

# The 2005 Upper Taum Sauk Dam Failure: A Case History

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## ABSTRACT

The Taum Sauk Pumped Storage Powerplant in Reynold County, Missouri (USA) was constructed between 1960 and 1963 to store water for generation during peak daytime power demands. The plant consists of a lower reservoir, which is sited along the East Fork of the Black River, and an upper reservoir, which is formed by a kidney-shaped rock-fill dike approximately 50 to 87 ft (15.2 to 26.5 m) high, capped by a 10-ft (3.05 m) concrete parapet wall set on crest that is 12 ft (3.66 m) wide. The Upper Reservoir held 4,600 acre-ft (5.67 billion liters) when filled. A variety of design/construction flaws, an instrumentation programming error, and human errors contributed to a catastrophic failure of the Upper Reservoir on December 14, 2005. Malfunctioning and improperly programmed and/or placed sensors failed to indicate that the reservoir was full and did not shut down the facility's remaining pump unit until water had been overflowing for 6 to 7 minutes. This overflow undermined the parapet wall and scoured the underlying embankment, leading to a complete failure within that time frame. The peak discharge from this outbreak flood was estimated to be 289,000 cfs (8,184 m<sup>3</sup>/s), obliterating most of Johnson Shut-ins State Park, where, miraculously, only five people were injured. The flood pulse was significantly mollified by capture within Lower Taum Sauk Reservoir, and the maximum discharge over the Lower Taum Sauk Dam was limited to just 1,600 cfs (45.3 m<sup>3</sup>/s), precluding any significant downstream damage.

## INTRODUCTION

The Union Electric Company of St. Louis began searching for a suitable site to implement a pumped

storage scheme in 1953. In 1958, Union Electric began exploring the possibility of employing a smaller storage reservoir with more than 300 vertical feet (91 m) of pressure head. They zeroed in on the St. Francois Mountains south of St. Louis, where vertical differentials approaching 1,000 ft existed between mountain peaks and valley bottoms (Gamble, 1960a). Union Electric retained Sverdrup-Parcel & Associates to conduct feasibility studies of possible sites in the St. Francois Mountains, about 90 mi (145 km) southwest of St. Louis (Figure 1), where the Ozark Uplift has elevated resistant Precambrian strata, and valleys are 200 to 1,000 ft (61 to 305 m) deep. They also retained J. Barry Cooke (1915–2005), who was employed by Pacific Gas & Electric Co. in San Francisco and was recognized as one of the preeminent figures in the use of concrete-faced rock-fill dams (CRFD).

Sverdrup initially targeted Taum Sauk Mountain, the highest point in Missouri (1,772 ft/540 m), but this proved politically unacceptable because of its perceived scenic and recreational value (the area was subsequently incorporated into Taum Sauk Mountain State Park in 1991). A resistant ridge of Precambrian rhyolite extends some 6 mi (9.65 km) southwest from Taum Sauk Mountain and terminates at the southern extremity of Proffit Mountain, Missouri's 6th highest peak with an elevation of 1,720 ft (524.2 m). Here, the East Fork of the Black River has excavated a steeply incised gorge where the river passes through Johnson Shut-ins, overlooking the junction of Taum Sauk Creek, Little Taum Sauk Creek, and the East Fork of the Black River (Figure 2). An offstream storage reservoir on this southernmost promontory of a long linear ridgeline made sense because of the proximity of the Black River as a source of water and the relatively short distance (27 mi/43.4 km) to the existing power transmission grid.

The project retained the name Taum Sauk Pumped Storage Hydroelectric Project even though it was shifted to the southern end of Proffit Mountain. The

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## GEOLOGIC SETTING

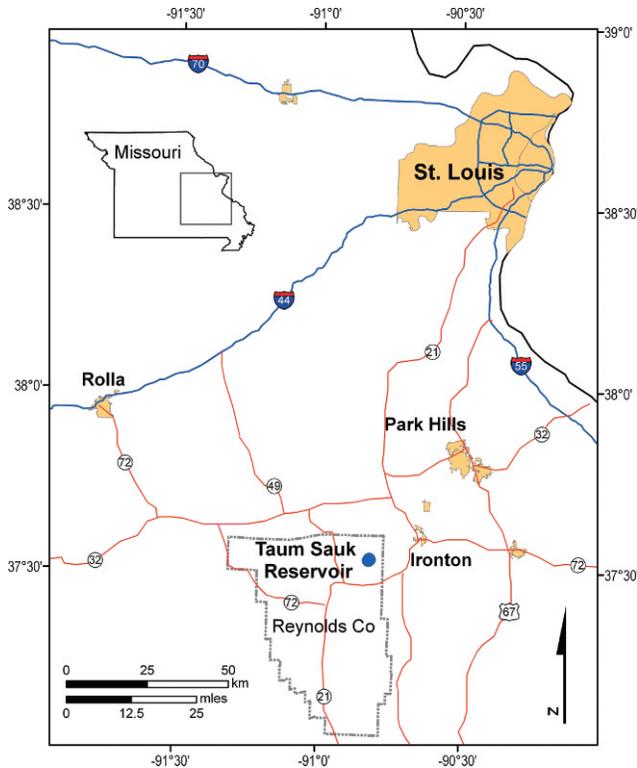


Figure 1. Location of the Taum Sauk Pumped Storage Project in rural Reynolds County, Missouri, about 90 mi (145 km) south of St. Louis.

severe topographic relief provided the required head for the efficient operation of a pumped storage power plant, and the Precambrian rocks were assumed to provide a stable foundation for the construction of the highest head hydroelectric plant east of the Rocky Mountains, with a maximum head loss of 875 ft (266.7 m). A lower storage reservoir was constructed on the East Fork of the Black River, which provided sufficient storage to enable daily transfer of ~4,600 acre-ft (5.67 billion liters) of water into the Upper Reservoir. The project was located just one mile downstream of a designated Missouri Geologic Natural Area called Johnson Shut-ins State Park, about 10 mi (16 km) from the small town of Lesterville, in Reynolds County. The objective of this paper is to briefly summarize this precedent-setting project, describe the site geology and the impacts of unforeseen foundation conditions on the operation of the facility, and show how a string of human errors led to erroneous assumptions by decision makers, and the likely failure scenario. The article also seeks to explain why the catastrophic breach occurred on the northwestern side of the dike when a greater volume of water actually passed over the southeastern side, into the Lower Reservoir.

The St. Francois Mountain region is an exhumed Precambrian terrane that is part of a volcanic belt extending from southern Ohio to the Texas Panhandle (Berry, 1976). During Precambrian time (>1,500 Ma), this region was intruded by upwelling granite, forming structural highlands that were subsequently intruded by volcanism (Unklesbay and Vineyard, 1992). These volcanoes erupted large quantities of ashy pyroclastic flows and rhyolitic lava. Thick layers of pyroclastic materials were deposited throughout the region as air-fall and ash-flow tuff. The Taum Sauk Rhyolite was one of several units named by Berry (1970, 1976). It 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 (Figure 3). Anderson and Scharon (1961) thought it reached a thickness of ~2,500 ft (762 m), with individual flows as thick as 500 ft (152.4 m). Berry (1976) subdivided the Precambrian volcanic rocks in the western St. Francois Mountains into 14 mappable units, based on their spatial distributions and structural and stratigraphic relationships, with an aggregate thickness of over ~20,000 ft (6 km) and a volume in excess of 720 cubic miles (3,000 km<sup>3</sup>). The Taum Sauk Rhyolite was the thickest unit, exceeding 3,280 ft (1,000 m). Detailed mapping of these volcanoclastic units exposed at Johnson Shut-ins State Park has been summarized by Blades and Bickford (1976), Zeller (1980), and Hebrank and Kisvarsanyi (1987).

The upper 1,000 ft (305 m) of the Taum Sauk Rhyolite is exposed on the south end of Proffit Mountain. In this area, residual heat from the eruptions appears to have melted or “welded” the pyroclastic ash fragments together, which, upon cooling, formed a steel-hard (Moh’s hardness of 7.4) igneous rock referred to in the literature as welded tuff or ignimbrite (the “Rhyolite-Porphry” of Dake, 1930). Anderson (1962) envisioned a recurring cycle of extrusion that expelled large volumes of volatile-rich materials, capped by more effusive extrusions that produced layered ash-flow tuffs. The thicker flows can be traced over distances of up to 11 mi (17.7 km). Most of the ash-flow tuff present in the Proffit Mountain area has a light pinkish to violet hue, commensurate with its felsic composition. The depositional relationships were best revealed during the original construction in 1960–63, and these suggest that the Precambrian rhyolite flows were extruded onto exposed surfaces of the granite porphyry, which are the oldest rocks exposed in the region.

## 2005 Upper Taum Sauk Dam Failure

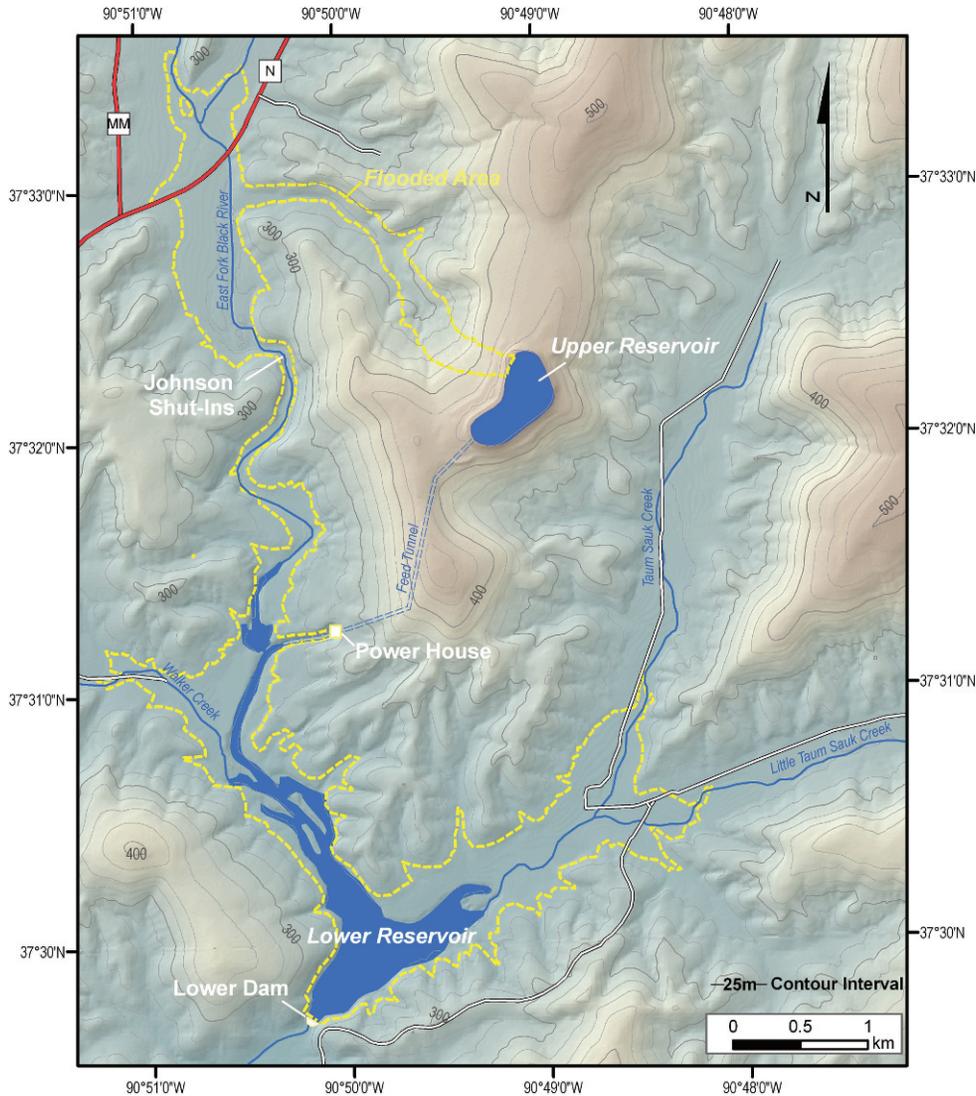


Figure 2. Physical layout of the Taum Sauk Pumped Storage scheme, constructed on Proffit Mountain, using water from the East Fork of the Black River. Note feed tunnel between the Upper Reservoir and powerhouse and proximity to Johnson Shut-ins. Dashed yellow line delineates the areal limits of flooding when the reservoir breach occurred on December 14, 2005.

Sometime after the rhyolite flows were extruded, the region was permeated by narrow vein-like intrusions of diabase, which cut across the granite and rhyolite. These dikes exhibit noticeable alteration in the baked zones of the surrounding rock, and they are often altered to clay, creating linear seams, usually between 1 and 8 in. (2.5 to 20 cm) thick. Most of the altered diabase clay seams exposed on Proffit Mountain are gently dipping, between 10 and 30 degrees, parallel and opposing to the native slopes (Rizzo, 2009).

After volcanic activity ceased Precambrian time, the area was uplifted by the Ozark Dome (Unklesbay and Vineyard, 1992). This uplift exposed the more resistant igneous knobs and ridges that typify the St. Francois Mountains today. This was the first of four

distinct periods of subareal weathering, accumulation of colluvium, and development of residuum that have occurred in this area (the other periods of significant subareal weathering include the Cambrian, Ordovician, and late Quaternary). Thick masses of fanglomerate accumulated on the Precambrian granite during the Precambrian, and to lesser extent, on the more massive, but laterally restricted limbs of rhyolite. These heterogeneous assemblages represent Precambrian erosion surfaces, which are locally preserved between the older igneous rocks and the sediments that began to be laid against the bedrock promontories during the Cambrian. These discontinuous “caps” of blocky material are sometimes referred to as Precambrian conglomerate or fanglomerate (Hayes et al., 1961), which are seldom noted unless exposed

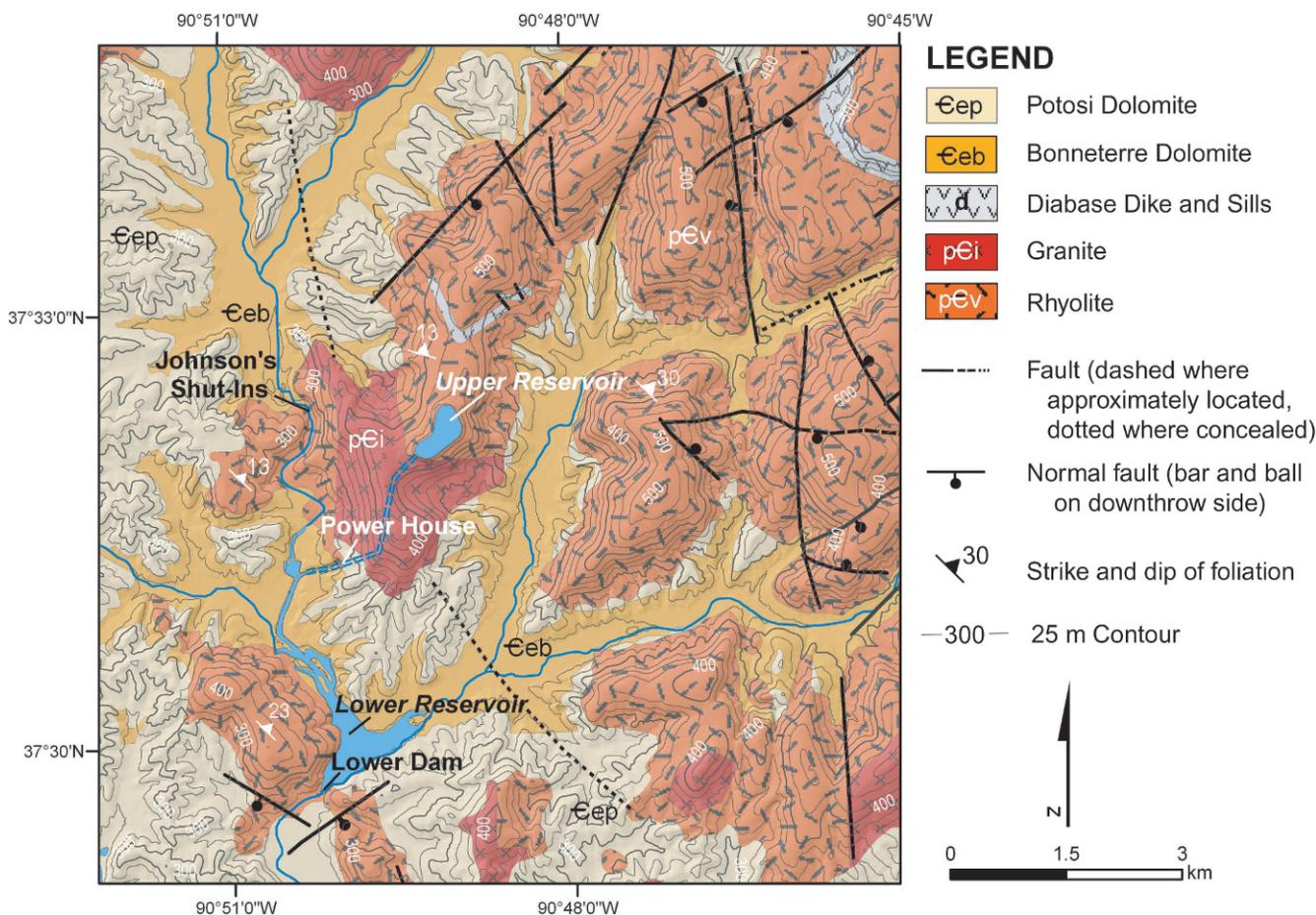


Figure 3. Geologic map of the Proffit Mountain–Johnson Shut-ins area, showing the major stratigraphic units that underlie the ridges and mountaintops in the western St. Francois Mountains of Missouri (modified from MoDGLS, 2007).

by excavation (such as road cuts). They are easily recognized by the severe weathering of their entrained blocks and the unconformable relationship with younger sedimentary strata, such as the Lamotte Sandstone or Bonneterre Dolomite.

When the Cambrian seas began to rise, much of 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 conglomerate, arkosic sandstones, and dolomites on the seafloor and draped layers of the same material on the flanks of the resistant ridges and highpoints, often with exaggerated dips (~25 degrees), assumed to have been caused by post-depositional consolidation of the sediment (Howe, 1968). Proffit Mountain is located along one of those resistant ridges that was not covered by the Lamotte Sandstone, the basal Cambrian unit. The Bonneterre Dolomite is exposed on the lower flanks of the mountain where the outbreak flood occurred in December 2005.

Regression of the Cambrian seas exposed the younger sedimentary deposits and the igneous high-

points. Erosion of the Cambrian strata produced new drainage patterns, which were influenced by the geometry of the resistant rhyolite flows. 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 softer Paleozoic units (of Cambrian and Ordovician age). Where the largest modern water-courses cross these resistant ridges of rhyolite steep bedrock, narrows form locally referred to as “Shut-ins.” Shut-ins are typified by an extremely rough and uneven texture influenced by secondary joints, and they usually contain numerous joint-influenced scour potholes (Beveridge, 1978). Johnson Shut-ins is considered to be one of the finest examples of these features.

As with the most of the Ozark Plateau, the St. Francois Mountains were not glaciated during the Pleistocene Epoch. This preserved many ancient, deeply weathered zones of bedrock and saprolite present throughout the region. These saprolite zones are most pronounced adjacent to the diabase dikes and along the contacts between the lithic ignimbrites.

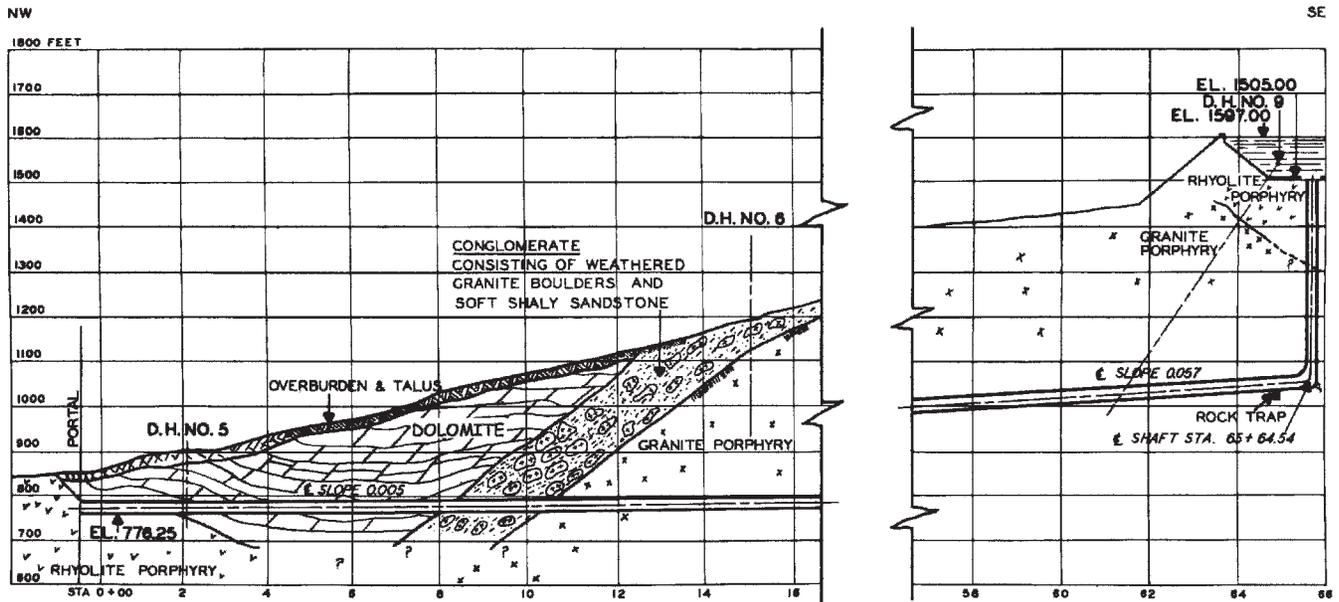


Figure 4. Partial geologic profiles along the lower and upper portions of the Taum Sauk feed tunnel, excavated in 1960–62. The project originally envisioned an inclined shaft, which was explored using a test boring inclined at 55 degrees, shown at right. The unit overlying the granite porphyry is a Precambrian fanglomerate (taken from O'Brien, 1961).

In several areas, dike swarms, or locally thick zones of intruded diabase exhibit severe weathering, and have degraded to a soil-like appearance and consistency. The unpredictable presence of these friable materials has allowed the rock-fill embankment to become contaminated with fines, which has led to excessive consolidation and differential settlement (Cooke, 1967; FERC, 2006; and Rizzo & Associates, 2006). Several of these saprolite zones required extensive over-excavation during the reconstruction of the new Upper Reservoir dike in 2007–09.

#### Consultants and Site Exploration

The pumped storage site was chosen by planning engineers at Union Electric, who then hired Sverdrup-Parcel consulting engineers of St. Louis to prepare the plans and specifications for the project, suitable to allow an accurate estimate of the project cost. In 1959, Sverdrup retained Frank A. Nickel as a consultant on the project (Gamble, 1960a; O'Brien, 1961). Nickel was a respected figure in dam geology, having served as the head geologist of the U.S. Bureau of Reclamation (Nickel, 1942). After the project was authorized, Union Electric also retained I. C. Steele as a consultant to review the details of the proposed scheme and make useful suggestions, because the contractors were suggesting a number of changes to reduce costs.

Nickel recommended a minimal program of sub-surface exploration because the rhyolite was so impregnable it required 30 NX diamond core bits to

recover just 50 ft of core! Nickel's recommended program of exploration employed 14 core holes, with depths of 50 to 700 ft, drilled during the fall and winter of 1959–60 (O'Brien, 1961). These borings revealed that 30 ft of residuum and colluvium overlaid dolomite along the planned feed tunnel, and that granite porphyry lay under the massive rhyolite at the south end of the Upper Reservoir site, and that the tunnel would penetrate the granite within 200 ft (61 m) of the reservoir floor (Figure 4). Hayes (1965) reported that upward of 200 ft (61 m) of Cambrian and Ordovician residuum blanketed the slopes in the immediate area, covering most exposures.

Nickel's exploration only drilled two exploratory holes in the Upper Reservoir area (at the north and south ends) because he assumed very little cover blanketed the resistant rhyolite. These holes encountered 10 ft (3.05 m) of residual cover and only extended 50 ft (15.2 m), to the planned elevation of the reservoir floor (around elevations 1,505 ft/458.7 m). Pump tests were performed in 10-ft (3.05 m) intervals using downhole packers with injection pressures of 200 psi (1,379 kPa) for 10 to 20 minutes to evaluate the hydraulic conductivity of the rhyolite and ascertain if the reservoir floor needed to be lined to limit infiltration losses. The water loss in the proposed reservoir floor varied between 113 and 350 gpm (0.43 to 1.32 m<sup>3</sup>/min) under 200 psi (1,379 kPa) for 20 minutes, necessitating paving of the rhyolite floor to curtail seepage loss. The reservoir floor was paved with 4 in. (10.2 cm) of asphaltic

concrete to provide a seal. Numerous uplift relief wells had to be installed soon after the initial filling to alleviate slab buckling and uplift induced by rapid drawdown.

### CONSTRUCTION AND EARLY HISTORY

From its inception, Taum Sauk was intended to be the first power plant in the United States designed as a completely automated system, controlled remotely via microwave transmissions from Union Electric's Osage Hydroelectric Plant at Bagnell Dam, about 120 mi (193 km) away. It was also America's first pure pumped storage scheme, which relied solely on pumped water to run the turbines (Warnock, 1975). The Lower Reservoir has a drainage area of approximately 80 square miles (207 km<sup>2</sup>), but, after initial filling, the natural flows of these streams were passed through the lower dam via a 20-in.-diameter (0.508 m) sluiceway. The lower dam was also configured with an 8 × 10 ft (2.4 × 3.05 m) sluice gate to allow removal of coarse sediment from the Lower Reservoir (Rudolph, 1963). The pumped hydroelectric scheme had a distinct advantage in being able to be turned on and off for relatively short periods of time, augmenting the coal-fired thermal plants during daily periods of peak demands over the Missouri-Iowa-Illinois service area (Gamble and Rudolph, 1964).

Union Electric chose the Proffit Mountain site as their best candidate for a pumped storage plant after performing cost-benefit analyses comparing pumped storage and thermal generation schemes (Whitlow, 1961; Vencill, 1961). However, the final decision to proceed depended on actual costs. In December 1959, a contract was awarded to a joint venture of Utah Construction Company of San Francisco and Fruin-Colnon Contracting Company of St. Louis (Gamble, 1960b; Rudolph, 1963). Fruin-Colnon installed the generating and transmission equipment, while Nooter Corporation was brought in to install the high strength (T-1) steel penstock and tunnel lining. Utah Construction began mass grading for the Upper Reservoir area in early June 1960, and the various project components, including transmission lines, were constructed over the following three years (1960–63), at a cost of \$50 million over 36 months (Rudolph, 1963). The project cost an additional \$2.5 million because of unanticipated problems with seepage and mechanical problems with the hydroelectric plant during the last half of 1963, as they attempted to bring the facility online (Gamble and Rudolph, 1964).

As soon as crews began excavating the crest of Proffit Mountain unforeseen conditions manifested

themselves. It was expected that not more than 10 ft (3.05 m) of unconsolidated residual cover would be required across the Upper Reservoir area because that was all that was encountered in the two exploratory borings. However, work crews soon began encountering deep and irregular pockets of residuum, as well as numerous diabase saprolite veins between the rhyolite outcrops. Sverdrup brought Dr. Nickel back to the site, and he quickly designed a secondary drilling program consisting of ten additional NX core holes carried 50 ft into the rhyolite, along with 14 dozer test pits across the 50-acre (202 × 10<sup>3</sup> m<sup>2</sup>) reservoir floor area (from which the excavated muck was to be used as rock fill for the circular dike). Using these data, Nickel supervised the construction of a depth-to-bedrock isopleth map that showed the average depth of cover was actually slightly more than 20 ft (6.1 m), about twice what had been assumed after the preliminary drilling program (O'Brien, 1961).

Several months later, the contractor also discovered a "deeply weathered zone" (diabase saprolite of "unlimited depth") where there had been "mass kaolinization of the rhyolite porphyry" on the west side of the Upper Reservoir (O'Brien, 1961). It was decided to realign the footprint of the Upper Reservoir dike to avoid as much of the deeply weathered area as possible, by employing reverse curvature on the west side, creating the dam's distinctive "kidney shape" of the circular dike (when the replacement roller dam was constructed in 2007–09, this same saprolite zone was encountered again, and it was over-excavated to an average depth of 40 ft [12.2 m] and backfilled with roller compacted concrete [RCC] dike).

The Lower Reservoir was formed by constructing a 60-ft-high (18.3 m) concrete gravity dam, 360 ft (110 m) long across the East Fork of the Black River about 3 mi (4.8 km) upstream of Lesterville, MO. The spillway sill of the lower dam is at an elevation of 750 ft (228.6 m), and it was designed to pass a maximum flood flow of 70,000 cfs (1,982 m<sup>3</sup>/s), with 12 ft (3.66 m) of water passing over an Ogee crest. The Upper Reservoir had a storage capacity of 4,600 acre-ft (5.67 million m<sup>3</sup>) over an area of 370 acres (914 ha), with a maximum head differential of approximately 861 ft (262.1 m) between the operating pools in the upper (1,597 ft) and Lower Reservoirs (designed for operation between 735 and 750 ft [224.02 to 228.59 m], but usually maintained at 736.25 ft [224.40 m]) (Gamble, 1960b; Whitlow, 1961). No more than about 70 percent of the available volume of the Upper Reservoir pool was routinely drained during its daily operational cycle, dropping the upper pool to

elevation 1,530 ft (466.3 m) (Vencill, 1961). More than half of the water in the Lower Reservoir would be pumped into the Upper Reservoir, dropping the Lower Reservoir pool about 15 ft (4.57 m).

The combined length of the shaft, tunnel, and penstock, about 7,100 ft (2,164 m), had to be excavated between the Upper Reservoir and powerhouse. The tunnel's diameter was variable, depending on location. A standard throated inlet for the headrace tunnel was originally laid out on a 55-degree slope (and a core hole of this inclination was drilled during site exploration in 1960). When the project went out for bid in December 1959, several contractors noted that a vertical shaft could be excavated at a substantial savings using the raised-bore method, resulting in less rock pull-out, so this alternative was selected (O'Brien, 1961). The unlined vertical shaft dropped 451 ft (137.5 m) from the Upper Reservoir floor and was 27.2 ft (8.3 m) in diameter. The contractor employed an 8-ft (2.44 m) pilot shaft, raised from the bottom of the shaft to the floor of the Upper Reservoir, and then enlarged it downward.

All muck was removed via the powerhouse portal and dumped on natural bench along the southern bank of the East Fork of the Black River. The middle unlined portion (4,764 ft/1,452 m long) downstream of the elbow was a 25.5 ft × 25.5 ft (7.77 m × 7.77 m) horseshoe-shaped tunnel (sloping 8 percent), while the last 1,807 ft (550.75 m) was excavated at a 5.7 percent grade and steel lined, with a diameter of 18.5 ft (5.64 m). The quasi-glory hole entry was located about 100 ft (30.5 m) from the inboard toe of the dike's southern end. The reservoir floor was fitted with a vortex-suppressing basin about 20 ft deeper than the rest of the Upper Reservoir in this area (Rudolph, 1963). At a depth of 195–200 ft (59.4 to 61 m), the vertical shaft passed through a weathered diabase dike, and thence into the granite porphyry, where it passed through a 94.6-degree elbow and then into a straight section (Figure 4). Two rock traps were excavated below the tunnel invert in this section to catch any loose debris that might spill into the shaft, to prevent debris from reaching the turbines. The headrace tunnel continued through the granite for another 5,425 ft (1,653.5 m) until piercing the Precambrian fanglomerate, shown in Figure 4. The last 1,807 ft (550.75 m) of the feed tunnel was lined with high-strength (T-1) steel where it had insufficient cover to meet the design specification. The rock overburden had to weigh more than 50 percent of the maximum static head, plus 25 ft at any given point along the feed tunnel. In the lined section, the headrace tunnel traversed granite saprolite, Precambrian fanglomerate, covered by beds of shaly and

arkosic dolomite and carbonate muds that Howe (1968) assigned to the upper Davis Formation and the Derby-Doerun Dolomite. The tunnel passed back into the Taum Sauk Rhyolite near its portal (Figure 4). The operating velocities within the tunnel varied between 15 fps (4.57 m/s) in the unlined sections to 31 fps (9.45 m/s) in the lined section.

Excavations for the rock-fill dike encountered rhyolite porphyry and lesser amounts of granite saprolite, altered diabase dikes, and crushed and sheared materials, including clay seams, between these Precambrian units. The kidney-shaped embankment required 3.5 million cubic yards (2.675 million m<sup>3</sup>) of rock fill, placed by hydraulic sluicing and cursory roller compaction of the upper 16 ft (4.9 m) portion. The crest of the circular dike was 6,562 ft (2,027.3 m) long, with a maximum height of 84 ft (25.6 m) above the reservoir floor. A typical cross section of the dike is shown in Figure 5. After placement of the fill materials, preparations began for lining the inboard side of the rock fill with a 10-in.-thick (25.4 cm) shotcrete facing, which averaged about 18 in. (45.7 cm) thick because of the uneven face of the dike. These slabs were divided into 111 vertical panels to accommodate contraction, expansion, and settlement (Figure 6), using conventional expansion joints, employing copper bellows strips covered by a bituminous-covered plank as the spacer between the panels. The slabs were reinforced with No. 7 bars at 12 in. (30.5 cm) in both directions.

The crest of the dike was capped by a 10-ft-high (3.05 m) parapet wall, intended to be filled to a depth of 8 ft (2.5 m) during each fill cycle, to maintain 2 ft (0.6 m) of freeboard (Vencill, 1961). This parapet wall increased the reservoir capacity about 11 percent, to a maximum volume of 4,600 acre-feet (5.67 billion liters). Much of the dike footprint was founded on substantive cuts (shown in brown on Figure 6), ranging from zero to as much as 40 ft (12.2 m) high at panel 37. The dike was floored in rock cut between panels 14 and 45, 49 and 70, 72 and 77, and 82 and 86. These rock cuts were covered with shotcrete and welded wire mesh to retard infiltration and provide a more even structural transition with the concrete panels formed on the inside face of the dike (Gamble and Rudolph, 1964). A great many of the joints leaked badly during the first year of operation, necessitating repairs and placement of additional drainage measures.

The majority of the Upper Reservoir's rock-fill embankment appears to have been constructed through simple end-dumping and hydraulic sluicing of the excavated material. The fill was allowed to tumble down the side of the embankment, lying near its natural angle of repose, from whence it was

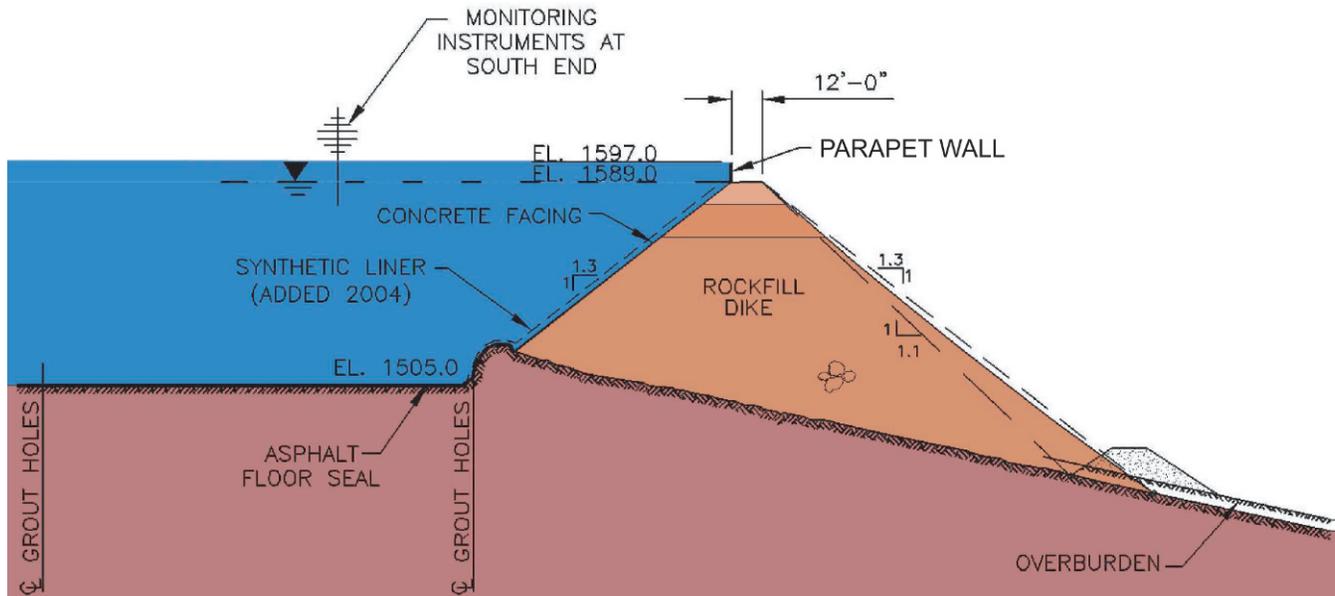


Figure 5. Typical section through the Upper Reservoir dike, illustrating how much of the dike was founded on a descending bedrock surface. Side slopes were to have been 1.3:1 (horizontal to vertical). Subsequent measurements show that the inboard side of the dike was most often inclined at 1.4:1 (Luna et al., 2007). The random rock and soil fill was end-dumped and sluiced, using a hydraulic monitor. Only the uppermost 16 ft (4.9 m) of the dike was placed using roller compaction.

sprayed with a jet of water to help consolidate the mass and fill in voids. The embankment materials were not mechanically compacted until the upper 16 ft (4.9 m) of fill, which was compacted in four separate 4-ft-thick (1.2 m) lifts. According to the report by the Federal Energy Regulatory Commission (FERC, 2006), this was the “last uncompacted concrete-faced rock-fill 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 Figure 7).

A 350-MW capacity powerhouse was constructed at the southern base of Proffit Mountain. It was connected to the Lower Reservoir through a 1,600-ft-long (488 m) by 65-ft-wide (19.8 m) channel excavated in the rhyolite to a maximum depth of 140 ft (42.7 m) below grade, with a bed elevation of 724 ft (220.7 m) above sea level, about 20 ft below the low-water level of the reservoir (the channel bed was 50 ft wide). A somewhat larger conveyance channel over a mile long had to be excavated within the East Fork of the Black River to convey the 7,000 cfs (200 m<sup>3</sup>/s) in a channel that normally passed a mean annual flow of just 100 cfs (2.83 m<sup>3</sup>/s). This channel was excavated to a base width of 100 ft (30.5 m) with 3:1 (horizontal to vertical) side slopes. Two late winter–early spring floods brought unanticipated quantities of sediment into the Lower Reservoir through Johnson Shut-ins, forcing Union Electric to remove this debris from the Lower Reservoir using a clamshell and suction dredge

(Gamble and Rudolph, 1964). In the wake of this shut-down, they issued an additional contract to have a protective dike constructed across the Black River just upstream of the confluence between the powerhouse tailrace and conveyance channels, to catch whatever sediment was carried down the channel during high-flow events. This was an essential element to protect the conveyance channel from becoming clogged during pump cycles, which dropped the Lower Reservoir level by 15+ ft. The trapped sediment was periodically excavated from behind this dike, and a 15-acre muck pile was created above the right bank of the river.

The powerhouse was originally equipped with two 175-MW reversible pump/turbine units of which one or both operated in pumping or generation, depending on the power demand and available water in the reservoir. At the time, these were the largest pumps and most powerful turbines ever produced, built by Allis Chalmers, while the generators were fabricated by General Electric (Rudolph, 1963; Warnock, 1975). Allis-Chalmers developed reversible pump turbine runners specifically for pumped storage projects. It was necessary to situate the units so the pump impeller (turbine runners) would be 32 ft (9.75 m) below the minimum lower pool level, to avoid cavitation (Vencill, 1961; Rudolph, 1963). This drop was accomplished through a J-shaped penstock 248 ft (75.6 m) long between the tunnel and the turbines. The invert of the turbine draft tube was at elevation 675 ft (205.7 m), about 60 ft (18.3 m) below the design

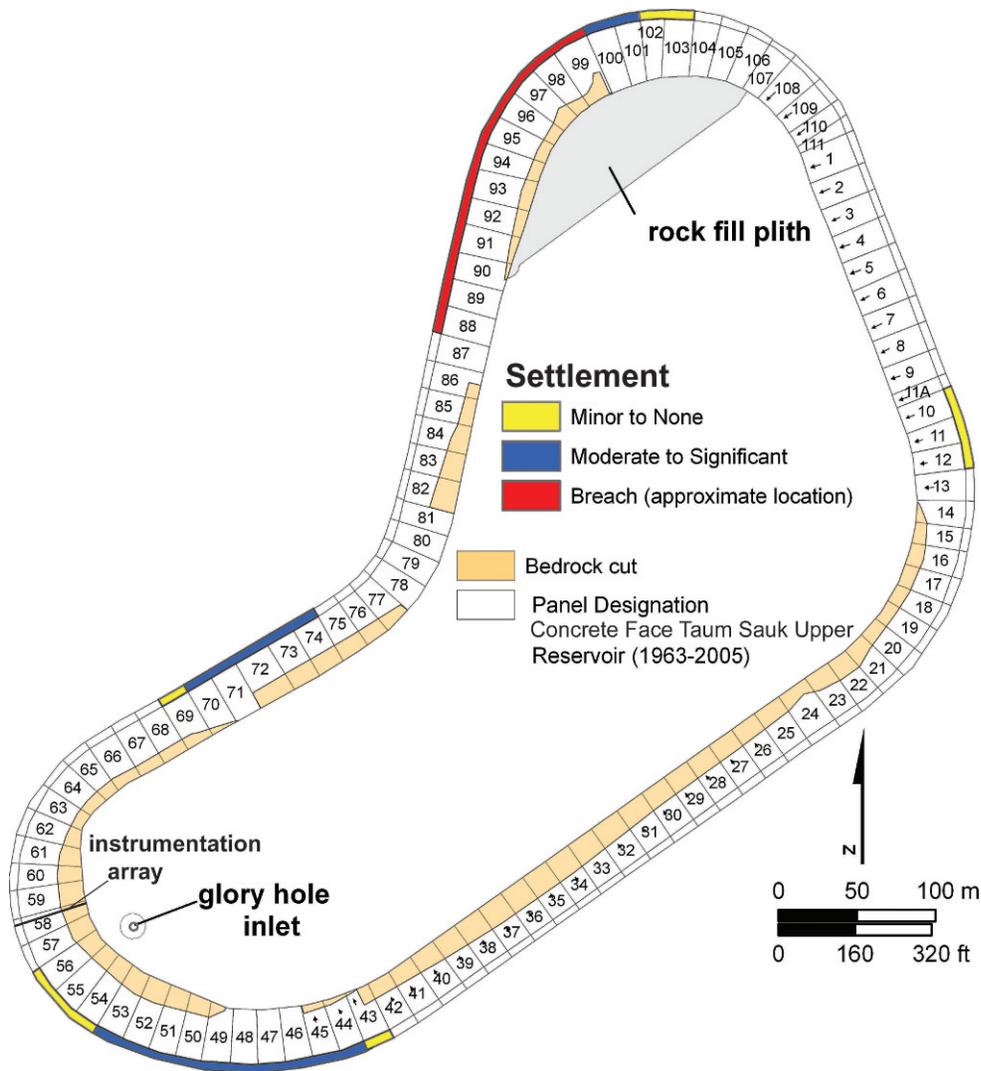


Figure 6. Plan view of Taum Sauk Upper Reservoir, showing all 111 concrete panels lining the inside face of the circular dike, which was capped by a 10-ft-high (3 m) cantilever retaining wall. The colored areas denote those portions of the wall that were overtopped on the morning of December 14, 2005. Note the variable height of cut on the inboard toe of the dike and height of embankment on the outboard side. Leakage and settlement problems manifested themselves between panels 89 and 107 in 1963–64, in the vicinity referred to as the “plinth” by FERC’s Taum Sauk Investigation Team (2006).

low water surface of the Lower Reservoir (Gamble, 1960b). In 1999, both units were upgraded to 225-MW capacity, bringing the plant to an aggregate generating capacity of 450 MW. The new pump turbines could lift as much as 5,238 cfs (148 m<sup>3</sup>/s) into the Upper Reservoir under full head of 767 to 861 ft (233.8 to 262.4 m). The plant generated 63.5 units of electricity for every 100 units expended in the pumping cycle (Whitlow, 1961). The Upper Reservoir could be completely drained in 24 hours and refilled in 16 hours (Rudolph, 1963).

The lower dam was closed on February 19, 1963, reaching the spillway crest 27 days later, on March 18th. Union Electric began filling Upper Reservoir on July 3rd. When they began testing the turbine

generators in late August, they encountered excessive high-frequency vibrations in the draft tube pits (Gamble and Rudolph, 1964). These problems were solved by a team of experts from Allis Chalmers and General Electric, and the plant was dedicated on October 9th, although uplift problems in the turbines at high rpm values (velocities reached 150 mph/241 km/h) were detected that required further study and mitigation. Because of these mechanical problems and recurring seepage problems in vicinity of panels 90 through 107 in the Upper Reservoir, the project did not begin generating electricity until December 20, 1963, and electricity generation was not consistent until after June 1964. Power generation and pumping operations were controlled remotely via microwave

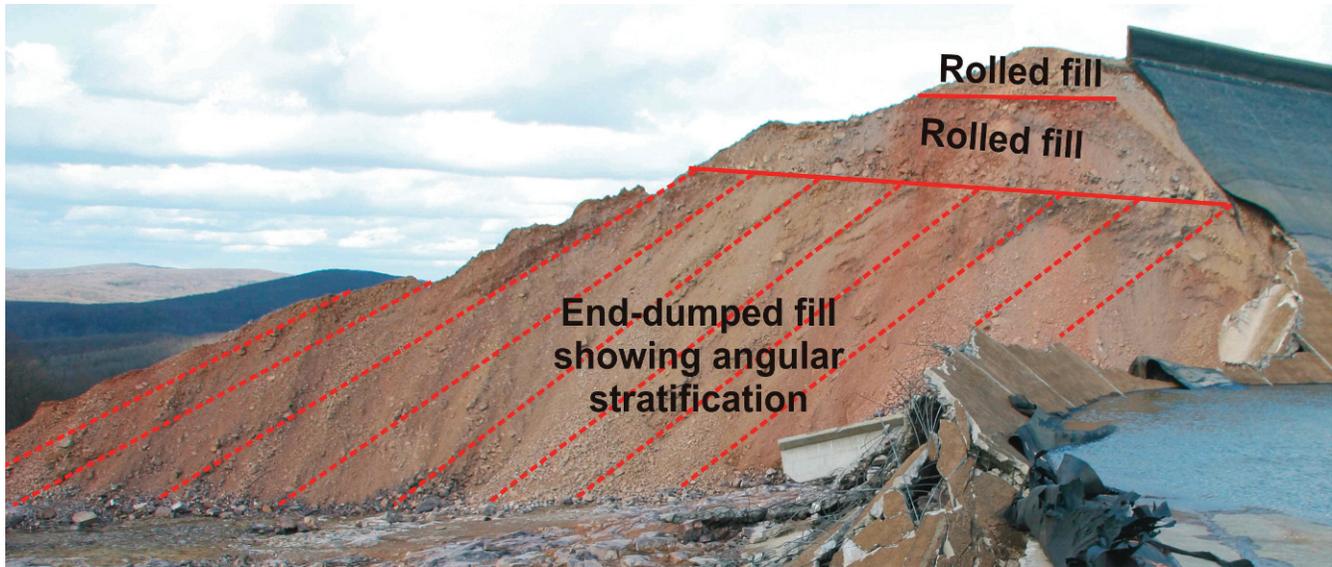


Figure 7. North side of the breached embankment reveals that most of the fill was end-dumped while two sequences of rolled fill capped the embankment. There does not appear to have been any significant effort made to mechanically compact the embankment because everyone assumed it would be a relatively clean rock fill.

transmissions from Union Electric's Osage Hydroelectric Plant at Bagnell Dam, over 120 mi distant.

#### BATTLING SEEPAGE AND SETTLEMENT

In September 1963, Barry Cooke was again summoned by Union Electric to provide recommendations on how they might reduce the leakage in the vicinity of panels 90–107, which reached a maximum flow of 103 cfs (2.92 m<sup>3</sup>/s). Union Electric battled recurring seepage problems in this area between September 1963 and April 1964, which required draining the reservoir for extensive repairs on five separate occasions. These repairs included emplacement of seepage cutoff walls, grout curtains, and laying down new reinforced concrete slabs over those that had been placed just a few months earlier, which had become undermined by piping of fines beneath the northwestern corner of the reservoir. These leaks appeared to be centered on the reservoir floor adjacent to panels 90–95, where the rock dike was founded on a relatively thin veneer of rock fill over a layer of residual soil developed upon the rhyolite. This was the “plinth,” or platform base upon which the embankment rested in the northwest corner, referred to in the FERC Independent Panel (2006) forensic report.

The Upper Reservoir rock-fill dike also experienced unexpectedly high rates of settlement during its first few years of operation. During the first four and a half years of operation, between 0.5 and 0.8 ft (0.15 and 0.24 m) of settlement was measured, which

correlates to 0.60 percent to 0.95 percent of the total embankment height, respectively. Problems with predicting the consolidation of hydraulic fill dams had long been recognized and appreciated, but these concerns were limited to the impervious clay cores of such embankments (Gilboy, 1934). In August 1967, Cooke was again summoned, this time to comment on seepage and differential settlement of the rock-fill dike, after three and a half years of operation. In a letter to Union Electric in 1967, Cooke (1967) noted that the average settlement of approximately 0.10 ft/yr (>3 cm/yr) during the first few years of operation was unprecedented when compared to other rock-fill dams, but was acceptable. 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.”

Settlement continued at a diminishing rate up to the time of failure in December 2005, at which time differential settlements were approaching 2 ft (0.6 m), reducing the crest height of the reservoir's parapet wall (Figure 8). The differential settlement also caused cracking of the concrete lining, which exacerbated ongoing problems with leakage, which also worsened with time (until the geomembrane liner was placed in 2004, described next).

**Taum Sauk Upper Reservoir Parapet Wall Crest Elevation**  
(Using MoDNR Data Collected 3-7-2006)

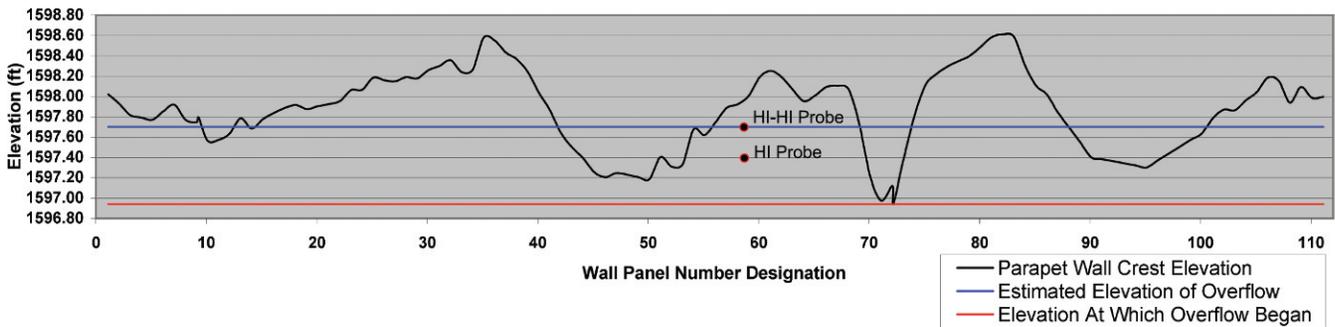


Figure 8. This figure illustrates the differential settlement along the crest of the Upper Reservoir parapet wall and the positions of the Warwick probes at the parapet wall at panel 58. Four segments of the parapet wall were almost 2 ft (61 cm) lower than their original elevation. The elevations of the breached panels (shown here in red) were estimated by AmerenUE after the failure (data from MoDNR, 2006).

**INSTALLATION OF GEOMEMBRANE LINER**

During the first 35 years of operations, the Upper Reservoir operated about 100 days per year, typically employing a 60-ft drawdown of the 94 ft of available storage within the diked enclosure. This tended to lessen undesirable vortex effects around the glory hole entry to the headrace tunnel at the south end of the Upper Reservoir. A 60-ft drawdown provided 6 to 7 hours of dual-unit power generation. During the peak power demands of the hot summer months, the plant employed dual generation cycling; generating power in the mid-morning hours, and then pumping water back into the Upper Reservoir during the afternoons, followed by another cycle of power generation in the evening, and then capped by

refilling during the early morning hours. In the fall, winter, and spring, the plant typically generated electricity only one cycle per day.

Between 1964 and 1999, the Upper Reservoir experienced repeated episodes of accelerating leakage that necessitated the employment of mitigation measures. These remedial measures periodically reduced the losses to acceptable levels, usually between 18 and 50 cfs (0.51 to 1.4 m<sup>3</sup>/s) (Figure 9). In 1999, the project utilization underwent a three-fold increase, to two cycles of filling and draining about 70 percent of the reservoir volume ~300 days per year. This change in the operation tempo soon exacerbated the leakage problems, triggering increased losses of between 100 to 300 percent (Figure 9). By mid-2000, the Upper Reservoir was losing about 110 acre-ft

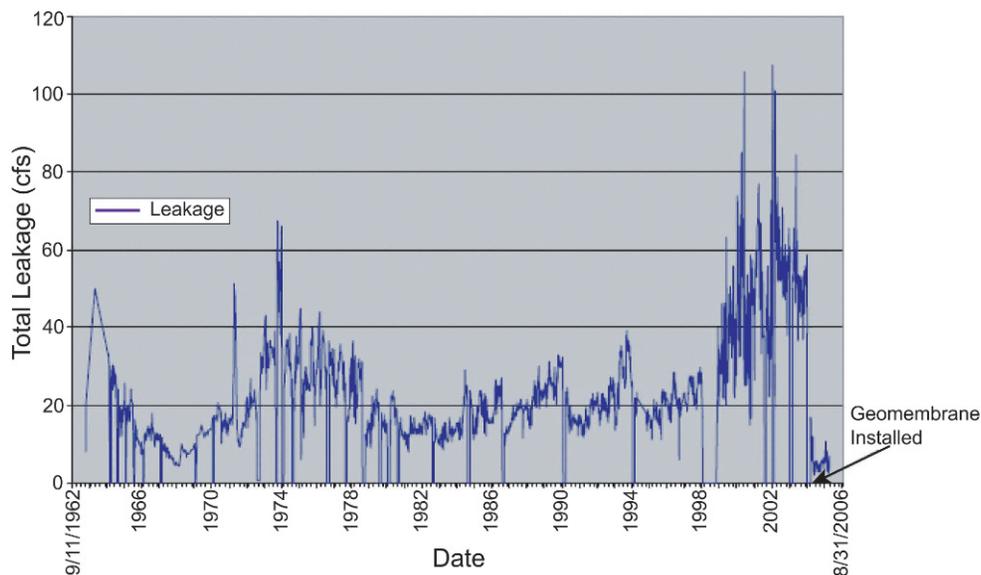


Figure 9. Leakage from the Upper Reservoir exhibits a gradual upward trend until the basin was lined in 2004, dramatically reducing seepage losses (taken from FERC Independent Panel of Consultants Report, 2006).

(135.7 m<sup>3</sup>) of water each day (Tomich and Leiser, 2006). Some of this water was collected in small ponds and pumped back into the Upper Reservoir to maintain efficiency. In the fall of 2004, Geo-Synthetics Inc. (GSI) was contracted to line the Upper Reservoir with a geomembrane at a cost of ~\$2.4 million to provide a permanent solution to the worsening leakage problems. Between September and November 2004, GSI supervised the placement of 1.3 million square feet (0.12 m<sup>2</sup>) of 80 mil high-density polyethylene (HDPE) textured geomembrane and geocomposite material as a seepage membrane. 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 (Figure 9). Leakage rates dropped by an order of magnitude, from an average of around 50 cfs (1.4 m<sup>3</sup>/s) to about 5 cfs (0.14 m<sup>3</sup>/s), and the overall efficiency of the facility reached ~70 percent (FERC, 2004, 2005). In December 1997, Union Electric and the Central Illinois Public Service Company merged, creating Ameren Corporation and its principal operating companies—AmerenUE and AmerenCIPS. AmerenUE assumed responsibility for operating the generating plants in Missouri.

#### DEREGULATION LEADING TO INCREASED UTILIZATION

Prior to the deregulation of electric power markets by the National Energy Policy Act of 1992, the Taum Sauk facility operated approximately 100 days a year. 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 the generation facilities they supervised (Leonard, 2005, 2006, 2007). Increased utilization likely influenced the decision to upgrade the pump/turbine units in 1999, which increased the efficiency and profitability of the plant.

#### PLANT RECOGNITION AND INSTRUMENTATION PROBLEMS

The Institute of Electrical and Electronics Engineers (IEEE) declared the Taum Sauk Plant an “Engineering Milestone” in a ceremony staged at the facility on September 26, 2005. Some AmerenUE employees visited the Upper Reservoir on Sunday,

September 25th, the day before the ceremony. They found water pouring over the parapet wall along the northwest portion of the reservoir, in what came to be described as the “Niagara Falls incident” (Tomich and Hand, 2006). Operators quickly shut off the pumps and flipped on the generating units to lower the reservoir as quickly as possible. Normally, workers would not have been onsite on a Sunday morning.

Inspections after the September 25th overtopping revealed new erosion rills on the rock-fill embankment up to 1 ft (30.5 cm) deep. AmerenUE thought wind-whipped waves from the remnants of Hurricane Rita caused the overtopping, but the reservoir level was well above the design freeboard.

Media reports suggest that a second, minor overtopping occurred on September 27th, two days after the initial waterfall incident. On this occasion, water levels were observed 4 in. (10 cm) from the top of the parapet wall (when the water level was supposed to be maintained at least 24 in. [61 cm] below the parapet wall), and moisture was noted on the land side of the wall panels, suggesting minor overtopping had occurred earlier that morning. No direct observations of overtopping were made, only the inferences noted here.

According to newspaper 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 this second overtopping incident. Divers were summoned, and they soon ascertained that the new sensor conduits had become detached from their mountings along the sloping concrete face of the reservoir at panel 58. Maximum water levels in the reservoir were supposedly reprogrammed to reduce the operating level by an additional 2 ft (61 cm) to provide an additional “margin of error” to account for the loose stage instrumentation conduits. Permanent repairs were postponed until the annual maintenance period, scheduled for the following spring, to avoid an unscheduled shutdown of the facility (FERC Taum Sauk Investigation Team, 2006). Figure 10 is a photo taken by AmerenUE on October 5, 2005, when the reservoir pool was at its twice daily low stage, about 30 percent full. This shows that the sensor conduits have detached from their anchorages.

It was assumed that the additional 2-ft (61 cm) increment would double the operating freeboard, increasing it to 4 ft (1.22 m), and that this would be sufficient to preclude any additional overflows until repairs could be completed during the reservoir’s annual maintenance shutdown in the late summer of 2006. This reasoning ignored the almost 2 ft of

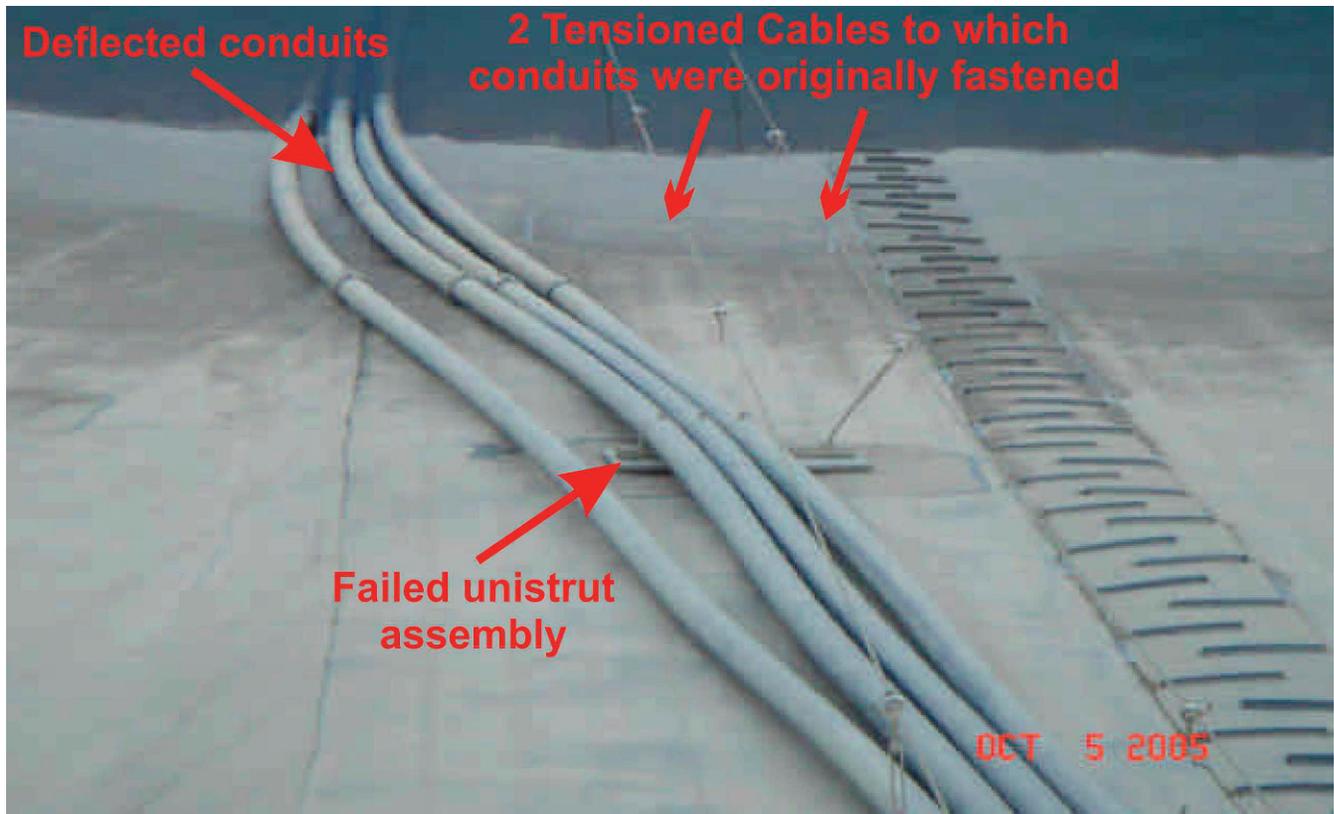


Figure 10. Photo showing the linear distortion of the four reservoir stage instrumentation array conduits after they detached from the unistrut anchors (taken on December 15, 2005, by David J. Hoffman).

settlement in four different locations about the perimeter of the Upper Reservoir (Figure 7). The reservoir was also equipped with two “fail-safe” Warrick probes, which were intended to shut down the pumps automatically if the water level reached their elevations on the parapet wall (using conductivity sensors, triggered by submergence). The Warrick probes were intended to be a “fail-safe backup” in case the reservoir stage sensors in the submerged conduits somehow failed or malfunctioned. When the lower Warrick probe activated, one of the pumping units was programmed to shut down immediately (instead of the normal method of gradually decreasing the inflow), and the other unit would shut down when the upper probe became submerged. They actually worked on the morning of the failure, but too late to forestall 10+ minutes of overtopping at those portions of the parapet wall that had settled as much as 2 ft (0.61 m).

#### LIKELY FAILURE SEQUENCE

The 6,562-ft-long (2,000 m) parapet wall was composed of 111 panels, with average lengths of about 60 ft (18 m). These were numbered between 1 and 111, as shown in Figure 6. These numbers are

referred to in the descriptions that follow. Figure 6 also shows the locations where overflow occurred (shown in yellow and blue) and the breach that drained the reservoir (shown in red).

In the pre-dawn hours of Wednesday, December 14, 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 because their absolute elevations were mischaracterized by 3.9 ft (1.2 m). Pump 2 shut down at 4:43 AM, but pump 1 did not shut down until 5:16 AM (right about the time the parapet wall between panels 88–100 toppled over). Water began pouring over the reservoir’s parapet wall in four areas that had settled (shown in Figure 7). Data recovered from the reservoir’s control system and back-calculations indicate that the overflow likely initiated over panels 71–72 (elevation 1,596.95 ft) around 5:03:20 AM; over panel 50 (elevation 1,597.18 ft) beginning at 5:06:20 AM; and over panel 95 (elevation 1,597.3 ft) around 5:09 AM (see reservoir stage relation in Figure 11). The parapet wall eventually failed between panels 88 and 100, though physical evidence of overtopping was also observed below panels 100–103, 70–74 on the west

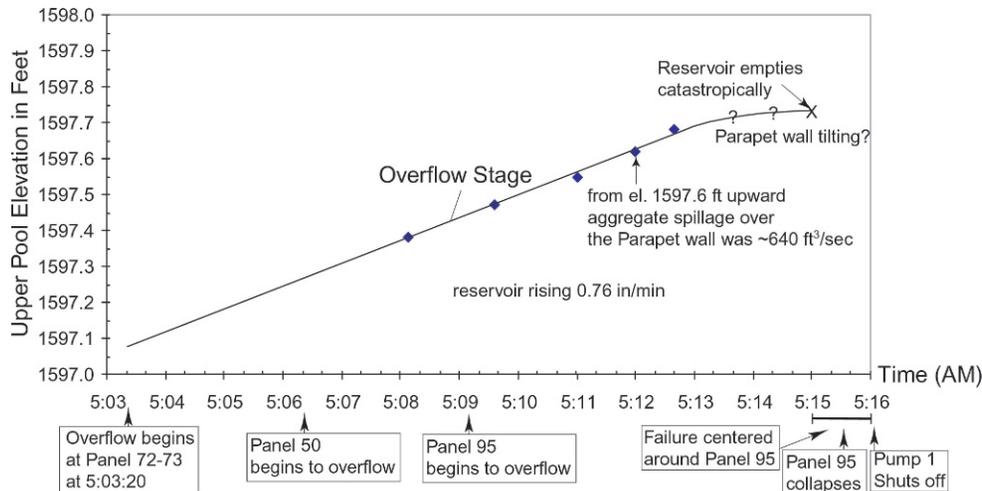


Figure 11. Reconstruction of Upper Reservoir stage versus time during the interval when the reservoir breached, between 5:03 and 5:16 AM on December 14, 2005. Note how the three principal zones of spillage began at 5:03, 5:06, and 5:09, with the last zone being the one that undercut the parapet wall first. The wall toppled over just about the same time that the last pump shut down.

side, and 44–53 on the southeastern side. Pump 2 had been programmed to shut down when the reservoir water level was 4 to 6 ft (1.2–1.8 m) below the crest of the parapet wall, and pump 1 was supposed to “top off the pool,” typically shutting off about 30 minutes later, when the water level came within 3 to 5 ft of the wall crest, but pump 1 continued running for another 33 minutes (5:16 AM), when the depth of overflow at panel 95 was likely close to ~4.3 in. (10.9 cm). The second pumping unit was originally programmed to shut down when the reservoir rose to within 2 ft (61 cm) of the wall crest, but it was supposed to have been re-programmed after the “Niagara Falls” incidents in late September. Based on back-calculations in a series of reports (Rizzo & Associates, Inc., 2006; FERC Independent Panel, 2006; and FERC Taum Sauk Investigation Team, 2006), the reservoir was likely within seconds of failing or just failed when the second pump unit shut down at 5:16 AM.

Prior to the collapse of the parapet wall, the spillage would have resembled flow over a broad crested weir, with an extremely broad V-shape over a distance of almost 900 ft (274 m) between panels 88 and 101. This condition would have trained more discharge toward the center of the settled section (see Figure 7), where an access road created a second bench a short distance down the descending slope (which may also have influenced flow concentrations on the embankment).

The discharge passing over the parapet wall initially spilled onto the wall’s outboard footing, which was 3 ft (91.4 cm) wide. When the depth of overtopping exceeded 4.1 in. (10.4 cm), the lower nappe of the spillage would have begun spilling onto the unprotected embankment, scouring the

unpaved aggregate base and underlying rock fill (Figure 12).

Post-failure observations at panels 10–12, 44–53, and 70–74 suggest that the water quickly scoured deep plunge pools where it poured directly onto the contaminated rock fill, which contained a fine-grained matrix. The eroded debris was rapidly transported downslope and deposited in debris fans just beyond 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 descending face of the dike (1.3:1, horizontal to vertical) would have exacerbated this situation, allowing for the lateral removal of blocks along the descending slope. This would have allowed a much deeper plunge pool to develop. This inference is drawn from post-failure observations below panels 71–72, shown in Figure 13.

An intriguing aspect of the Taum Sauk overtopping was the apparent contrasts in erodibility between various locations about the embankment. Figure 14 compares the cross-sectional areas of overflowage in the three zones that allowed the greatest volume of water to pass onto the rock-fill embankment. As shown in Figure 11, spillage initiated around 5:03:20 AM at panels 72–73, where the parapet wall had settled the most, to a crest elevation of just 1,596.95 ft (486.73 m). This zone was the most restricted of the three largest spill zones, being only 322 ft wide at the time of failure, when the water reached a level of 1,597.7 ft (486.96 m). Water next began pouring over panel 50 (elevation 1,597.15 ft) about two minutes later (5:06:20 AM). By 5:14 AM, when the water level



Figure 12. Blue arrows representing the overflow nappe when it began extending beyond the 3-ft-wide (91.44 cm) wall footing and directly onto the underlying rock fill. The rate of scour and erosion increased dramatically at this juncture. At 5:15 AM, only 6 minutes after overtopping initiated, the 60-ft-long (18.3 m) segment near wall panel 95 toppled, unleashing a 15-ft-deep (4.6 m) mass of flowing water over the remaining embankment and concrete liner (Figure 17). Photo courtesy of David J. Hoffman.

reached 1,597.7 ft, the pour-over zone between panels 44 and 54 was about 730 ft (222.5 m) wide. This zone actually had 9 percent greater overflow area than panels 88–99 (Figure 14). This would have allowed about 16 percent greater discharge to flow over panels 44–54 because the parapet wall was lower in that area and the water began spilling several minutes sooner.

There were two principal differences between the embankment at panels 44–54 and 88–99. The first was that the upper embankment in the vicinity of panels 46–47 had been armored with concrete to reduce runoff-induced erosion; and the second was that the embankment contained visibly fewer fines in this area as compared to that observed in the excavated faces of panels 88–91 and 98–101 after the failure (described later under “Fill Contamination”). The lower extremities of the embankment between panels 44 and 54 was a much cleaner rock fill than anything observed between panels 89 and 101 (the lowest elevation along the parapet wall was along panels 70–

74, but this was a much shorter reach than 44–54 or 88–99, so much less water passed over this section, as shown in Figure 8). It would appear to have been more able to pass the imposed seepage that spilled into the voids between the block, affording itself a much better degree of subdrainage.

The overtopping flow centered about panel 95 appears to have undermined three adjacent panels of the parapet wall in 6 to 7 minutes, eventually triggering overturning of at least one wall panel around 5:16 AM. Excerpts from our reconstructed failure sequence and complementary photos illustrate the likely failure mode by undercutting (see Figures 15 through 24).

The outbreak flood water surged down the upper slopes of Proffit Mountain, reaching a peak volume of 289,000 cfs (8,184 m<sup>3</sup>/s) where it poured out of the Upper Reservoir, around 5:20:30 AM (Rydland, 2006). This volume of water was sufficient to strip its flow path of vegetation and soil into the



Figure 13. This photo, taken below panels 71–72, shows the plunge pool excavated beneath the parapet wall, which completely undermined the wall and is the basis for Figure 16 (photo courtesy of David J. Hoffman).

underlying bedrock and expose the geology of the reservoir foundation. The flow was highly turbid and included rock fill, 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 ft (30.38 m) before entering the floodplain of the East Fork of the Black River, where the peak flow reached  $95,000 \pm 5,000$  cfs ( $2,690 \pm 141.6$  m<sup>3</sup>/s), and the depth of the flooding was about 34 ft (10.4 m) just upstream of the Shut-ins (Rydlund, 2006). The park was heavily damaged and filled with debris. Rydlund (2006) estimated that approximately 289,722 cubic yards (221,508 m<sup>3</sup>) of non-organic debris was deposited between Highway N and the Lower Reservoir, along the East Fork of the Black River. This estimate excluded the sediment that was carried into the Lower Reservoir, or passed over the Lower Taum Sauk Dam. Most of the discharge and debris were captured by the Lower Reservoir, which limited the spillage over the lower dam to just 1,600 cfs

(45.3 m<sup>3</sup>/s). Downstream damage was limited to increased turbidity of the Black River from silt and other fines carried over Lower Taum Sauk Dam.

#### KEY FACTORS CONTRIBUTING TO THE FAILURE

##### “Dirty” Rock-Fill Embankment

After the reservoir failure, the Federal Energy Regulatory Commission (FERC), the Missouri Department of Natural Resources, Dam and Reservoir Safety Division (MoDNR-GSRAD), and Paul Rizzo & Associates conducted investigations into the failure. Although the embankment was intended to be clean rock fill (less than 5 percent passing the No. 200 sieve), an excessive amount of fine-grained material was visually recognized in the exposed dike after the failure. Forensic analyses by FERC’s Taum Sauk Investigation Team (2006), FERC’s Indepen-

## 2005 Upper Taum Sauk Dam Failure

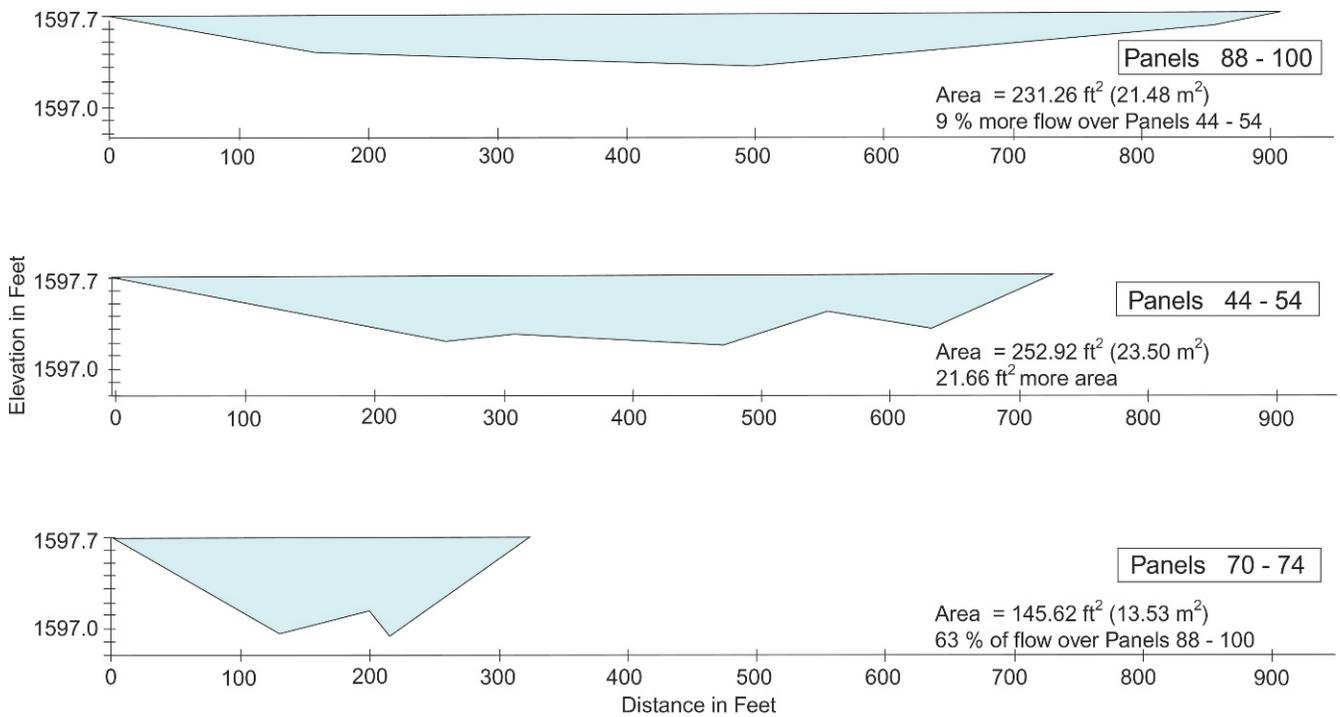


Figure 14. Comparative cross sections along the crest of the Upper Reservoir between panels 70 and 74, 44 and 54, and 88 and 101. Note how the greatest cross sectional area and volume of spillage appears to have occurred between panels 44 and 54, not where the breach occurred between panels 88 and 99.

dent Panel (2006), and Rizzo & Associates, Inc. (2006) confirmed a fines content of between zero and 26 percent in the exposed embankment bordering the breach (shown in Figures 7 and 25). In addition, the percentage of sand-sized material within the embankment was as great as 45 percent in certain locations, creating a far more erodible mixture than normally expected of clean rock fill (FERC Independent Panel, 2006). This problem was manifest by the unusual level of surficial erosion during the project's 42-year life, simply from rainfall-induced runoff (FERC Independent Panel, 2006). Cooke (1967) surmised that

portions of the dike could not have been clean rock fill because the observed settlement was almost an order of magnitude greater than those recorded on other concrete-faced rock-fill dams (prior to 1967).

### Weathered Material in Embankment

The design section for the original embankment (Figure 26) indicated that only the surficial cover was to be removed, leaving no more than 2 in. (5 cm) of soil or residuum in place. This recommendation assumed that everything beneath the soil would

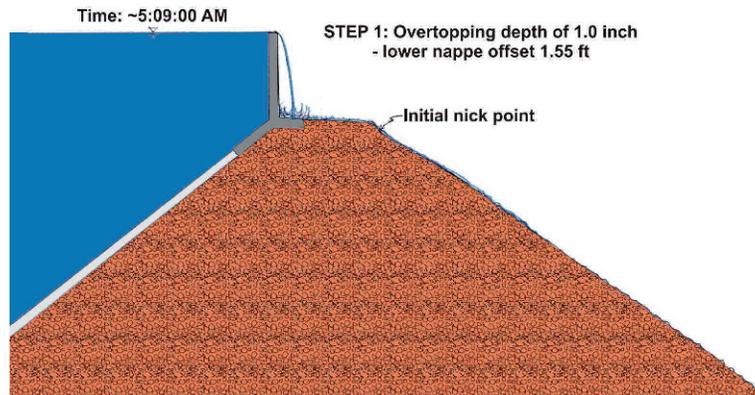


Figure 15. This presents the first figure from the reconstructed failure sequence illustrating the onset of overtopping around 6 minutes prior to failure. Water is spilling onto the 3-ft-wide (91.44 cm) parapet wall footing and just beginning to erode the crest of the embankment.

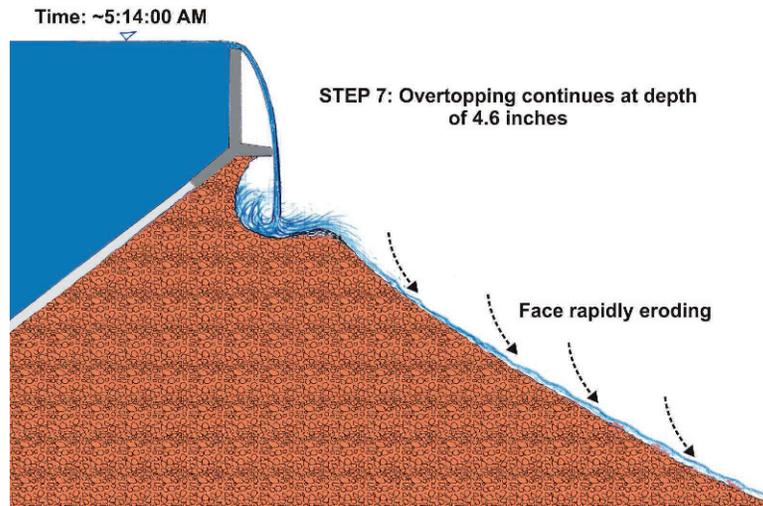


Figure 16. This step of the reconstructed failure sequence illustrates the undermining of the parapet wall and rapid erosion of the upper embankment. The wall remained standing at this point due to the hydrostatic load on the inclined portion of the wall footing and three-dimensional effects engendered by the adjacent panels.

provide adequate bearing for the rock-fill dike. The December 2005 outbreak flood exposed deeply weathered zones in the upper slope of Proffit Mountain. The breach area was underlain by a zone of deeply weathered diabase saprolite, the likely remnant of a disintegrated dike swarm (see Figure 27). Adjacent core stones of granite and rhyolite also exhibited a high degree of weathering, likely from hydrothermal alteration associated with intrusion of the diabase. The weathered diabase exhibited a soil-like texture, though it retained the rock's original fabric and fracture patterns. Core stones and remnant spheroidal weathering rinds were also visible (Figure 27).

#### Erodibility of Embankment Materials

The principal difference between panels 44–54 and 88–99 was that the rock-fill embankment was visibly more contaminated with fines at 88–99 as compared with 44–54, even though that section was somewhat steeper and higher. The fill exposed in the breach between panels 88 and 89 was an earth–rock-fill mixture, containing 0.06 and 26 percent of the material passing the No. 200 sieve (Rizzo & Associates, 2006). This percentage of fines is atypical of rock-fill embankments, which usually restrict fines contents to less than 5 percent (Cooke, 1984).

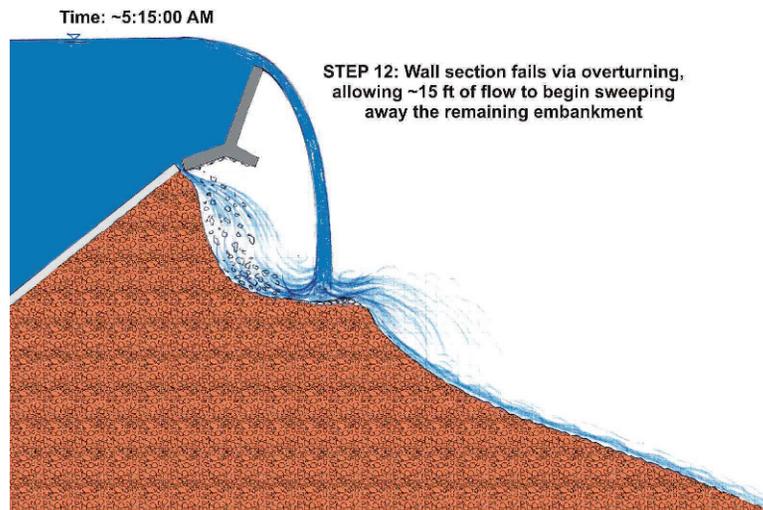


Figure 17. This step portrays how the parapet wall likely toppled in vicinity of panel 95, initiating the catastrophic failure of the reservoir by suddenly unleashing approximately 15 ft (4.6 m) of flow over the remaining embankment.

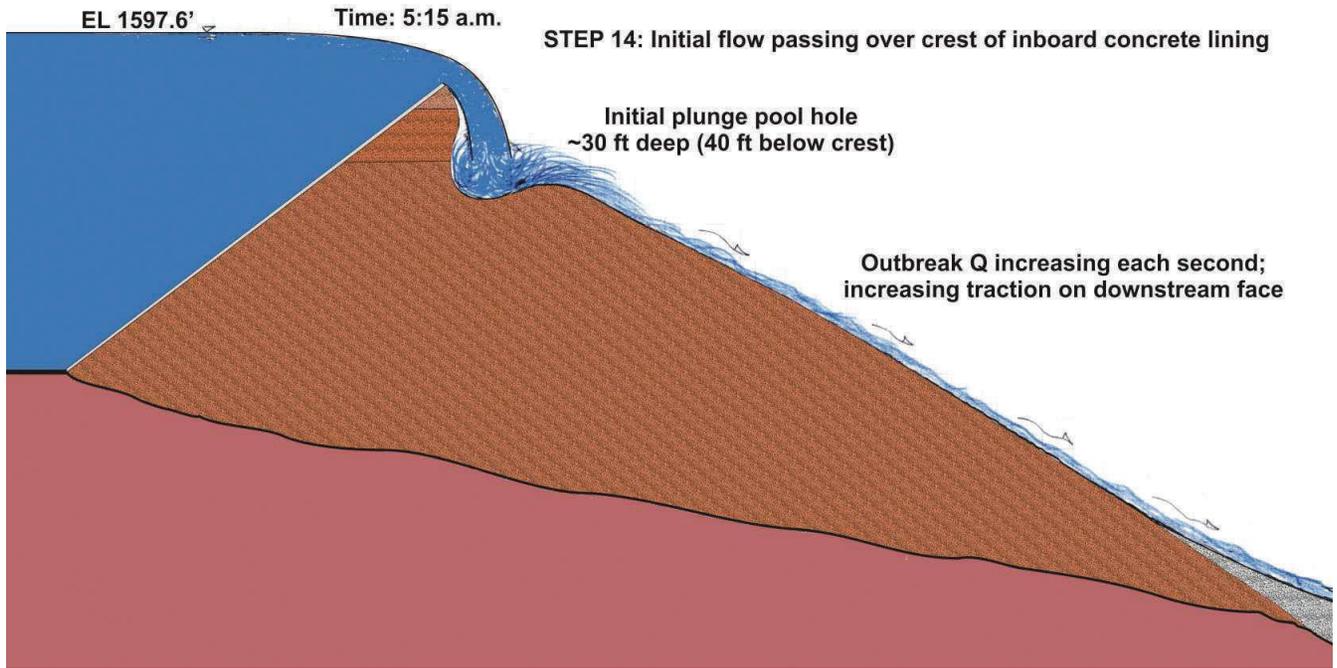


Figure 18. This figure illustrates approximately 15 ft (4.6 m) of water overflowing the embankment immediately after toppling of the parapet wall. The reinforced concrete liner behaved as a thin-crested weir, promoting the formation of a deep plunge pool.

This marked disparity in fines content could be expected to influence erodibility by water rushing down the face of an inclined embankment. Briaud (2008) presented tables that depict the critical velocity for scour as a function of mean grain size, shown in Figure 28. This chart suggests that gravel and rip-rap

can begin to experience scour problems at a velocity of 3.2 ft/s (1.0 m/s) up through 82 ft/s (25 m/s). We can gain an idea of the potential velocity ( $v$ ) of the discharge spilling over the parapet wall and down the steep embankment by considering the relationship between kinetic and gravitational potential energy,

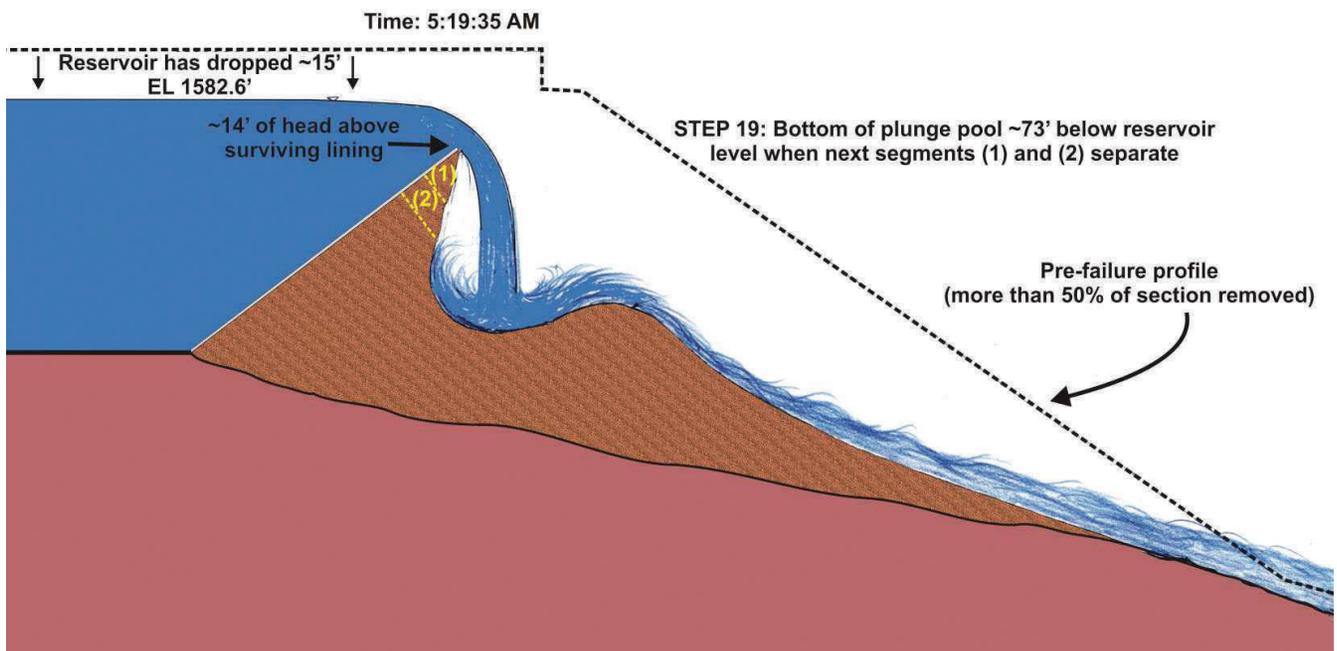


Figure 19. The embankment and concrete liner are progressively removed by the outflow. About half of the embankment remains at this stage.

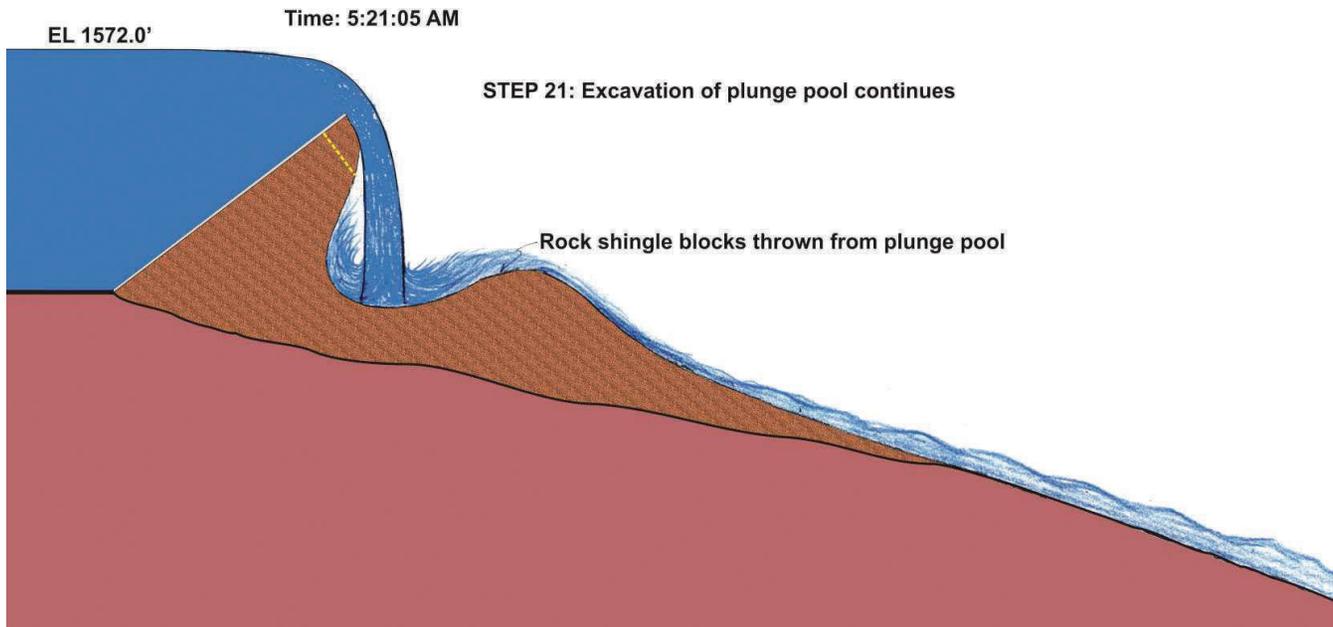


Figure 20. A continuation of the sequence shows deepening of the plunge pool and ejection of large shingle blocks/boulders from the plunge pool.

given as:

$$1/2mv^2 = mgh \quad (1)$$

By grouping units to eliminate the mass, the potential velocity can then be described by:

$$v = (2gh)^{1/2} \quad (2)$$

The height of the water free fall,  $h$ , had an initial value of 10.38 ft (3.16 m) (over the parapet wall), increasing to approximately 30 ft (9.14 m), shown

schematically in Figure 17. These combinations would suggest that the impact velocities likely began around 26 ft/s (7.87 m/s) and increased to as much as 44 ft/s (13.4 m/s), which would be expected to erode rock rip-rap (Figure 28). In this manner, the rapid excavation of the “plunge pool” immediately beneath the embankment crest would be expected.

As shown in Figure 29, when the critical shear stress,  $\tau_c$ , exceeds 2.1 lb/ft<sup>2</sup> (100 N/m<sup>2</sup>) for median grain sizes ( $D_{50}$  values) greater than ~4 in. (100 mm), velocity-triggered erosion of such materials can be

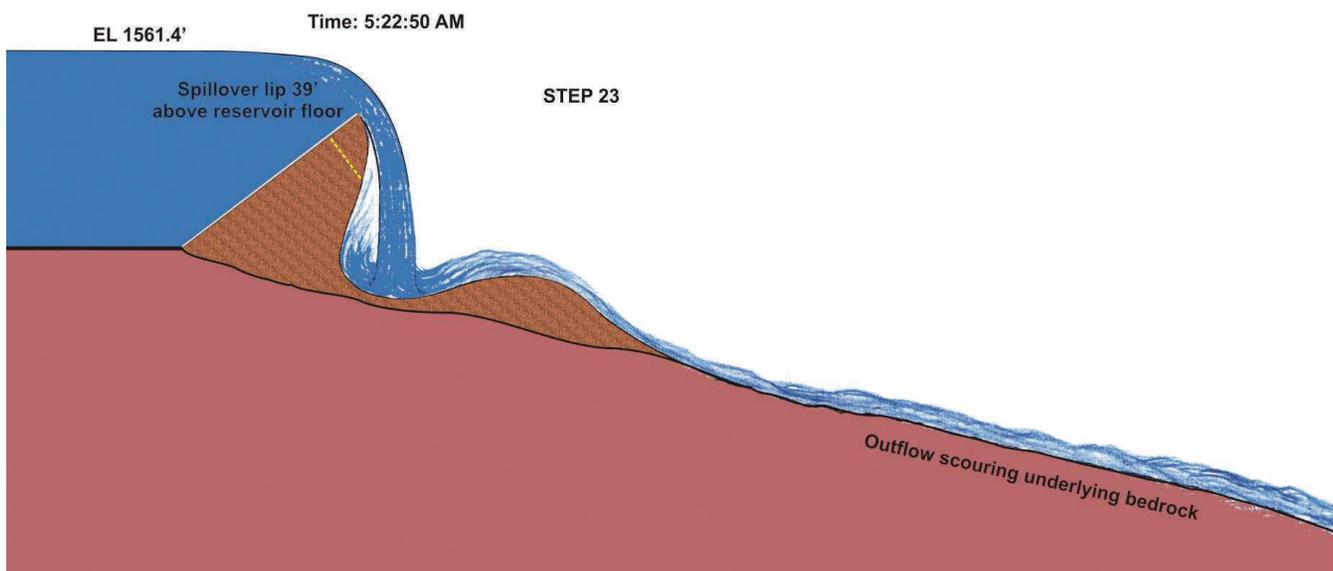


Figure 21. The plunge pool deepens toward the foundation interface and outflow has exposed much of the underlying bedrock, which is then scoured, exposing zones of saprolite.

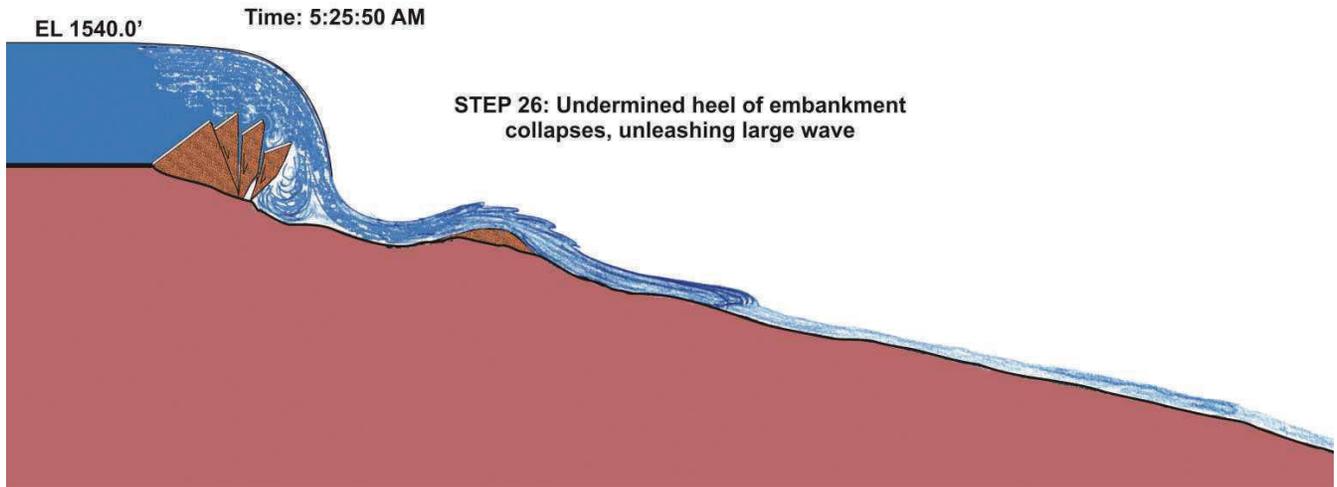


Figure 22. The embankment undergoes one last block collapse involving the concrete liner, sending one final surge down the mountain side, which deposits a coarse imbricate layer over finer-grained strata in the natural swale below.

expected. The value of 4 in. (100 mm) was the upper range of  $D_{50}$  values for this embankment in vicinity of panels 88–99 (Rizzo & Associates, 2006). The striking contrast between panels 44–54 and 88–99 is their fines content. The rock fill placed at panels 44–54 appears much cleaner, with less fines content, even though it was significantly higher (125 ft versus 95 ft). It would appear that water spilling onto this portion of the embankment was more readily lost in its open void space, generating lower velocities and boundary shear stresses that fostered rapid erosion at the water-soil interface (Briaud, 2008). The embankment in vicinity of panels 70–74 and 88–99 was visibly more contaminated with fines. The matrix of fine-grained soils prevented infiltration of the overflowing water, allowing much higher runoff velocities to develop, causing considerably increased levels of erosion, because in low cohesion materials (sand and gravel), sediment carrying capacity is a power function of velocity (Colby, 1964; Toffaleti, 1969).

Erosion and deposition of non-cohesive sediment are affected by the amount of sediment the imposed flow is capable of carrying. If the amount of sediment being transported is less than the amount the flow can carry for the hydrodynamic conditions, material will be scoured from the bed upon which flow is occurring. This process continues until the actual rate of sediment transport reaches the carrying capacity of the flow or until the available bed material is all scoured. Conversely, deposition occurs if the sediment transport rate exceeds the flow’s capacity to carry the materials. In most cases, carrying capacity is a power function of flow velocity. The velocity of water falling vertically or flowing on slopes exceeding 37 degrees is sufficiently high to trigger rapid erosion of the rocky fill within a fine-grained matrix. The transition between eroded rills and fan deposits was easily observed below panels 44–56 (Figure 30) and 70–74, from which we may infer similar physical processes occurring below panels 89–99 before the

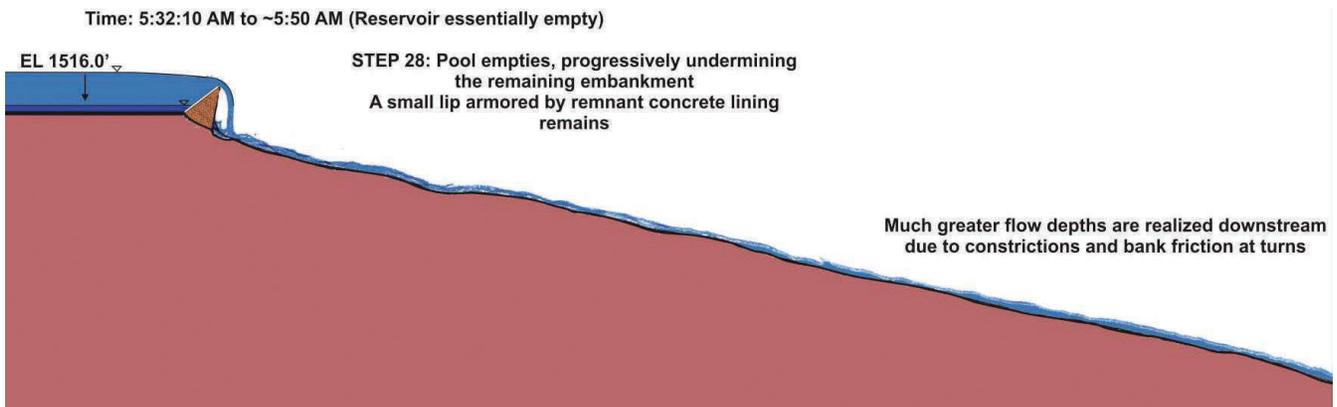


Figure 23. A small lip formed by the concrete liner armored the final last remnants of the embankment. Several feet of water remained in the basin formed by the “plinth” in the northwest corner of the Upper Reservoir.



Figure 24. The remnant lip as seen from immediately downslope, the day after the failure.

outbreak flood removed everything in the vicinity of the catastrophic breach.

A somewhat less intuitive aspect is the erosion of the embankment face, known as rill erosion, which is shown to good effect in Figure 30. Rills are erosion gullies caused by the running water when velocities exceed the critical shear stress of those materials. These contrasts can be clearly seen at the toe of the slopes at panels 70–74 (Figure 13) versus those at the toe of panels 44–54, which exhibited 1- to 8-ft-deep (0.30 to 2.44 m) rills (Figure 30). It would appear that eroded materials were being rapidly deposited along the lower quarter of the embankment, approaching the toe of the slope and transition to the native grade. Materials were being deposited in this area so rapidly that a massive rotational slump began to translate along the lower third of the embankment slope. This incipient slumping was likely being promoted by the sudden deposition of so much material, and excessive pore pressures developing in the lower slope due to the accretion of the infiltrated seepage, likely perched on the embankment-bedrock contact. This situation was also exacerbated by the diminishing cross-sectional area of the embankment in this same area, where the rock fill was placed over a steeply inclined descending slope. This slope would likely have collapsed within a few minutes had the embankment between panels 88–99 not failed first.

#### Insufficient Foundation Preparation

As mentioned previously, the design specifications only specified stripping of surficial soil cover to minimum thickness of 2 in. (5 cm) over the bedrock (Figure 26). The specifications also note that the remaining soils were to be saturated prior to placement of rock fill. 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 in. (46 cm) of residual soil had not been stripped off prior to placement of the rock fill in some locations. Soils containing tree roots and other organic matter were also observed beneath the breached section of the embankment (see Figure 31). This oversight likely exacerbated some of the seepage the project suffered during its initial year of operations, when the reservoir floor across this “plinth area,” behind panels 89–107 experienced problems with seepage caused by uplift and heave of the reservoir floor. The entire floor of the reservoir had originally been paved with two 2-in. (5 cm) layers of asphalt to curtail infiltration. However, as soon as the reservoir was filled, they began experiencing problems with seepage through cracks that developed in the asphalt, especially in the “plinth” area abutting the northwest corner. Between August 1963 and May 1964, leakage



Figure 25. Close-up of the contaminated “dirty rock fill” exposed on lateral margins of the breach. The larger clasts are between 10 and 12 in. (25.4 to 30.5 cm) in diameter.

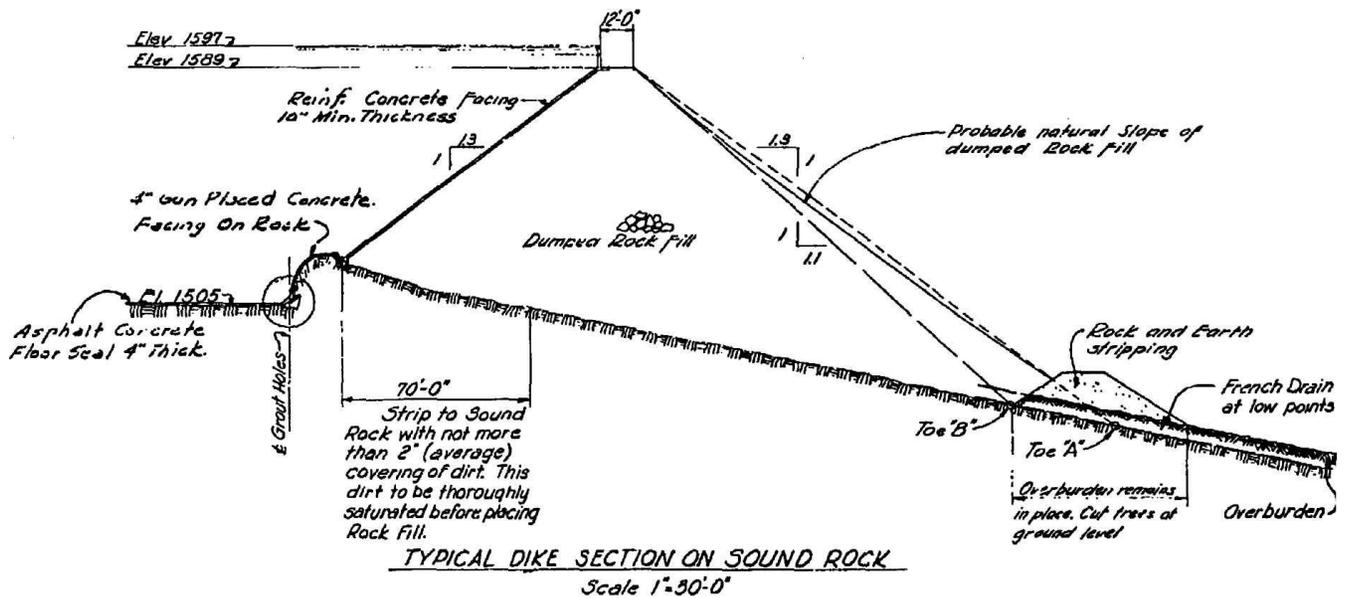


Figure 26. Typical section through the rock-fill dike where it was assumed to be founded on sound bedrock on a descending grade, take from the project specifications in 1959–60 (from FERC Taum Sauk Investigation Team, 2006).



Figure 27. 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.

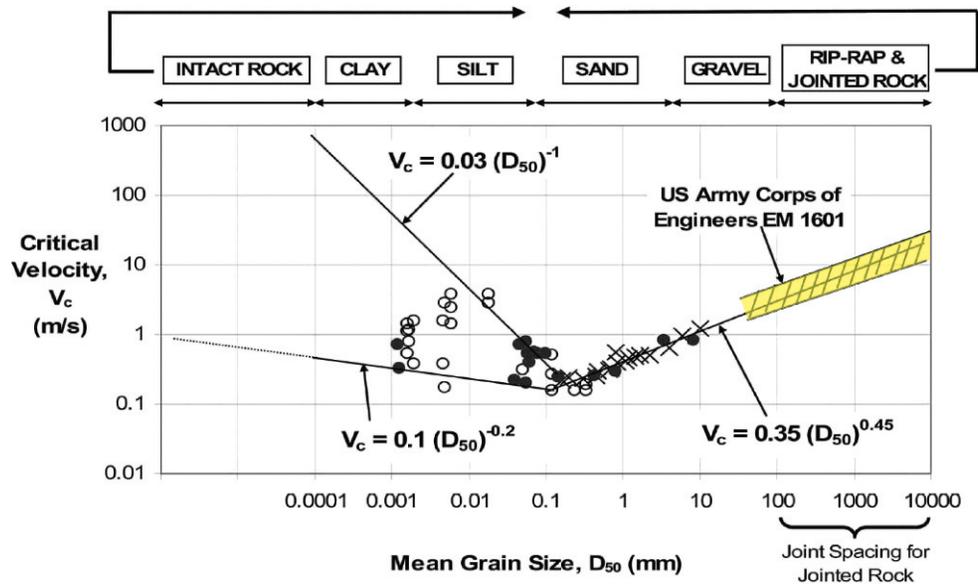


Figure 28. Critical velocity triggering erosion versus mean grain size, taken from Briaud (2008). The yellow shaded zone represents data for coarse gravel, rip-rap, and jointed rock. Sediment carrying capacity of non-cohesive materials like sand and gravel is a power function of velocity. Data points are from measurements at Texas A&M and Army Corps of Engineers Waterways Experiment Station, described in Briaud (2008).

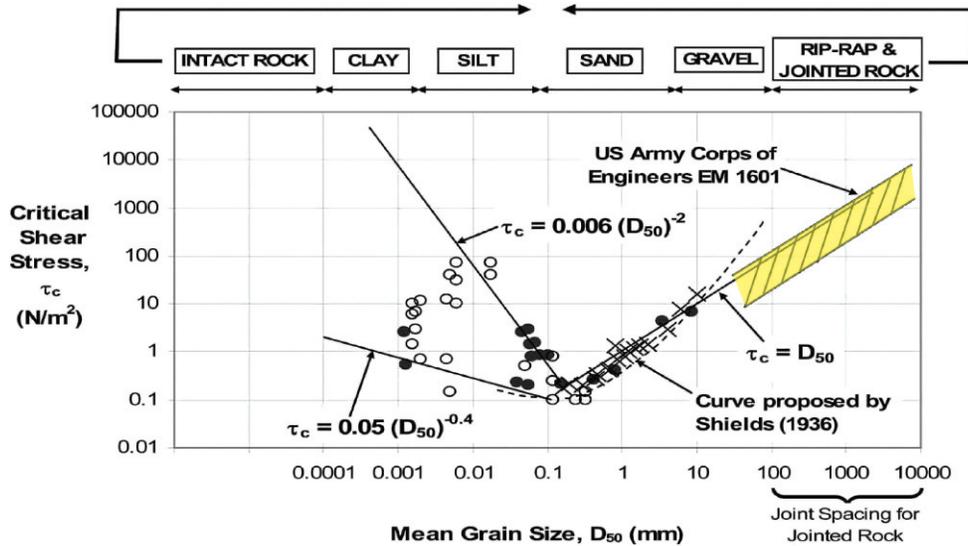


Figure 29. Critical boundary shear stress as a function of mean grain size, taken from Briaud (2008). The yellow shaded zone represents data for coarse gravel, rip-rap, and jointed rock. Data points are from measurements at Texas A&M and Army Corps of Engineers Waterways Experiment Station, described in Briaud (2008).

from this area varied between 40 and 103 cfs (1.13 to 2.92 m<sup>3</sup>/s), which required the reservoir to be shut down and drained to carry out increasingly more extensive repairs to alleviate the losses (described previously and documented in Pickel, 1964). The uplift problems were likely exacerbated by the immense permeability contrast between the reservoir floor slab, a 2-ft-thick (0.61 m) layer of coarse rock fill, and the underlying fine-grained soil residuum (Figure 32), which thickened westward, toward the downslope side. The outboard toe of the embankment in this area was placed over a deeply weathered diabase intrusion, which had altered to fine-grained saprolite. The local concentration of the saprolite is what likely contaminated the fill in this area. Under normal conditions, the thin layer of coarse rock fill would have served admirably as an underdrain beneath the floor slab, but its down-gradient outlet was probably blocked by the low-permeability waste dump placed at the toe, shown in Figure 25. This waste dump was where the stripped spoils of low quality and low permeability were to be segregated from the main rock-fill dike. Water that seeped into the pervious rock fill underlying the “plinth” area (Figure 32) could not easily percolate laterally when the Upper Reservoir drew down during its daily cycle of filling. This pore-pressure entrapment explains the severe uplift of the three generations of asphalt and concrete floor slabs that were laid across this area in an attempt to seal off the losses. The trapped water appears to have occasionally found outlets, which led to the erratic increases in observed seepage. A costly program of installation of cutoff walls, pressure

grouting, and numerous uplift relief wells eventually corrected the problem. This delayed power generation for six months and kept the plant out of operation during much of its first year of operation (Pickel, 1964; Gamble and Rudolph, 1964).

The increased fines content of the embankment in this area was likely exacerbated by the proximity of the diabase and granite saprolite zones, causing a slight natural depression abutting the mountain’s summit. The northwest corner of the dike had been over-excavated during construction, due to a “highly weathered zone” noted at the time (FERC Taum Sauk Investigation Team, 2006). The plinth area was the only portion of the Upper Reservoir floor that required placement of fill to be brought up within 5 ft (1.5 m) of the rest of the reservoir floor (nominally supposed to be at elevation 1,505 ft/458.7 m).

The dike reached 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 ft (274 m) long (Figure 8). The only area with greater settlement was in the vicinity of panels 71–72, but this was a very narrow zone. Another contributing factor to the excessive settlement in the low area was probably hydrocompression-induced consolidation, a common problem with hydraulically sluiced fine-grained soil mixtures (e.g., Brandon et al, 1990; Rogers, 1998). Placement of a highly heterogeneous fill mixture using hydraulic sluicing is known to create low relative density, leading to poor long-term performance (e.g., Lee and Fitton, 1969; Dyvik et al., 1984; and Mitchell, 1988).



Figure 30. a: Embankment erosion at panel 47, on the southeast side of the Upper Reservoir, the day after the failure. After the spillage penetrated the cohesive road-base cover (light pink color) for the crest road, it rapidly eroded the underlying rhyolite rock fill (lavender color) in a series of deep rills, caused by coalescing flows. The upper slopes in this area had been armored with shotcrete to abate excessive erosion from rainfall-induced runoff (photo by David J. Hoffman). b: Deep rill erosion below panel 46, on the southeast side of the Upper Reservoir, the day after the failure. This rill was about 40 ft (12.2 m) wide and 8 to 10 ft (2.4 to 3 m) deep. Note the contamination of the rock fill by 3 to 15 percent fines, seen in the exposed shoulders of the rill. These fines had been sluiced out of the fill within the rill by the high-velocity discharge (photo by David J. Hoffman).



Figure 31. Residual soils, including tree roots, were observed in-place beneath the “plinth area” in the northwest corner of the Upper Reservoir, where a portion of the embankment was founded on as much as 10 to 16 ft (3 to 4.9 m) of residuum and a thin layer of rock fill.

The inclusion of more than 20 percent fines and placement via hydraulic sluicing significantly reduced the relative density of the soil-rock mixture (e.g., Miedema and Farrar, 1988). Subsequent seepage into the embankment likely promoted hydrocompression of those materials, which increased with increasing effective stress. In this manner, the hydrocompression increased with increasing load, and the deepest portions of the fill may have settled as much as 8 percent (e.g., Noorany and Stanley, 1994; Rogers, 1998). Fill contamination by locally available fines would then serve to explain the uneven pattern of embankment settlement that was observed (Figure 8), which approached 2 percent of embankment height (along panels 88–99, where the embankment reached its maximum cross section).

#### Liner Installation and Flawed Re-attachment of Instrumentation in 2004

The Upper Reservoir’s monitoring system was anchored to the concrete lining prior to the fall of 2004, when it was replaced during installation of an

High Density Polyethylene (HDPE) geomembrane liner. The new sensor network was composed of four perforated HDPE conduits (Figure 10). The original design assumed that two of the conduits would contain pressure transducers, with the third serving as an extra, and the fourth conduit was supposed to be filled with concrete ballast to secure the array, since they would all be subjected to near-daily reservoir cycling. The original design specified that the four 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 be attached to the concrete face beneath the liner (Rizzo & Associates, 2006).

The alternative scheme employed a pair of untensioned steel cables passing through eye bolts that were only anchored to the concrete lining at two locations: at the toe and crest of the upstream face of panel 58. It was also decided to dispense with the concrete-filled ballast conduit, so this pipe was installed as a second spare, and the array was bereft of any ballast whatsoever.



Figure 32. Rock fill exposed in the plinth beneath the northwest corner of the Upper Reservoir. Several feet of clean rock fill was placed over the rhyolite bedrock, and then covered with a 4-in.-thick asphalt liner, which failed miserably under uplift when the reservoir was initially filled in 1963. The asphalt was replaced by several generations of reinforced concrete mats in 1963 and 1964, which were subsequently covered by the HDPE geomembrane liner in 2004 (taken from FERC Taum Sauk Investigation Team, 2006).

The eye bolt scheme was then discarded in favor of turnbuckles, so the anchor cables could be adjusted in response to the rising and lowering of the reservoir pool. Unfortunately, the turnbuckles were not locked after being tensioned, and they appear to have loosened themselves during the daily cycles of filling and emptying the reservoir. The four conduits were attached to one another every 20 ft (61 m) with a clamp assembly known as unistruts. A unistrut is a series of galvanized U-bolts fastened to a flat galvanized steel bracket. At each bracket, there were four U-bolts, one wrapped around each conduit and attached to the unistrut anchor plate. However, nothing was holding the unistrut anchors to the upstream face of the dike, just the two steel cables. The cyclic uplift caused by near-daily reservoir filling and draining gradually loosened the instrumentation 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 were trapped inside of them. The two anchor cables running through the unistrut anchors were unable to secure the array because their turnbuckles also managed to become loosened.

As the turnbuckles failed, the unistruts were subjected to additional cyclic stresses and began slipping sideways. Once the four conduits were no longer attached to each other, they began to deform individually, instead of collectively (Figure 10). Since the individual stiffness of the conduits was less than their cumulative stiffness, the failure of the anchorage system played a seminal role in the eventual failure of the stage instrumentation system.

The base of the instrumentation array was also located approximately 120 ft (36.6 m) from the glory

hole inlet, which pumped water into the reservoir at a rate of 5,238 cfs (148 m<sup>3</sup>/s). 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. Due to these problems, the sensor readings were erroneously low, transmitting a reading of 1,593.7 ft when the actual pool level was 1,597.6 ft, a difference of 3.9 feet (1.19 m) (FERC Independent Panel, 2006).

#### Elevation Discrepancies Introduced during Liner Installation

The Upper Reservoir was designed to have 2 ft (61 cm) of freeboard between the water surface (elevation 1,597 ft) and the top of the parapet wall (elevation 1,599 ft). 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 1 ft (30.5 cm) over 42 years. The old gauging system was operated relative to this staff gauge at panel 58, so freeboard at the staff gauge remained constant throughout the years, even as the embankment settled. Unfortunately, differential settlement at several other locations about the reservoir perimeter was greater. After the HDPE lining was installed, two new staff gauges were affixed to the parapet wall at panel 58. These gauges were installed about a foot too high because AmerenUE engineers assumed the elevations shown on the original project drawings were still valid. As a consequence, the HI probe was installed at elevation 1,597.4 ft, and the HI-HI probe was installed at 1,597.7 ft (shown on Figure 7), which were too high to be effective, as panels 44–54, 70–72, and 90–96 were all below 1,597.4 ft. This error resulted in a 1-ft (30.5 cm) reduction of absolute freeboard, which unknowingly lowered the margin of error against overtopping.

#### Error in Programming of “Fail-Safe” Probes

The Warrick auto-stop probes failed to activate during the two overtopping incidents in late September, so AmerenUE operators decided to add an additional 2 ft of operating freeboard by programming their filling cycles to cease at a lower stage level. The mistake in assumed elevation of the Warrick probes was not discovered until after the December 2005 failure. The probes were also programmed with a 1-minute delay before alarms would sound and the pumps would shut down (FERC Independent Panel, 2006; FERC Taum Sauk Investigation Team, 2006). This delay would have allowed between 0.75 and 1.5 in. (1.91 and 3.81 cm) 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 were activated for the programmed time (60 seconds). Although the HI limit probe was located 4.92 in. (12.5 cm) above the lowest point along the crest of the parapet wall, it could have kept an additional 3.6 in. (9.14 cm) 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 the 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 could not 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 been programmed with a proper delay, this error could have resulted in an identical failure if pumping unit 2 had been set to shut down last.

Water may have reached as high as 1,597.7 ft (486.98 m) due to the combination of the excessively high probe elevation, the programming of the probes to operate in series, and the programmed 1-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 because there was no designated dam safety engineer in the corporate structure. There was also no formalized procedure to test such changes to ensure they were properly implemented. Also, there was not 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 ft (30.5 cm) 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 ft [0.9 and 1.5 m]). This low margin of error was exacerbated by differential settlement of the parapet

wall, which allowed four other zones to be about a foot (30.5 cm) lower than assumed by the plant operators (FERC Independent Panel, 2006). Visual oversight of the pumped storage operations was recommended by Cooke (1967) and initially implemented by Union Electric soon thereafter (Weldy, 1968). Sometime between 1968 and the failure in 2005, visual oversight (by a Pinkerton guard) was discontinued as being an unnecessary precaution by AmerenUE management (probably because there hadn't been any safety incidents of note until the Niagara Falls incidents in September 2005; Tomich and Hand, 2006). The absence of visual oversight prevented the gradual deterioration of freeboard (due to progressive displacement of the instrumentation conduits) from being 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 3 ft (91.44 cm) would provide an adequate margin of error to account for the obvious stage instrumentation problems.

A retrospective review of the reservoir stage records suggests that something was awry with the stage instrumentation array because it repeatedly conveyed water levels that did 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 ft (30.5 cm) in 20 minutes with both pumping units on (it should have reported a 2.5 ft [76 cm] rise), and (3) a 1.9-ft (58 cm) 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 data were recorded in the facility's computers (FERC Taum Sauk Investigation Team, 2006). These anomalous stage records suggest that the submerged instrumentation array was probably moving around as all of its anchorages were detached (Figure 10).

### IMPACTS OF THE FAILURE

During the spring 2006 and 2007 legislative sessions, the Missouri governor and state legislature considered revising their dam safety act (i.e., established 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 engineer was 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 paid for all of the clean-up and repair activities at Johnson Shut-ins State Park. In 2007, FERC approved AmerenUE's plan to rebuild the Upper Reservoir on the same footprint and capacity that existed prior to the failure. A 2.9 million cubic yard (2.217 million m<sup>3</sup>) roller compacted concrete (RCC) dike, 100 ft (30 m) high, was constructed between October 2007 and December 2009. The RCC embankment was fitted with a weir spillway discharging over the eastern side of Proffit Mountain onto the Taum Sauk Creek arm of the Lower Reservoir, to prevent any spillage on Johnson Shut-ins State Park.

### Record Fine by FERC

The Federal Energy Regulatory Commission fined AmerenUE \$15 million, the largest fine ever assessed 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 September 25, 2005, overtopping to FERC;
- failure to report unusual instrument readings on September 27, 2005;
- failure to report the release of the transducer retention system;
- addition of 0.4 ft (12.2 cm) 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;
- movement of fail-safe probes to an elevation higher than the lowest point on the reservoir parapet wall;
- programming of system to have a 1-minute delay in pump shutdown after activation of probes;
- reprogramming of probes to operate in series instead of in parallel; and
- failure to program lowest of two probes 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 outcome of this failure could have been far worse had it occurred at a different time of year. Hundreds of unsuspecting campers would 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 toward 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 just five injuries (the miraculous survival of the park superintendent's family is described by Hendryx, 2006).

### CONCLUSIONS

Although multiple factors contributed to the disaster, these might never have culminated in a catastrophic failure had a simple spillway been included in the original design or subsequent retrofit. Those in the chain of corporate decision making assumed that the 3-ft (91 cm) *ad hoc* adjustment to the reservoir stage levels would suffice to account for the overt instrumentation deficiencies and thereby afford another 6+ months of operation, without quantifying the actual errors or verifying the mechanisms responsible for the erroneous readings. The two Niagara Falls incidents should have triggered more in-depth investigations and assessment of the problem by qualified engineers, not to mention reporting the incidents to FERC. Any engineered system is capable of malfunctioning for an array of reasons and/or circumstances, including aging and/or unforeseen circumstances. Many of the facility's operational vulnerabilities were adroitly pointed out in the first few years of operation by Cooke (1967), which were noted by the FERC Taum Sauk Investigation Team (2006).

The impact of the differential settlement of the dike should have been appreciated by whoever was responsible for replacing the 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 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 more severe load cycling than conventional storage facilities. In addition, last-minute connection details can often prove problematic. Hidden design and construction flaws can also cause unforeseen difficulties with operation and maintenance throughout the life of a reservoir.

The overflow incidents on September 25 and 27, 2005, should have triggered an active monitoring program, at the very least, to ascertain whether the problem was worsening with each cycle of filling and emplaced some measure of observation to monitor the situation.

In conclusion, the principal contributing factors appear to have been a series of errors in human judgment. It is estimated that only 6 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 gauges 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 panel made up of people with substantive experience in the operation of pumped storage and hydroelectric generation operations, and at least one individual familiar with stage instrumentation schemes and systems.

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