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.

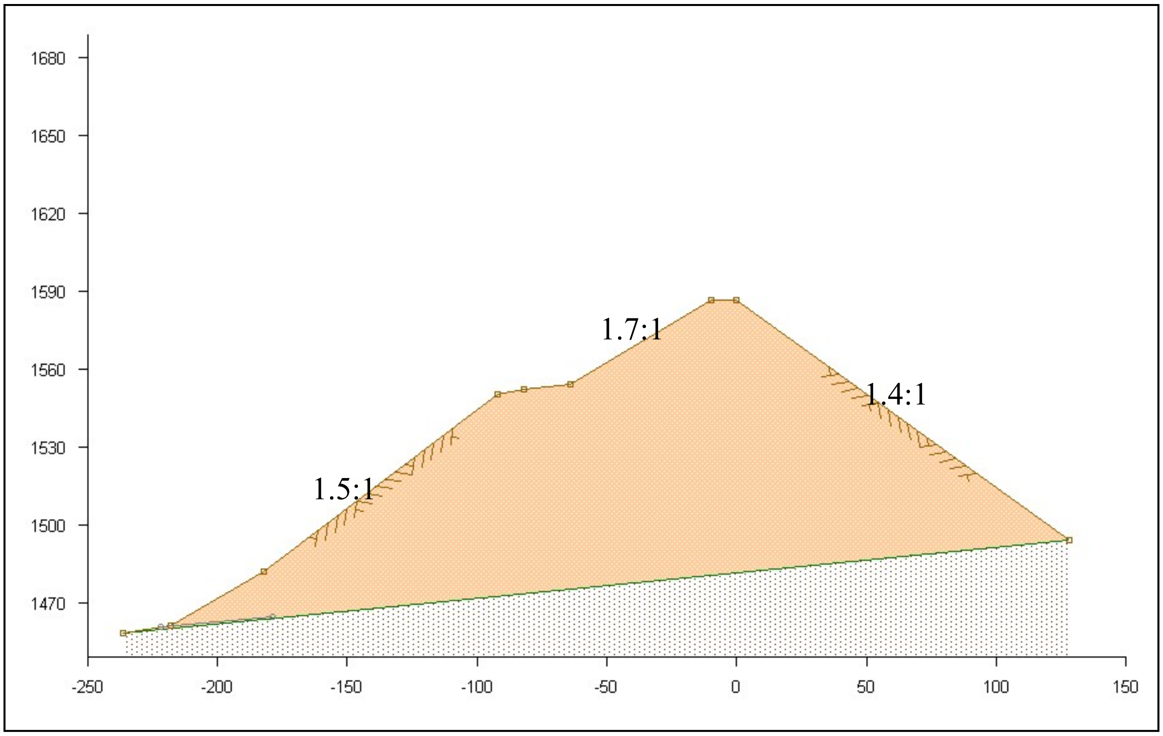

Figure 9.1– Embankment Cross Section

The upstream slope (right side of above drawing) is 1.4H:1.0V. The portion of the downstream slope that is above the access road is 1.7H:1.0V, and the portion of the downstream slope below the access road is 1.5H:1.0V. The downstream slope used in the stability analyses for the section taken at the center of the breach was 1.5H:1.0V. Rather than cutting into the original embankment to construct the access road, it appears the access road was placed with material dumped on top of the original embankment. The slope of the road fill material is slightly steeper than the original downstream embankment slope (1.5H:1.0V versus 1.7H:1.0V, respectively). The steeper slope of 1.5H:1.0V was used to represent of the downstream slope in the area of the breach.

9.1.3 Vertical Curve in Crest of Dam

Due to differential settlement, a vertical curve or sag, developed in the crest of the dam in the area of the breach (see Appendix D – Figure D.2). The lowest point in the curve is around the survey pin near Panel 95, with a crest elevation of 1587.39 (based on a 2004 survey). The crest elevation increased towards the north and south of Panel 95, up to elevation 1588.33 at Panel 85 and elevation 1587.70 at Panel 100. Based on our estimate of the maximum pool elevation during overtopping, overtopping flows occurred from Panel 88 through Panel 100, which roughly corresponds to the breach area. Initial overtopping flows in this area would tend to flow along the length of the crest towards the lowest point at Panel

94 and down the access road at Panel 95. Concentrated flows such as this may be expected to increase the erosive forces at the break point where the access road meets the crest of the dam, which corresponds to the center of the breach area. The stability analyses performed here do not take into account potential erosion, which may have been an important factor leading to undermining of the parapet in the area of Panels 94 and 95.

9.1.4 Foundation Geology and Rockfill Zonation

Paul C. Rizzo Consultants prepared a geologic map of the foundation area, which was used to evaluate the engineering behavior of the various materials present. The bedrock is a jointed rhyolite that is considered competent rock. No singular, continuous planes of weakness were observed within the bedrock that could be modeled as a failure plane. However, there is an area in the south side of the breach, that trends along the centerline of the low pond in the northwest corner of the reservoir floor, which contains weathered rhyolite (Figure 9.2). The material is slightly cohesive in some areas and granular in others and still contains some of the original rock fabric. Along the north side of the breach, near the road that ran along the toe of the dam, there is an area with about 6 to18-inches of a clay rich soil with roots and oganics, resting directly on fresh rhyolite (Figure 9.3). Most of the area of the breach is fresh competent rock with no traces of soil or weathered rock (Figure 9.4).

There is no evidence to support a conclusion that the weathered rock or clay layer extended beneath the entire footprint of the breach area. However, it was assumed in the stability analysis that there was a layer of weak material resting on top of bedrock throughout the area that was stripped by the discharge through the breach. Both the clay layer and weathered bedrock were treated as having the same shear strength. This assumed continuous layer of weak material results in more conservative (lower) factors of safety than would have existed if the rock fill had been placed directly on top of bedrock. This should give a lower-bound estimate for stability of the rock fill in the breach area. The foundation was divided into two components; 1) sound rock and, 2) weathered rock/topsoil.

Figure 9.2 - Area of weathered rhyolite, which is in-line with the "fish pond" depression in the reservoir (background).

Figure 9.3 - Residual topsoil on top of fresh rhyolite.

Figure 9.4 - Fresh rhyolite bedrock surface.

Based on descriptions of the construction of the dam, material that did not meet the specifications for clean rock fill was used to construct access roads. There are two access roads in the area of the breach; one at the toe of the dam and another that ran up the side of the dam. Although the upper portion of the rockfill was a compacted rockfill, no attempt was made to differentiate between the dumped and rolled sections of the rock fill. The dam was, therefore, divided into three sections; the rockfill section, the lower access road, and the upper access road (see Appendix D - Figure D.3). Both access road fill sections were assumed to have similar shear strengths, but lower than the rock fill.

A ten to sixteen-inch-thick reinforced concrete facing is present on the upstream side of the rock fill. This was also included in the analyses.

9.1.5 Shear Strengths

Stable slopes of 0.97H:1.0V on the south side and 0.98H:1.0V on the north side of remained after breach of the dam (averaged from top to bottom of breach). These slopes had remained stable for three months at the time this report was written. Slopes with this angle equate to a shear strength of φ=45.9º. However, there is a definite break in slope in the breach sides, with much steeper slopes near the top half of the rock fill (Appendix D – Figure D.1). The steepest portion of the breach slopes are 0.65H:1.0V, or φ=57.0º, which may represent better compaction near the crest of the dam. The lower portion of the breach slope is 1.2H:1.0V, which equates to φ=39.8º, which may represent the dumped rock fill. Hence, the phi angle of the rock fill is estimated to be between 40º to 57º.

9.1.6 Stability Analysis

The stability analyses were done using three ranges of shear strengths for the various materials present in the breach area. These trials are summarized below:

Material/Trial No.	1	2	3
Bedrock	φ'=45, c'=2000 psf	φ'=45, c'=2000 psf	φ'=45, c'=2000psf
Weathered Rock/Clay	φ'=15, c'=0	φ'=23, c'=0	φ'=30, c'=0
Road Fill	φ'=36, c'=0	φ'=40, c'=0	φ'=42, c'=0
Rock Fill	φ'=39, c'=0	φ'=43, c'=0	φ'=45, c'=0
Reinforced Concrete	φ'=0, c'=2000 psf	φ'=0, c'=2000 psf	φ'=0, c'=2000 psf

An infinite slope analysis was also conducted to compute a factor of safety for the saturated downstream slope. The lowest factor of safety computed using this method is 0.54. As a comparison, factors of safety computed using the UTEXAS4 - Spencer solution method were in the range of 0.30 to 0.33. The Spencer method computes the factor of safety based on simultaneous solution of mobilized shear strength along the base (for the given factor of safety) and the computed side force inclination required for force-moment balance and therefore will yield a slightly different value for the factor of safety than that computed by the infinite slope method. Comparing the computer analysis and the infinite slope analysis, while the exact results (0.54 and 0.30) do not appear complementary, both methods yield a factor of safety significantly below 1.0 indicating that the embankment was indeed susceptible to failure from overtopping saturation

Please note that extra conservatism is added by neglecting cohesion for the weathered rock/clay layer, although there is expected to be some cohesion present in these materials. In addition, cracks through the concrete facing were assumed in the analyses.

9.1.7 Phreatic Levels

Four phreatic levels were assumed for each trial of shear strengths. A summary of the different levels of phreatic levels assumed for the analyses are shown below:

Phreatic Conditions Assumed

a	b	c	d
Lower 1/3 of dam saturated	Entire base of dam saturated to upstream toe	Entire base of dam saturated up to middle upstream face	Condition b plus upper portion of downstream slope saturated

9.1.8 Other Trials

Trial 4 was done to evaluate the stability of the shallow failure of the downstream slope without water saturation.

Trial 5 was done to evaluate the stability of the toe of the dam if it were saturated by the overtopping water. The phreatic level assumed in this analysis assumes the geomembrane liner is effective, but that a water saturation front extends from the center of the downstream slope.

Trial 6 was done to evaluate the post-shallow failure stability of the remaining portion of the dam. This involved evaluating wedge failure along the weakest foundation zone with a moderate phreatic level (phreatic level b).

9.1.9 Results

Non-circular (wedge) slope stability analysis was evaluated using the non-circular search method of Utexas4 and the Spencer method of solution. Graphical results for each trial run are included in the Appendix and the factors of safety are summarized below:

Phreatic Condition/ Trial No.	1	2	3	4 shallow wedge	5 toe wedge	6 post-slide stability
a (deep wedge)	1.15	1.83	2.51	-	-	-
b (deep wedge)	1.10	1.73	2.10	-	-	1.24
c (deep wedge)	0.84	1.31	1.75	-	-	-
D (shallow slope wedge)	0.30	0.35	0.38	-	-	-
no overtopping	-	-	-	1.24	-	-
overtopping	-	-	-	-	0.75	-

These results only consider static stability and do not take into consideration the exacerbating affects of potential rapid erosion from overtopping flows.

The results indicate the following:

1. The downstream toe is likely to fail as the phreatic surface rises at the toe. The outer layers of the downstream slope are likely fail as overtopping flows saturate these layers. Progressive failures would likely occur with continued overtopping.
2. At the lowest shear strength assumed in the analysis for the weathered bedrock/clay layer (phi = 15 degrees, no cohesion) combined with a phreatic surface located at the mid-height of the embankment indicates massive embankment failure could occur.
3. Analyses using higher strength parameters (> phi = 20 degrees) in the weathered rock/clay layer indicate the embankment is likely to be stable even with the phreatic surface located at the mid-height of the embankment.

9.2 FLAC Analysis of Parapet Wall Considering Erosion of Downstream Face

FERC staff performed an analysis of the parapet wall and embankment considering downstream erosion from overtopping using the FLAC model. Erosion of the downstream slope was simulated by allowing the FLAC model to come to equilibrium, removing a 1-foot-thick slice of the downstream face at an angle slightly less than the friction angle, and then re-iterating. The analysis assumed a friction angle of the embankment material of 42 degrees and no phreatic surface under the wall.

The analysis was stopped when the top of the parapet wall deflected more than 1 foot, which occurred when embankment erosion approached the toe of the wall. This deflection would cause significantly more overtopping to occur, further undermining the wall. Also, it is possible the geomembrane and concrete liner would have ruptured due to the significant wall movement allowing substantial leakage through the open joint, accelerating the loss of embankment material beneath the wall. (See Figures 9.5 and 9.6)

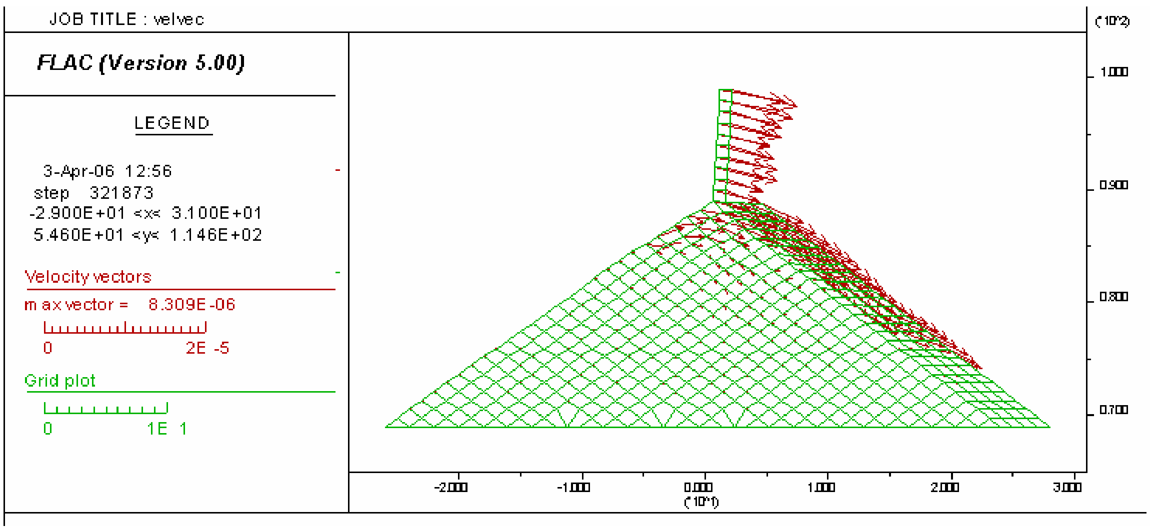

Figure 9.5 – FLAC Analysis

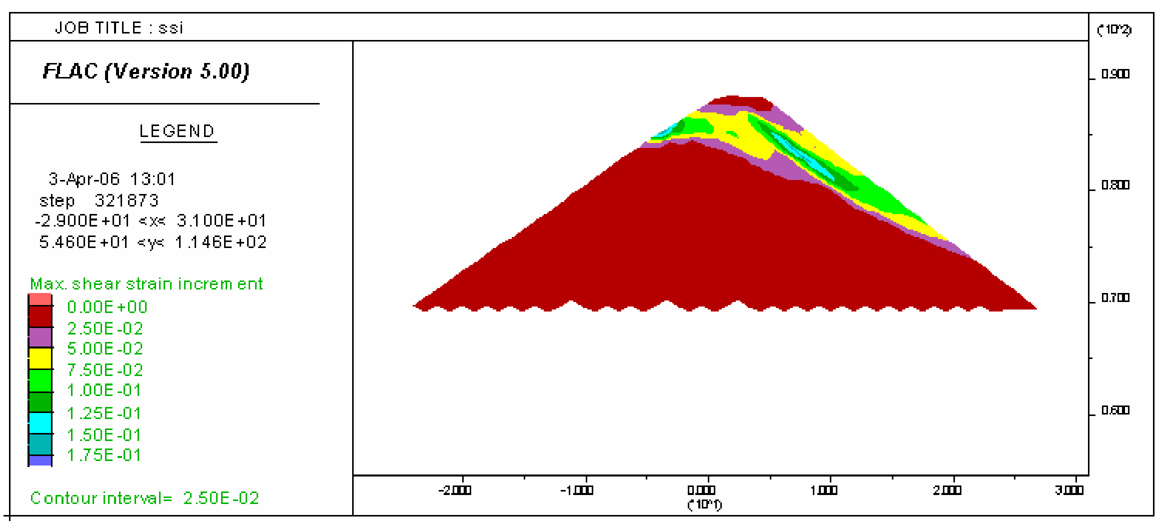

Figure 9.6 - Shear strain from the final FLAC iteration.

Note the band of high shear strain parallel to the slope