet wall. They were overturning and sliding stability of the wall itself and shear failure of the rock-fill directly beneath the base of the wall.

The analyses revealed that the concrete parapet wall, before overtopping initiated and during the operational life of the structure, could effectively withstand the applied loads. The operational life safety factor against overturning was most nearly 2.4. The operational life safety factor against sliding along the base was most nearly 1.5. Both analyses did not consider the geosynthetic liner connections that could increase respective safety factors. When compared to typical design criteria the computed safety factors were satisfactory.

Analyses were performed to quantify overtopping water flow and scouring and undermining at the downstream crest. Figure 8 illustrates the concrete parapet wall profile, the probable elevation at which overtopping started, and the probable maximum water elevation during the failure events. The analyses revealed that overtopping flow could initiate scouring and undermining. The extent of scouring and undermining varied as a function of the parapet wall crest elevation and overtopping flow depth at various locations. Flow depth was defined as the height of water flowing over the parapet wall sections.

The segment between panel 70 and panel 74 experienced a greater effect from scouring and undermining than the location between panel 43 and panel 54. Due to the lower wall crest elevations between panel 70 and panel 74 and the resulting higher flow rates exiting the reservoir at this location. Also, the segment between panel 70 and panel 74 began overtopping before the segment between panel 43 and panel 54. The time factor associated with scouring and undermining caused less scouring and undermining between panel 43 and panel 54. The maximum differential settlement between the two locations was approximately 0.2-feet. If the parapet wall segments in the breach area experienced settlements similar to panel segments 70 through 74, the effects of scouring and undermining could have been similar.

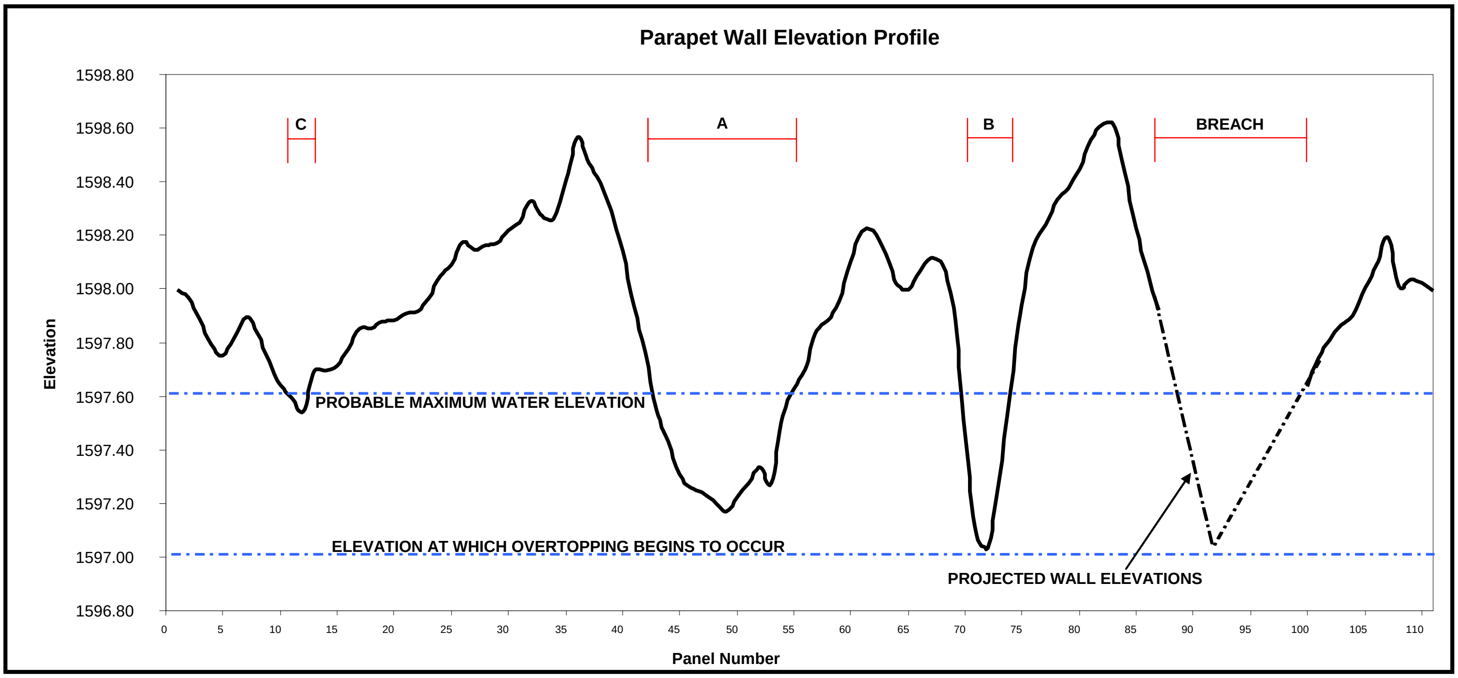

Fig. 8. Parapet wall elevation profile (source: MoDNR data).

Based on calculations presented in Gehring (2006), there was a relatively high potential for the rock-fill material at the crest to be scoured and eroded. To evaluate the effects of scour undermining on the stability of the parapet wall, an incremental approach to the extent of undermining was applied to sliding and overturning stability calculations. As expected, the resulting safety factors decreased with increased scour undermining. A safety factor of 1.0 was obtained for overturning stability when the scour undermining distance was roughly 3.3 feet, measured from the downstream toe of the parapet wall. Similarly, a safety factor of 1.0 was obtained for sliding stability when the scour undermining distance was roughly 2.0 feet, measured from the downstream toe of the parapet wall. A safety factor of 1.0 corresponds to impending failure. The analyses results supplement visual evidence of scouring and undermining within non-failed overtopped areas. If wall crest elevations in the breach area were similar to other overtopped segments, the effects of scouring and undermining could have been similar.

As undermining, scouring, and erosion progressed, the possibility existed that enough rock fill material was removed directly beneath the base of the wall to cause a localized soil shear failure. If localized soil shear failure occurred, the effects would have been similar to sliding or overturning instability of the wall. Both proposed circumstances could have caused wall failure, leading to the eventual complete discharge of stored water in the reservoir. The near 700-feet wide breach suggests, in the opinion of the writer, a sudden collapse of parapet wall segments and continued erosion and scouring down to bedrock.

Although it is difficult to assess which failure mechanism (i.e. overturning and sliding or localized shear) occurred first, the analyses support the concrete parapet wall instability and subsequent erosion of the rock-fill dike as a possible mode of catastrophic failure. From the discussion and analyses, the concrete parapet wall failure and resulting erosion of the rock-fill dike were likely mechanisms for the reservoir failure.

SUMMARY AND CONCLUSIONS

The events that lead to the failure of the Taum Sauk Upper Reservoir the morning of December 14, 2005 provided an opportunity to the academic and professional communities to learn from this interesting case study and provided a platform to bring dam safety into the regional and national spotlight. The desire to learn comes after realizing the potential for loss of life and the realistic loss of property value.

The scope of this academic exercise did not involve evaluating the causes leading up to the failure events nor does it attempt to address the causes pertaining to why the Upper Reservoir was allowed to overtop. The scope of this exercise was to systematically evaluate several postulated failure mechanisms and attempt to categorize each mechanism in terms of their relevance to the failure events.

Without actually witnessing the failure events as they occurred, only speculations can be made in terms of the true cause, or combination of causes, leading to the catastrophic failure of the Taum Sauk Upper Reservoir. In their own right, each proposed failure mechanism could have contributed solely to the failure events, or a combination of all three proposed mechanisms could be the culprit, or another failure mechanism not contemplated could be responsible. However, the phenomenon common to the three proposed failure mechanisms was overtopping water. Without water overtopping the Upper Reservoir, all three of the proposed failure mechanisms would be invalid.

Although each mechanism could lead to failure, the analyses and observations indicate that some failure mechanisms were more likely than others. The likelihood of occurrence, with 1 being the most likely and 3 being the least likely, is as follows.

1. Scouring and undermining of parapet wall.
2. Downstream toe erosion and stability failure.
3. Rising phreatic surface and stability failure.

The Taum Sauk Upper Reservoir, for all practical purposes, remained stable and operational throughout its 40 plus year service life. Circumstances such as increased seepage rates and surficial downstream slope instability were noticed in the past and may have decreased the overall stability of the Upper Reservoir in certain areas. The presence and migration of fine material may have caused greater susceptibility to erosion in certain areas of the dike. However, the presence and effects of fine material are not presented in this evaluation.

Acceptable construction practices during the late 1950's and early 1960's would not be deemed satisfactory by today's standards of practice. However, retrofitting all structures to accommodate the ever changing state-of-the-practice would be impractical. Given the stability of the Upper Reservoir over the 40 plus year operational life, even with seepage pressures being present before installation of the geosynthetic liner, inadequate construction practices were likely not the cause of the Taum Sauk Upper Reservoir Failure.

Spillways are typically specified to protect dams, dikes, and reservoirs from the potentially devastating effects of overflowing water. The Taum Sauk Upper Reservoir was designed without a spillway system. One possible reason for not specifying a spillway system for the Upper Reservoir was the lack of topographic drainage area contribution to the total inflow. The reservoir's only contributions to inflow were the intended pump-back procedures for filling and direct rainfall on the Upper Reservoir itself. This means that overtopping could be contrived from over-pumping/overfilling or excess rainfall with the later being highly improbable. Some form of spillway system could prevent this catastrophic failure event.

In conclusion, the true cause of the December 14, 2005 failure incident may never be ascertained but valuable insight and knowledge into the most likely failure mechanism(s) contributing to the event from research, study, and evaluation can be used to prevent future catastrophes of this magnitude. An understanding of failure events can be used as a tool to help protect the general safety and well being of society as a whole against future possibilities of dam, dike, and reservoir breaches. The Taum Sauk Upper Reservoir failure serves as a lesson for Missouri as well as the nation in regards to the importance of dam, dike, and reservoir safety.

Taum Sauk Today

The future of the Taum Sauk upper reservoir facility is now well defined. The owner, AmerenUE, was given authorization to rebuild at the same location. The current plans are to build a roller-compacted concrete (RCC) reservoir dam. The design will employ the latest in overflow structures, monitoring and control systems, seismic design criteria and other safety features. This new facility will be built using the existing materials from the existing embankment dam. The announcement to start the restoration project was made in late 2007 and as soon the testing, analysis and design are finalized construction will start. More information can be found at the owners website, click on: http://www.ameren.com/TaumSauk/

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