Section 8 Hydrology and Hydraulics

8.1 Upper Dam

The Upper Dam has no spillway. It's outlet works consists of a 451-foot long, 27.2-foot diameter, vertical shaft, the top 110 feet of which is concrete lined; a 4,765-foot long, 25-foot diameter unlined horseshoe tunnel sloping at 5.7 percent; a horizontal 1,807-foot long, 18.5-foot diameter steel lined tunnel; and a short penstock that bifurcates to the pump-generating plant. The shaft bellmouth intake is located in the southwestern portion of the reservoir in a localized area of the floor that is 20 feet lower than the rest of the reservoir floor to suppress vortex development.

8.1.1 Drainage Area/Surface Area/Storage

The Upper Dam has a drainage area equal to the surface area of the reservoir, about 55 acres, and has a total storage of about 4,350 acre-feet at elevation 1596 ft.

8.1.2 Flood of Record

There is no information on the flood of record since the reservoir's drainage area is its surface area.

8.1.3 Inflow Design Flood

The IDF is the PMF. Since the drainage area for the Upper Dam is the reservoir's surface area, the maximum inflow would be the probable maximum precipitation (PMP). The PMP was developed for the basin and found to be 34.24-inches within a 72-hour period with a maximum six hour amount of 22.38-inches (this is discussed further in the Lower Dam section).

It should be noted that if a precipitation event caused a significant increase in reservoir levels, the turbines could be operated to lower the reservoir.

8.1.4 EAP Dam Break Analysis

The inundation map for the Upper Reservoir in the EAP is based on a sunny day dam break analysis. For the Sunny Day dam break, releases from the Upper

Reservoir would flow to the Lower Reservoir and be contained there. Since there is minimal drainage area for the Upper Reservoir, only a Sunny Day dam break was performed. One 60-foot wide parapet wall was assumed to fail and initiate the breach. The rockfill was assumed to erode full depth. The peak outflow was estimated as **30,000cfs.**

The breach parameters used in the analysis are:

BR = Average width of breach = 160.0 feet. The bottom breach width is 60.0 feet and the top breach width is 260.0 feet.

Z = Horizontal component of side slope of breach = 1 (1H: 1V)

TFH = Time to fully formed breach = 3 hours

Breach depth=100 feet

The assumed breach width would encompass about four 60-foot-long panels at the crest. Figures 8.1 and 8.2 describe the breach parameters and assumed outflows.

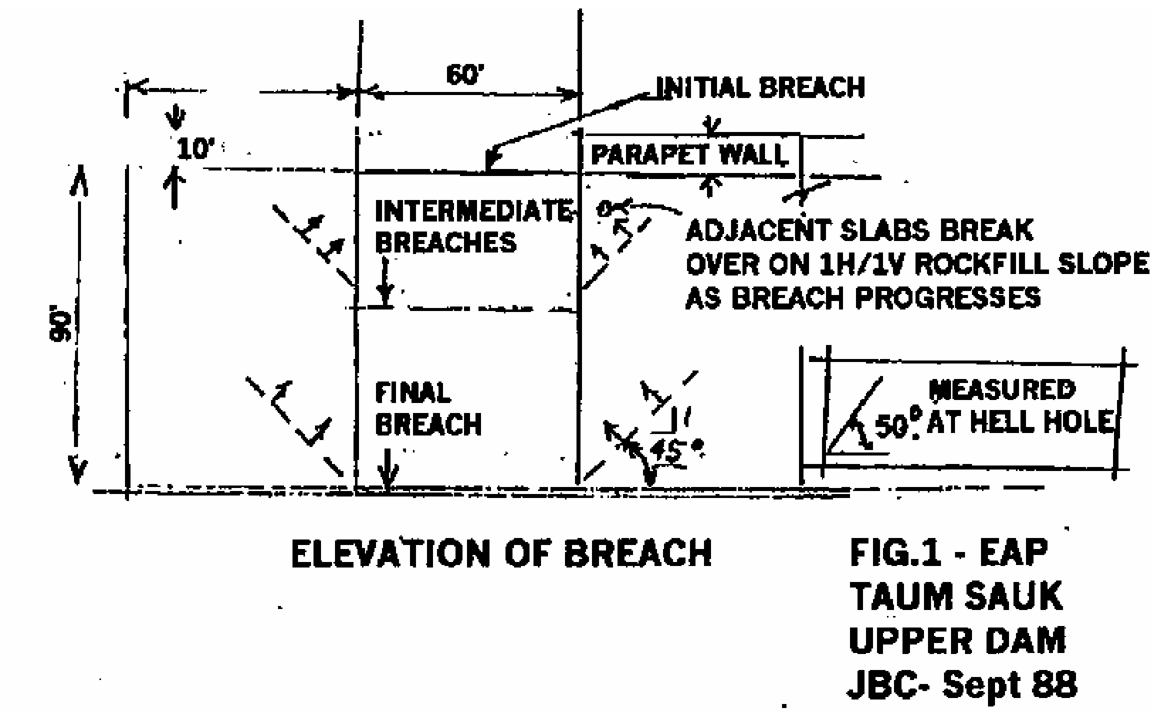

Figure 8.1- Breach parameters and breach formation for the upper dam.

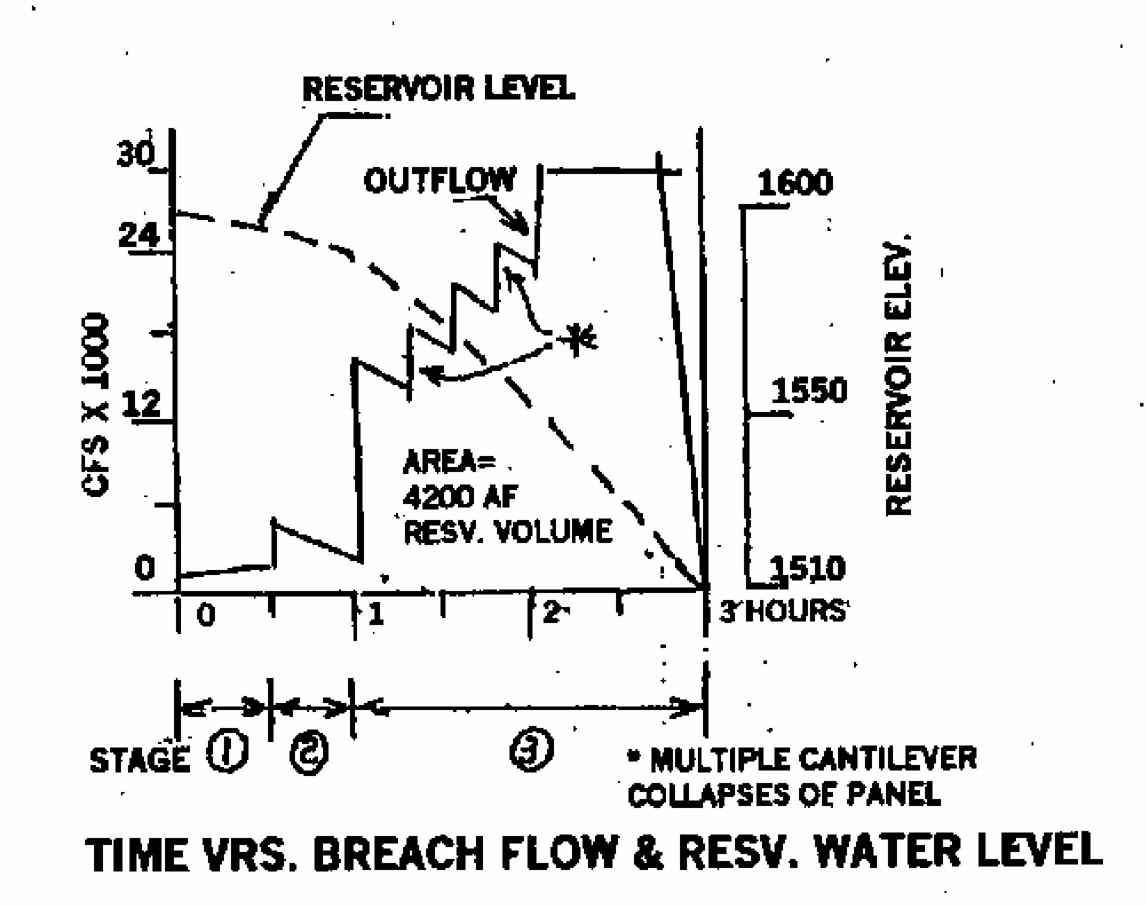

Figure 8.2 - Peak outflow and pool elevations as estimated in the dambreak analysis contained in the EAP.

8.1.5 Actual Dam Breach Parameters

The December 14, 2005 dam breach was significantly larger than the breach parameters assumed in the EAP. The actual breach parameters are as follows:

BR = Average width of breach =576 feet. According to the post breach aerial survey, the width of the breach at the crest is about 656 feet and the width of the breach at the elevation of the reservoir floor is about 496 feet. These are straight line distances between the ends of the breach and do not consider the curvature of the actual breached section. The actual breach included 12 parapet wall panels at the crest which is equivalent to about 720 feet.

Z = Horizontal component of side slope of breach = about 1:1. According to the post breach aerial survey, the weighted average of the left side slope (looking upstream into the reservoir) is 1V:1.06H and the right side slope is 1V:0.92H. The side slope is influenced by taking a straight line across the breach instead of going perpendicular to the contours. The side slope perpendicular to the contours is steeper.

TFH = Time to fully formed breach = 0.33 hour

Breach depth = 103 feet. This is based on the floor of the reservoir at elevation 1494 ft and the low point of the parapet wall at about elevation 1597 ft.

An elevation view of the breach based on the aerial survey is in Figure 8.3.

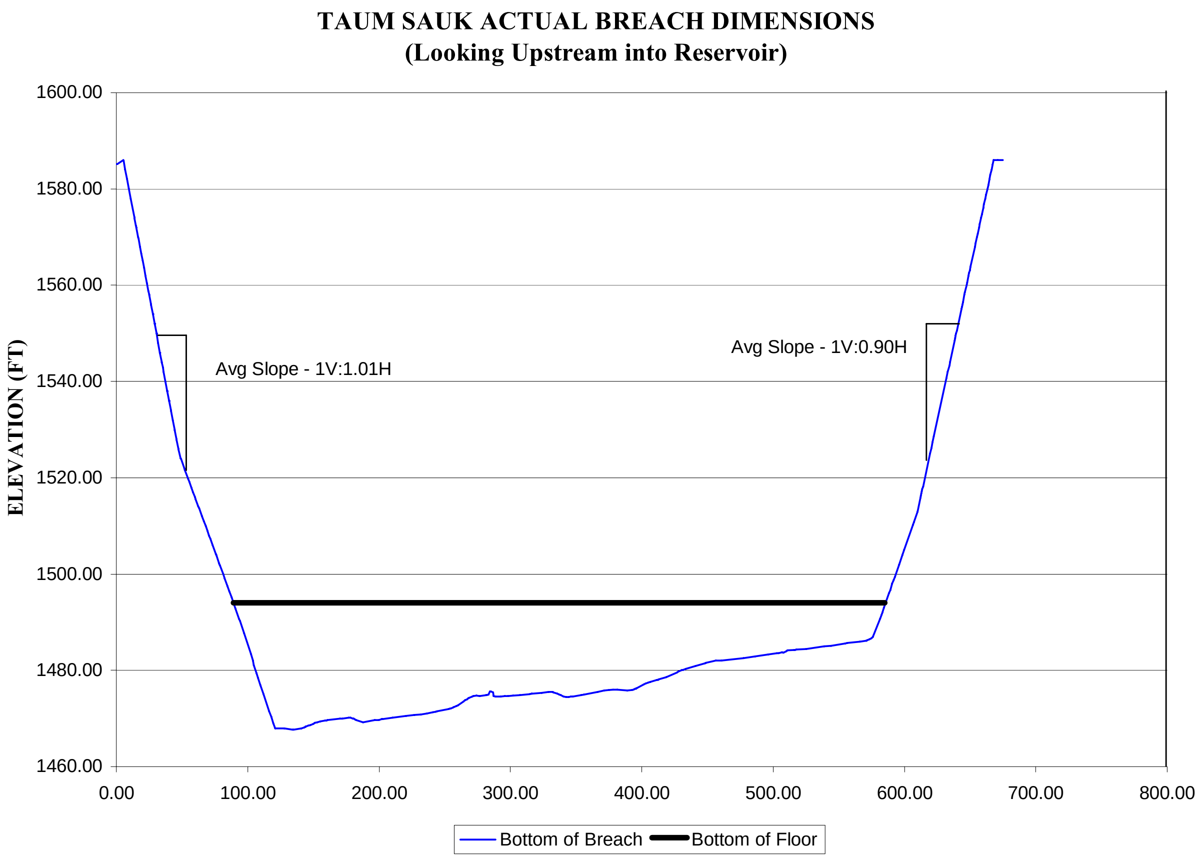

Figure 8.3– Breach Cross Section

8.1.6 Outflow hydrograph generation

The outflow hydrograph for the upper reservoir was calculated using the change in water surface height over time and the stage-storage curve for the reservoir. AmerenUE's December 27, 2005 filing included data showing reservoir levels vs.

time for the day of the event. The stage-storage curve was calculated in 1-foot increments using the post-breach aerial survey data (Figure 8.4) and a geographic information system (GIS). The stage-storage data was verified with the stage storage curve for the upper reservoir provided in AmerenUE's February 7, 2006 filing. As shown in Figure 8.5 the stage storage information from the two sources matches well.

Outflow was computed in one-minute intervals on December 14, 2005 from 5:15 am until the reservoir was mostly empty at 5:50 am. The change in stage for each one minute interval was interpolated on the stage-storage curve to a volume in acre-ft per minute, which was then converted to cfs. The outflow hydrograph is shown in Figure 8.6.

Because the reservoir level data was not reliable, with the reservoir approximately 4 feet above what the Druck pressure transducers were reading, a second curve was computed assuming a 4-foot under reading by the pressure transducers. The second curve should represent the upper limit of outflows due to the instrumentation movement.

Assuming that the pressure transducer readings were off by 4 feet, the calculated peak flow out of the breach was about 273,450 cfs. If the actual pressure transducer readings are used, the resulting peak outflow from the reservoir was about 269,000 cfs. Time to peak for the outflow hydrograph was approximately 8 minutes after the breach initiated.

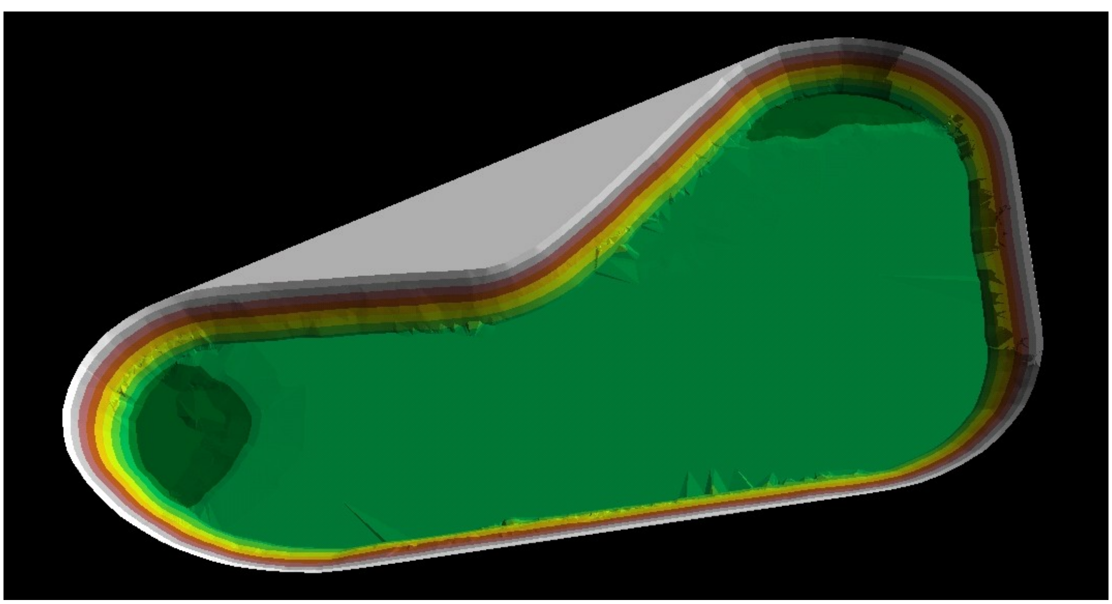

Figure 8.4 – Aerial Survey of empty upper reservoir

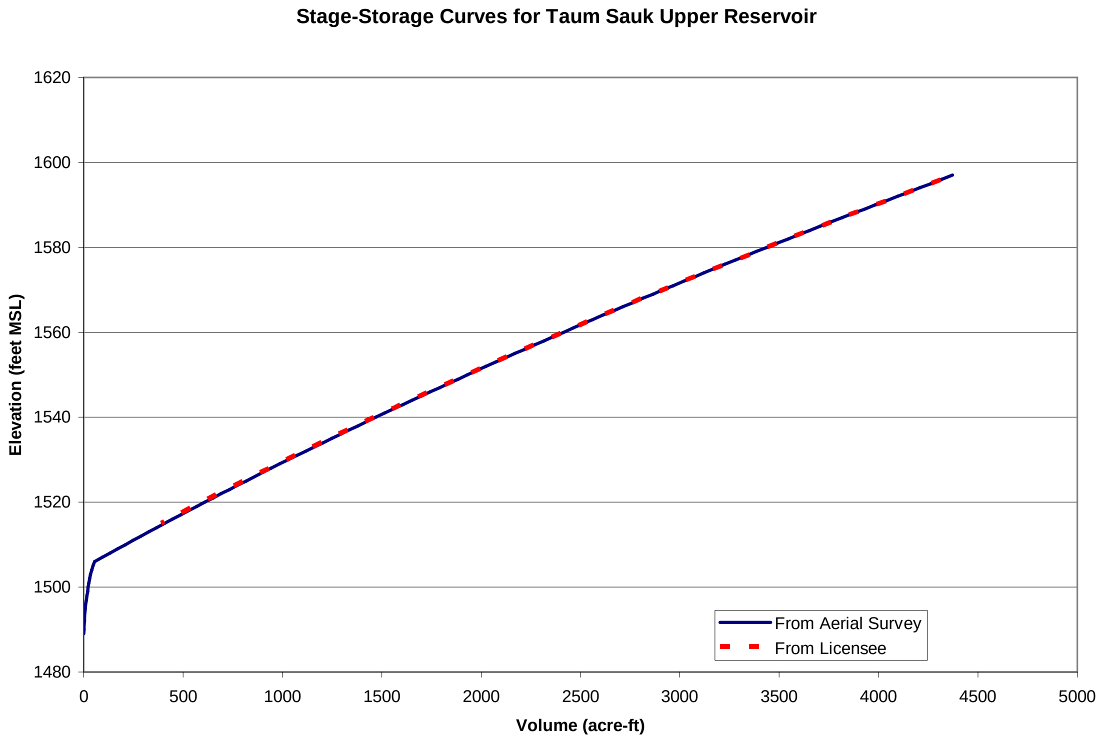

Figure 8.5 – Stage-Storage curves for Taum Saul Upper Reservoir

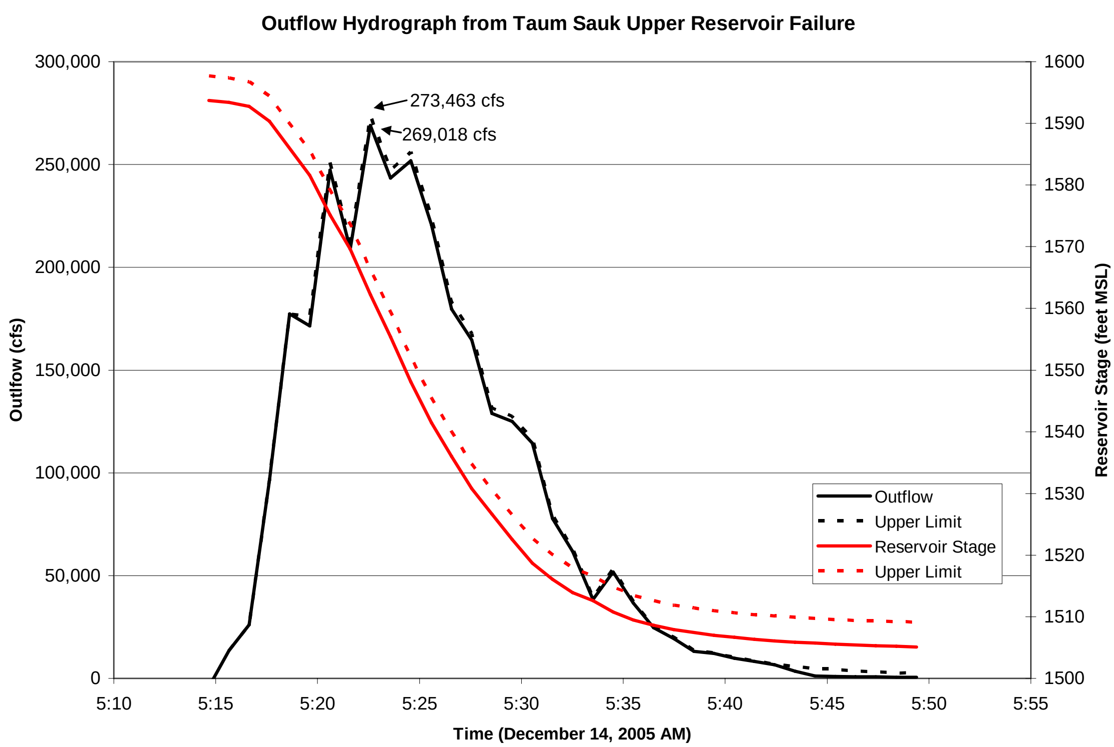

Figure 8.6 - Calculated Outflow Hydrograph for Taum Sauk Breach

The outflow hydrograph indicates initial flows may have been due to the loss of one and then two sections of the parapet wall. At 5:16 and 5:17 AM outflows were about 10,000 and 25,000 cfs, respectively. This corresponds with the outflow expected from the loss of one and then two sections of the parapet wall (see Section 8.1.9). After 5:17 AM flows increased rapidly peaking at 273,000 cfs at about 5:23 AM. This zig-zag shape of the outflow resembles somewhat the shape of the outflow estimated in the Emergency Action Plan (EAP), see Figure 8.2.

The calculated maximum outflow (273,000 cfs) is 9 times greater the maximum outflow assumed in the EAP dambreak (30,000 cfs). The major differences between the two events are the breach size and timing of event.

8.1.7 Wave Height Estimates

8.1.7.1 Wave Height Estimates for September 25

The overtopping on September 25 was described by eyewitnesses as occurring in waves near the Northwest corner of the Upper Reservoir (see February 8, 2006 interviews with Ronald Robbs, Chris Yordy, and Richard Cooper). AmerenUE's January 19, 2006 letter describes the affected panels were 90-96. On September 25, the remnants of Hurricane Rita were passing through the area. According to weather information from the NWS, wind data for the morning of September 25 at Farmington Airport is as follows:

> Max Wind Speed (Steady): 17 knots
> Max Wind Speed (Gust): 23 knots
> Wind Direction coming from 80-100 degrees from North

In Mr. Robb's interview, he indicated he heard on the Evening News that winds at Farmington Airport peaked at 38 miles per hour (33 knots).

FERC staff interviewed representatives from the National Weather Service (NWS) in St. Charles, Mo. on January 12, 2006. NWS personnel stated that there can be large variances in wind speed between the elevations of Farmington Airport and the Upper Reservoir, but they expect this would occur on clear days. They said it is not likely there would be drastically different wind speeds between the elevation of Farmington Airport and the Upper Reservoir on a rainy, cloudy day which was the case on September 25. They said one reason there could be a large difference in wind speeds between the two locations on September 25 is if there was an isolated thunderstorm on the mountain. According to the National Climatic Data Center website (www.ncdc.noaa.gov), there were no reported thunderstorms or high wind events from September 24 through October 1, 2005 in Reynolds County, MO. Mr. Robbs' and Mr. Yordy's interviews also did not indicate a thunderstorm was occurring when they witnessed the overtopping.

The table below shows a range of possible wave heights for the September 25, 2005 event using the USGS wave height formulae from the *Shore Protection Manual*, Coastal Engineering Research Center, U.S. Army Corps of Engineering Waterways Experiment Station (1984). In addition to wind speed (meters/second), other parameters needed are the fetch (kilometers) and depth (meters) of reservoir. Winds coming out of the East/Southeast would be almost perpendicular to Panels 90-96 and result in a maximum fetch of about 0.35 km.

Wave Calculations – September 25, 2005

Wind Velocity (knots)	Wind Velocity (m/s)	Fetch (km)	Reservoir Depth (m)	Wave Ht (m)	Wave Ht (ft)
17	8.74	.35	31	.10	.33
23	11.83	.35	31	.14	.46
33	16.9	.35	31	.22	.72
40	20.57	.35	31	.28	.92

8.1.7.2 Wave Height Estimates for September 27, 2005

According to the February 8, 2006 interview with Mr. Richard Cooper, he saw wet spots on the downstream side of the parapet wall, at the low point of the wall, during a morning visit to the upper reservoir. Mr. Cooper did not witness waves exceeding the top of wall. The wet spots were possibly due to spray from waves over the wall as opposed to waves exceeding the top of the wall. Panel 72 is the low point of the parapet wall. According to generation information the reservoir was filled to elevation 1596 this morning.

The weather information for the morning of September 27 indicated early morning fog leading to mostly to partly sunny conditions. During the morning there were maximum steady winds of 3-5 knots. The wind direction changed during the morning. Winds came from 10-40 degrees from North at around 8:00-9:00 am then from 110-140 degrees from North after 10:00 am.

According to the interview with NWS staff on January 12, 2006, it is more likely to have higher winds at the Upper Reservoir compared to the Farmington Airport on clear days than rainy days. Since September 27 was mostly to partly sunny, it is possible the wind speeds were higher than 5 knots at the upper reservoir. According to a September 27, 2005 email from Richard Cooper to Thomas Pierie and Chris Hawkens of AmerenUE (included in the January 27, 2006 AmerenUE submittal), he did not see any waves at the Upper Reservoir on September 27.

The table below shows a range of possible wave heights for September 27, 2005 using the USGS wave height formulae from the *Shore Protection Manual*. Winds coming out of the Northeast would result in a maximum fetch of about 0.5 km at Panel 72. Before 10:00 am on this morning, the wind direction was almost parallel to the alignment of panel 72

Wave Calculations – September 27, 2005

Wind Velocity (knots)	Wind Velocity (m/s)	Fetch (km)	Reservoir Depth (m)	Wave Ht (m)	Wave Ht (ft)
5	2.57	.45	31	.024	.08
10	5.14	.45	31	.06	.20
15	7.72	.45	31	.10	.33

8.1.7.3 Wave Height Estimates for December 14, 2005

The weather information for the early morning of December 14 indicated light snow, rain, and drizzle with temperatures in the mid-30s. At Farmington Regional Airport about 0.08 inch of precipitation occurred during the early morning. The recorded steady wind speeds ranged from 10-14 knots with gusts to 22 knots. Winds originated from 140-180 degrees from North.

According to the interview with NWS staff on January 12, 2006, it is more likely to have higher winds at the Upper Reservoir compared to the Farmington Airport on clear days than rainy days. The morning of December 14 was rainy, so we would not expect wind speeds to be drastically different between Farmington and the Upper Reservoir.

Winds coming out of the South-Southeast would be almost perpendicular to the areas near panels 72 and 95-100 and result in a maximum fetch at the breach area of about 0.5 km.

Wave Calculations – December 14, 2005

Wind Velocity (knots)	Wind Velocity (m/s)	Fetch (km)	Reservoir Depth (m)	Wave Ht (m)	Wave Ht (ft)
14	7.20	.5	31	.092	.30
22	11.32	.5	31	.16	.52
30	15.43	.5	31	.23	.95

8.1.8 Velocity of Flows over Parapet Walls

Prior to the Upper Reservoir breach, flows overtopped the parapet wall and began eroding the downstream slope of the embankment. The velocity of the overtopping flows falling 10 feet from the top of the parapet wall to the embankment crest would be approximately:

$$V = (2 \cdot g \cdot h)^{0.5} = 25.4[ft/s]$$

where g is the gravitational constant and h is the height of falling water.

8.1.9 Estimated Outflow for Loss of One Section of Parapet Wall

The broad crested weir equation (below) was used to estimate the outflow that would result from the collapse of a single panel of the parapet wall.

$$Q = 0.385 \cdot C \cdot L\sqrt{2g}H^{3/2}$$

where Q is the discharge in cfs, C is the assumed weir coefficient, L is the length of a rectangular weir, g is the gravitational constant, and H is the height of water over the weir. For the loss of one parapet wall the weir length would be 60 feet, and the height of the weir would be about 10 feet, at the instant of loss. Varying the weir coefficient from 0.85 – 1.05, the discharge resulting from the loss of one parapet wall section would be between 4,980-6,160 cfs.

We note the heel of the parapet wall extends about 3.5 feet below crest of the embankment. Including this distance to the height of the wall would increase the range of flows to about 7,800-9,650 cfs.

8.2 Lower Dam

The 390-foot-long Lower Dam is an ungated overflow spillway except for two piers, 13- and 4-foot-wide that support the operating deck. The spillway crest is at elevation 750 feet. The spillway discharges to a concrete flip bucket with a 28-foot-radius.

The lower dam also has two sluices: the small sluice is a 16-inch-diameter spiral welded pipe with an upstream invert at elevation 710 feet and downstream invert at elevation 707 feet. A 20-inch cast iron slide gate on the upstream face of the dam controls flow through the small sluice. The slide gate motor operator is located on the top of the 4-foot-wide pier on the crest of the dam. An intake structure extends 7 feet upstream of the dam and provides a single set of slots for either a trashrack or stoplogs. The large sluice is a horizontal 8-foot-wide by 10-foot-high steel-lined conduit with an invert elevation of 705 feet. An 8-foot by 10-foot cast iron slide gate located on the upstream face of the dam controls flow through the sluice. The slide gate motor operator is located atop the 13-foot wide pier on the spillway crest.

8.2.1 Drainage Area/Surface Area/Storage

The lower dam has a drainage area of about 88 square miles. The surface area at the ogee crest is about 390 acres. According to the stage storage curve provided in AmerenUE's February 7, 2006 filing, the total volume of the reservoir is approximately 4,360 acre-feet at elevation 750ft and 424 acre-feet at elevation 736 ft.

8.2.2 Spillway Curve

The Rating curve of the ogee spillway is shown in Figure 8.7.

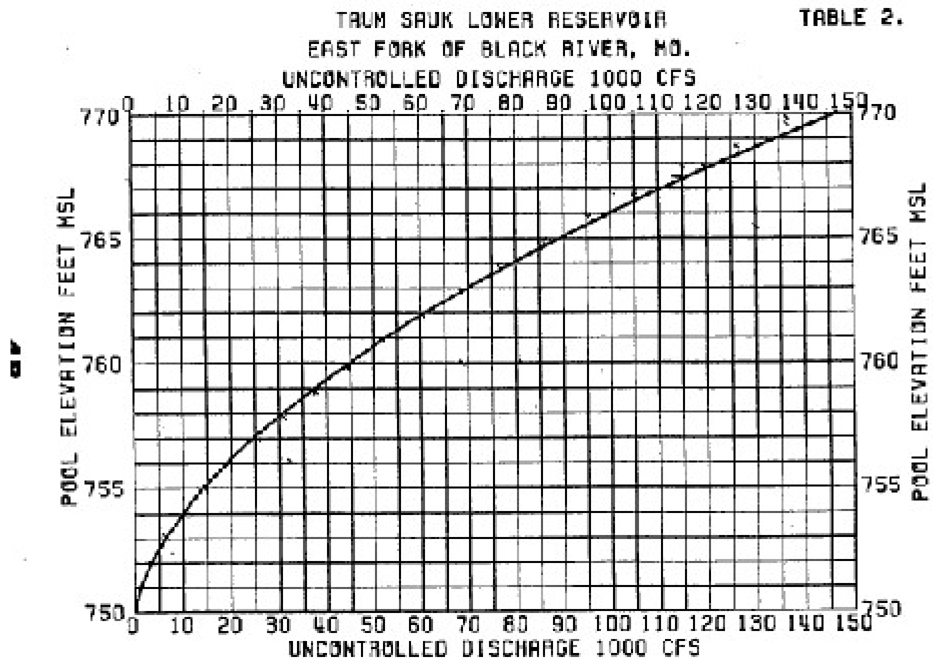

Figure 8.7- Lower Dam ogee rating curve.

8.2.3 Flood of Record

The maximum flow recorded by the USGS gaging station above the Lesterville Bridge was 35,800 cfs and occurred on November 19, 1985. Overflow depth at the Lower Dam was recorded at that time as 7 feet. From the Spillway Discharge Curve, the discharge at the dam was approximately 25,000 cfs. Adding about 2,500 cfs being released through the sluice gates gives a total flow of about 27,500-cfs.

On November 14, 1993, the depth of flow over the spillway reached about 7.5 ft or about 28,000 cfs. The sluices passed about another about 3,000 cfs, for a total flood of approximately 31,000 cfs. The Lesterville gage was no longer in service in 1993.

8.2.4 Inflow Design Flood

AmerenUE (1986) developed the Probable Maximum Flood (PMF) using the National Oceanic and Atmospheric Administration (NOAA) Hydrometeorlogical Reports (HMR) No. 52. The PMP for the basin was found to be 34.24-inches within a 72-hour period with a maximum six hour amount of 22.38-inches. The PMF was estimated to have a 2-hour crest of 120,464 cfs and produce peak stage of 767.09 feet or 17.09 feet above the spillway crest. The PMF hydrograph is shown on Figure 8.8. Considering that the significant depth of overtopping, an IDF less that the PMF may be justified. However, these studies have not been done and for the present, the IDF is assumed to be the PMF.

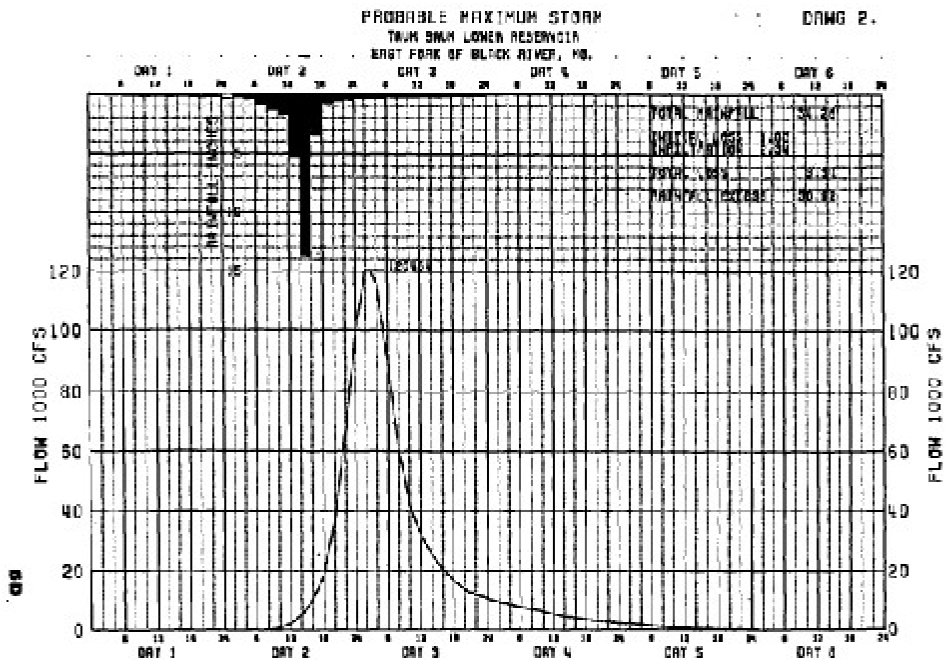

Figure 8.8 - The PMF hydrograph.

8.2.5 Freeboard Adequacy

Normal maximum water level for the Lower Dam is elevation 749.5 feet or 0.5 feet below the spillway crest. During floods, the entire dam overtops and freeboard is not a concern since the dam is also a spillway and the abutments are competent rock.

8.2.6 EAP Dam Break Analysis

A sunny day dam break analysis and associated inundation map for the Lower Reservoir are included in the EAP. The dam was assumed to fail quickly and the breach was assumed to be 3-blocks wide to the gallery elevation. The peak outflow from the Lower Reservoir was estimated to be about 51,000 cfs in 30 seconds.

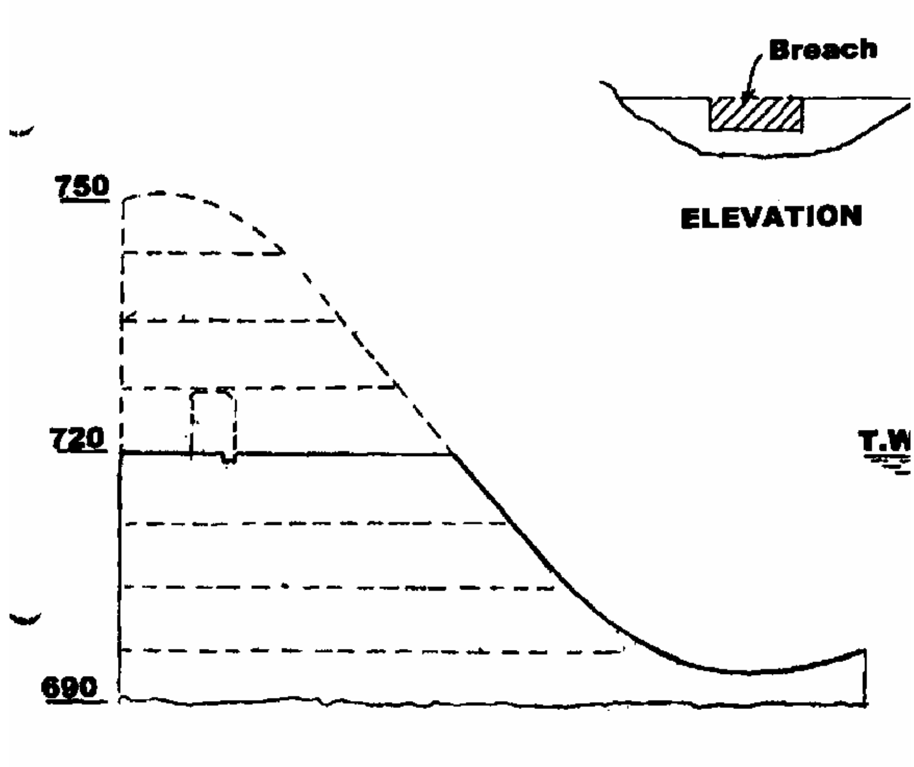

Figure 8.9 - Lower Dam Breach location.

The breach parameters used in the analysis shown in Figure 8.9 and described below:

BR (average width of breach) = 3*40 = 120 feet.

Z (side slope of breach) = 0 (vertical slopes)

TFH (time to breach) = less than 0.1 hour

Breach depth = 30 feet

8.2.7 Maximum Lower Reservoir Level Following Upper Reservoir Breach

Figure 8.10 shows water levels at the Lower Reservoir on December 13 and 14, 2005. The Lower Reservoir was able to store the majority of inflows from the Upper Reservoir breach. According to the reservoir level information provided by AmerenUE's letter dated December 27, 2006, the lower reservoir was drawn down to elevation 736.1 ft prior to the breach. This provided about 3,920 acre-feet of storage up to elevation 750 ft. The maximum recorded elevation in the lower reservoir following the breach was 751.1 ft, which occurred at about 8:00 am. This was approximately 1.1 feet of overtopping and resulted in a maximum outflow from the spillway of about 1,600 cfs (excluding flows through the sluice). The maximum reservoir level on December 14, 2005 was well below the flood of record.

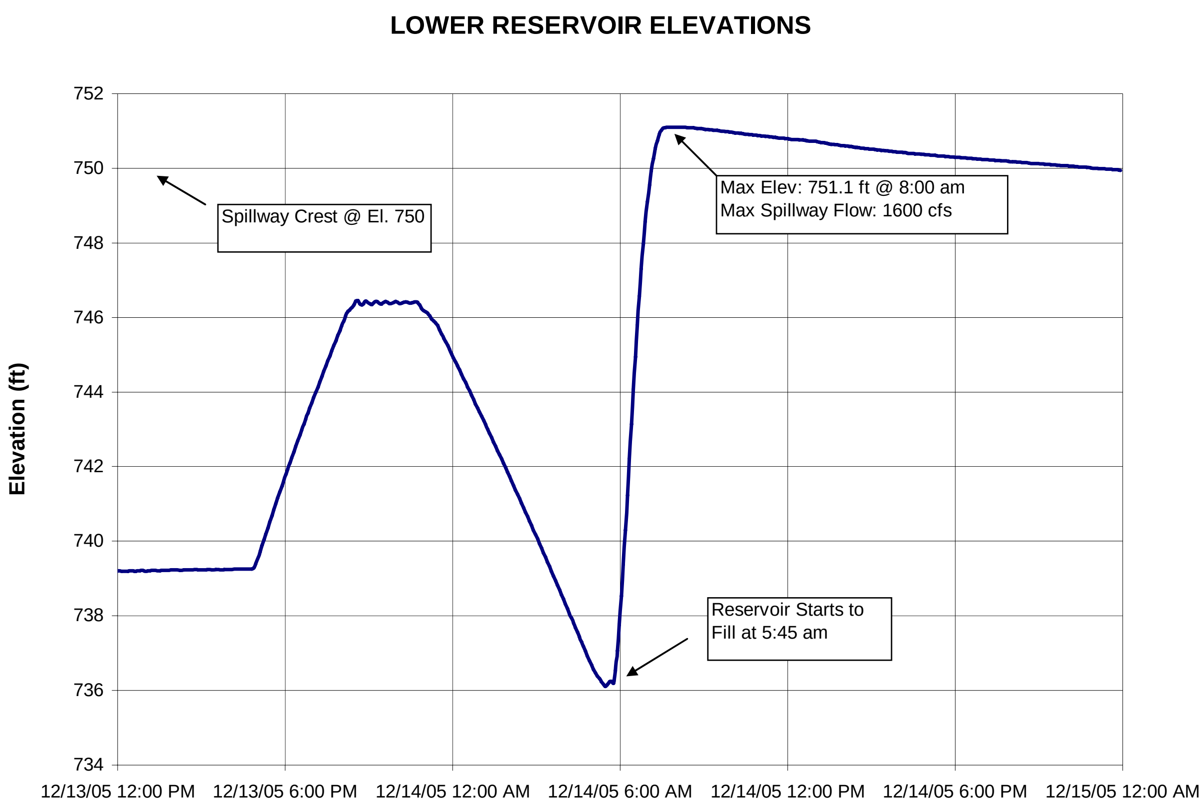

Figure 8.10 - Lower Reservoir Elevation December 13 and 14 2005

Section 9 Stability Analysis

9.1 Utexas4 Embankment Stability Analysis

FERC staff conducted forensic stability analyses in March 2006. Embankment and foundation parameters were determined from observations of the soils and bedrock in the breach area. A range of material shear strengths and piezometric levels were selected to evaluate embankment stability. A cross section was developed that passes through the center of the breach area based on the topography of the original embankment, original design drawings, and the aerial topography. The computer program Utexas4 was used in the analyses.

9.1.1 Reconstruction of the Embankment Section

The original project stationing was reconstructed using Sheets 8304-x-26052 and 8304-X-26117 of the as-built drawings (Disk 1 of the 9-CDs submitted February 7, 2006) with Sheet 1 of 1 of the SURDEX aerial topographic survey known as Exhibit 6. The center of the breach area occurred at approximately Station No. 21 + 69.81, which corresponds to the intersection of the access road and the dam crest on the northwest side of the dam. Using this information, the cross section of the dam was reconstructed and the access road was redrawn in its approximate position.

9.1.2 Original downstream slope angle

Questions were raised about the steepness of the downstream slope in the area of the breach. A second topographic section was made at the north end of the breach to assess the steepness of the slope in that area. Due to slope failures immediately adjacent to the breach, the section was taken 80 feet northeast of the breach edge (refer to Line 2 in Appendix D – Figure D.1). Figure 9.1 shows the cross section which represents the as-built configuration of the breach section.