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RE: Draft White Paper on TVA's Kingston Ash Failure

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BACKGROUND

TVA reports that Dike C of its Kingston Ash Facility was built in 1958 in a swampy area called Swan Pond of Watts Bar Lake adjacent to the Emory River. The cell created by Dike C filled in 1965 and TVA then modified Dike C to provide additional ash disposal capacity. Dike C was reportedly constructed using compacted earthen fill and was raised twice over sluiced ash. The first raising was done using compacted earthen fill and the second raising consisted of compacted ash.

In 1984, construction of dredge cells began on the sluiced ash approximately 200 feet upstream of Dike C when the sluiced ash was at elevation 768 feet. The outslopes of the dredge cells were constructed in stages using compacted ash. Sluiced ash was then deposited within cells created by the multi-sided outslopes.

The northeastern outslope of the dredge cells, located directly upstream of Dike C, was founded entirely on sluiced ash (i.e. splitter dike). The other outslopes of the dredge cells were founded partially on previous embankments or they abut other dredge cells. Based on the information I reviewed, no internal drainage provisions were included in the construction of Dike C, whereas numerous internal drains were installed at various levels within the northeastern outslope of the dredge cells.

On 22 December 2008, the Kingston Ash Facility failed when the level of the sluiced ash in the dredge cells achieved elevation 816 feet. TVA published historical data for the facility on its web site at http://www.tva.gov/kingston/tdec/index.htm, including aerial photographs of the site before failure at http://www.tva.gov/kingston/photo_gallery/KIF_webMaps_PhotosOnly_8-5x11_NAIP_2008.jpg and after failure at http://www.tva.gov/kingston/photo_gallery/KIF_webMaps_PhotosOnly_8-5x11_12_30_2008.jpg. Reports prepared in 1994, summarizing data from subsurface exploration and engineering calculations, are included at http://www.tva.gov/kingston/tdec/pdf/TVA-00001359.pdf and http://www.tva.gov/kingston/tdec/pdf/TVA-00013034.pdf, respectively.

A 2009 inspection report of the facility is provided at http://www.tva.gov/kingston/tdec/pdf/TVA-00013800.pdf. The report is based on data compiled during an inspection in October 2008, prior to the failure. Page 7 of that report references “old dewatering wells.” According to the report, “At the time of the inspection, the valves on some drains were closed in preparation for measurement of water levels and those wells showed water pressure above the dike slope. The valves of the other monitoring wells were open and were flowing clear water to the drainage ditch.” Figure 23 on page 14 of the 2009 Kingston inspection report is labeled, “Dewatering well overflowing redwater.”

Results of analysis were published on the TVA web site at http://www.tva.gov/kingston/tdec/pdf/TVA-00003149.pdf to identify the cause of a reported “blowout” that occurred in 2003 on the northwestern outslope of the dredge cells adjacent to Swan Pond Road. The installation of shallow drains was recommended to mitigate problems associated with reported “excessive seepage and piping.” Another “blowout” was reported in 2006 on the northwestern outslope of the dredge cells, but little information was found regarding subsequent remedial measures other than the reference to dewatering wells in the 2009 inspection report.

I understand that the forensic team retained by TVA analyzed the Kingston failure based on undrained conditions. Field and laboratory data were used to justify undrained shear strength generally between 600 psf and 800 psf for the ash and mixed ash/sediment zone beneath the ash. Undrained shear strength was also used for cohesive soil in the analysis of Dike C.

No discernable decrease in void ratio was reported with increasing depth during sampling and testing of failed and unfailed sluiced ash deposits. Furthermore, a thin layer of ash/sediment with a very low undrained strength and a shear plane were reportedly found at the base of the sluiced ash deposit. Finally, photographs of “liquefied ash” obtained after the failure reportedly validate the use of undrained strength based on total stress conditions in the forensic study.

Figure 1 shows the results of a stability analysis that back-calculates average undrained shear strength = 700 psf for the sluiced ash and ash/sediment layer as being required to produce a factor of safety less than 1.0. Figure 2 illustrates that a zone at the base of the sluiced ash with undrained shear strength = 600 psf could produce failure presuming the overlying sluiced ash has undrained shear strength = 725 psf.

CURRENT PRUDENT ENGINEERING PRACTICE:
EFFECTIVE STRESS VS. TOTAL STRESS ANALYSIS

One advantage of using an undrained analysis is its simplicity. A sample is tested while drainage is impeded, allowing pore water pressure to develop, and the resulting shear strength is measured. Because shear strength is measured as a result of the combined effects of loading, pore water pressure development, and shearing under undrained conditions, the method is also known as a total stress analysis.

The primary disadvantage of a total stress analysis is that pore water pressure in the sample during testing may differ from actual conditions experienced during and after construction. According to Bishop and Bjerrum in their landmark 1960 treatise, *The Relevance of the Triaxial Test to the Solution of Stability Problems*, “To work directly with undrained test results expressed in terms of total stress may be unsafe in low dams as it implies dependence on negative pore pressures which may subsequently dissipate; and uneconomical in high dams in wet climates as no account is taken of the dissipation of excess pore pressures during construction.”

I have found the prognosis of Bishop and Bjerrum to be accurate, which is why I work in effective stresses during design using analytical estimates of pore water pressures. Field pore water pressures can then be measured during and after construction to verify or disprove design estimates. If field levels approach design levels, then remedial steps can be taken before critical pore water pressures are reached.

I believe that the stability of an embankment composed of compacted ash, built over a period of decades on sluiced ash, exists in a condition of steady-state seepage according to the laws of nature and should be analyzed based on effective stresses. My opinion is confirmed by the design manual *Slope Stability* of the U.S. Army Corps of Engineers which recommends for steady-state seepage conditions, “Use drained shear strengths related to effective stresses and pore water pressures from field measurements, hydrostatic pressures computations for no-flow conditions, or steady seepage analysis techniques (flow nets, finite element analyses, or finite difference analyses).” According to that manual, total stress analysis based on undrained shear strength is not appropriate for steady-state seepage conditions.

Furthermore, I have reviewed failures of tailings dams and embankments that validate the use of effective stress analysis with estimates of pore water pressures in forensic studies; in particular, the Buffalo Creek Slurry Impoundment in West Virginia that failed in 1972. Initial opinions were offered that the Buffalo Creek embankment failed as a result of piping and liquefaction even though no earthquake occurred at the time of failure.

Post-failure studies performed by the U.S. Bureau of Mines (USBM) showed high hydraulic conductivity in the horizontal direction (k_h) as compared to the vertical direction (k_v) in the hydraulic fill. USBM engineers and geologists concluded that k_h/k_v on the order of 25 to 100 existed at Buffalo Creek, resulting in uplift pressure beneath the embankment which was founded entirely on hydraulic fill (i.e. splitter dike). Computed factors of safety less than 1.0 were reported by the USBM based on steady-state seepage and stability analysis for effective stress conditions as identifying the cause of the progressive failure. Field evidence was then provided to validate those conclusions.

In my opinion, the fundamental mistake made in analyzing the Kingston failure using undrained total stresses, as TVA's forensic team suggests, is that the impact of the water in the problem is masked. I disagree with such an approach. Violent embankment failures are typically caused by water under pressure suddenly being released, which is why I have a sign posted on my wall to remind me, "It's the water, stupid."

INDEPENDENT ASSESSMENT OF THE KINGSTON ASH FACILITY FAILURE

I analyzed the Kingston Ash Facility based on steady-state seepage and effective stress conditions in accordance with current, prudent, engineering practice as documented by the manual, *Slope Stability*, of the U.S. Army Corps of Engineers. Presented in the following section are the results, which are admittedly based on a limited amount of data. In particular, I have little information as to how the Kingston facility was built and monitored during construction. Also, I have incomplete data on the sequence of the failure(s) to assess if the results of my modeling match actual events.

Engineering properties used in the analysis for the sluiced ash, compacted ash outslopes, and compacted earthen fill dikes are based on information provided on TVA's web site and my experience at other facilities. Due to my limited information, I did not include the foundation soil in the modeling.

I believe that artesian pressures, as described in the 2009 TVA inspection report, may be indicative of high hydraulic conductivity in the horizontal direction (k_h) as compared to the vertical direction (k_v) and may be similar to the conditions described previously for the Buffalo Creek Slurry Impoundment. High k_h/k_v conditions can occur in hydraulic fill due to the presence of lenses of coarser-grained material such as bottom ash surrounded by finer-grained material such as fly ash. Values of k_h are controlled by the more pervious horizontal lenses, while values of k_v are controlled by the less pervious finer-grained material. Therefore, I performed a sensitivity study and varied k_h/k_v for the sluiced ash from 1 to 100 in my modeling of the Kingston facility.

Other factors not included in my modeling could also have influenced the failure. Specifically, the initial construction of Dike C using earthen fill appears to have been performed in part below the winter pool elevation (i.e. 735 feet) and the normal pool elevation (i.e. 741 feet) of Watts Bar Lake. Furthermore, annual fluctuations of Watts Bar Lake over the past 50 years could have weakened Dike C by repeated rapid drawdown. Nevertheless, I modeled Watts Bar Lake at its normal pool elevation of 741 feet and ignored possible deficiencies in construction of Dike C due to difficult field conditions. The model also ignores potential impacts associated with fluctuating lake levels and rapid drawdown.

Figures 3, 4, and 5 present the results of finite element seepage modeling for k_h/k_v in the sluiced ash of 1, 10, and 100, respectively. Figures 6, 7, and 8 illustrate that, even at k_h/k_v equal to 100 in the sluiced ash, failure of the northeastern outslope of the dredge cells at Kingston is not predicted. The modeling indicates that the extensive internal drains installed since 1984 provide a stable embankment under normal operating conditions.

The problem area appears to be Dike C, which was built beginning in 1958 with no apparent internal drains, but with a hinge zone in the area of the original earthen dike crest and its first raising. Modeling shows the potential for high artesian pore water pressures to exist in the sluiced ash beneath Dike C during steady-state seepage conditions. Specifically, the relationship between sluiced ash elevation and total water head elevation in the area of the Dike C hinge, for various k_h/k_v conditions, is shown on Figure 9. Computed artesian heads are also presented on Figure 9 for an imaginary piezometer installed from the Dike C bench at elevation 765 feet and screened in the sluiced ash at elevation 740 feet.

With regard to stability, a computed factor of safety greater than 1.5 is predicted in the hinge area of Dike C prior to construction of the dredge cells, when the level of the sluiced ash is at elevation 768 feet, as shown on Figure 10. When the sluiced ash in the dredge cells is at elevation 809 feet in the model, the computed factor of safety of Dike C is slightly greater than 1.0 as shown on Figure 11 and failure is still not predicted; however, when the sluiced ash reaches elevation 816 feet in the dredge cells, as it did on 22 December 2008, the model accurately predicts failure with a factor of safety less than 1.0 as shown on Figure 12.

As stated previously, I did not include the foundation of Dike C in the modeling due to lack of data. Although modeling of Dike C, independent of its foundation, shows the critical computed failure location to pass through the Dike C hinge zone at the time of the actual failure, high pore water pressures caused by steady-state seepage impacts the stability of the entire structure. Figure 13 shows that factors of safety only slightly greater than 1.0 are predicted for potential failure surfaces that pass near the foundation of Dike C. Therefore, alternating layers of clay and sand in the foundation would also be subject to high pore water seepage pressures, and could have impacted the actual failure of Dike C.

OBSERVATIONS AND OPINIONS

Based on the results of modeling, I believe that Dike C could have failed violently due to artesian pressures in the underlying sluiced ash. The sluiced ash upstream of Dike C, left unsupported by the Dike C rupture, would still have pore water under excess seepage pressure and could then have failed explosively. The loss of sluiced ash upstream of Dike C could have then undermined the downstream toe of the dredge cells. Although stable during normal operating conditions, I believe this rapid loss of its foundation could have caused the catastrophic collapse of the Kingston dredge cells in a progressive slope failure mode.

Based on post-failure aerial photographs, a considerable quantity of ash travelled as far as two-thirds of a mile upstream of the site to the northwest, depositing in the Swan Pond Embayment. How could an ash embankment outslope with a northeast orientation violate the law of physics and make this 90 degree turn to the northwest?

One explanation is that Dike C ruptured at its northwestern limb. The excess pore water seepage pressure could then relieve itself through the breach, resulting in the failure of the sluiced ash between Dike C and the downstream toe of the dredge cells in a northwesterly direction. Subsequent failure of the dredge cells due to undermining of its foundation could then have progressed in a northeasterly direction, as would be expected, with subsequent spreading of the failed material due to gravity after it reached Watts Bar Lake. The sequence of the failure, based on the results of steady-state seepage and stability modeling according to effective stresses and post-failure aerial photographs, is shown on Figure 14.

The explosive movement of sluiced ash with pore water under excess seepage pressure after loss of the adjacent supporting mass, followed by a progressive slope failure of the dredge cells, would explain reports by TVA's forensic team of no discernable decrease in void ratio with increasing depth. Also, massive failure of the dredge cells due to undermining of its foundation would explain the presence of a weak shear plane at the base of the sluiced ash. Specifically, measurements taken after the violent failure are probably not representative of the condition of the ash that existed prior to the failure.

If my modeling simulates conditions at Dike C in December 2008, then Dike C is a fitting example of the danger associated with working in undrained, total stresses. As stated previously, an effective stress analysis based on steady-state seepage conditions accurately predicts failure of Dike C. A gradual increase in pore water pressure, caused by the increasing elevation of the sluiced ash in the dredge cells, reduces the stability of Dike C until it ruptures as illustrated by Figures 9 to 14.

Ironically, a stability analysis performed using undrained strength, as suggested by TVA's forensic team, incorrectly predicts a stable Dike C regardless of the pore water pressure in the sluiced ash as shown on Figure 15.

RECOMMENDATIONS

Based on a cursory review of TVA's forensic study report, the relic survey corroborates the failure scenario triggered by the Dike C rupture shown on Figure 14. Nevertheless, the results of my analysis need to be tested against additional field evidence not available to me. Were any of the "old dewatering wells" and "dewatering well overflowing redwater" referenced in the 2009 inspection report located in Dike C? Were any piezometers installed beneath Dike C within the failure zone, and were any of those piezometers being monitored? Could the references to "redwater" indicate the presence of iron from pyrites in the ash? If so, did the dewatering wells eventually clog due to iron deposition, thus reducing their effectiveness? When the valves were closed on the dewatering wells during the October 2008 inspection, what water levels were measured and were the valves re-opened after the inspection?

Answers to these and many other questions are important, and will undoubtedly foment new questions that need answering in order to understand the cause of the Kingston failure so such situations can be avoided or mitigated in the future. As illustrated by Figure 15, use of undrained shear strength to analyze the stability of undrained hinges at other hydraulic fill sites as suggested by TVA's forensic study report is dangerous and not recommended.

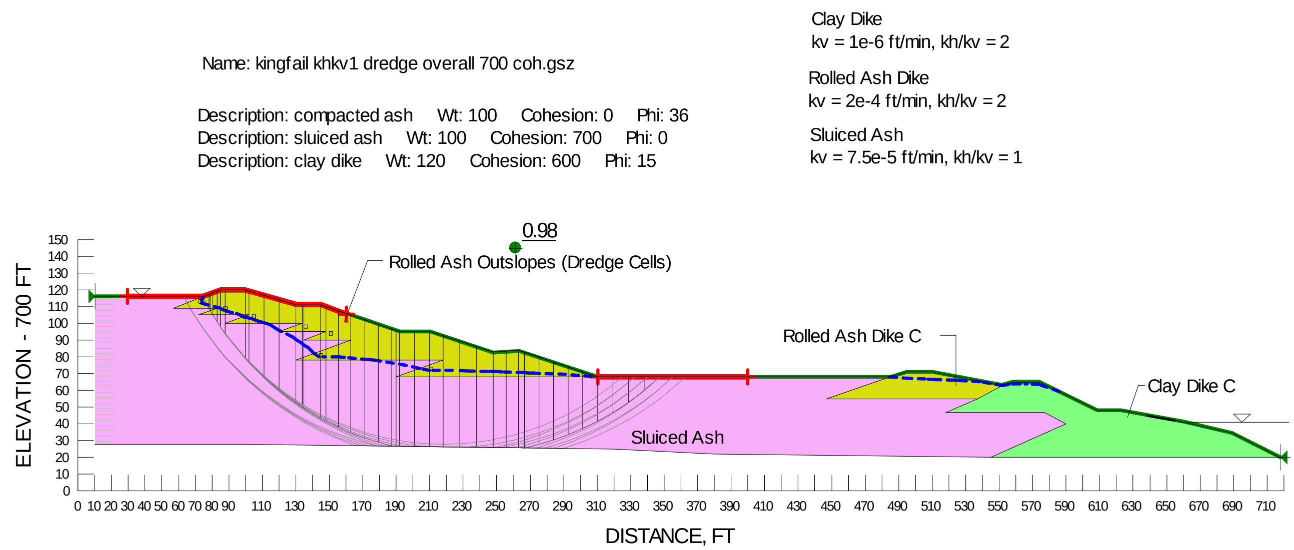

Figure 1. Results of stability analysis for undrained shear strength of sluiced ash = 700 psf and minimum factor of safety less than 1.0 according to the undrained failure theory suggested by TVA's forensic team at Kingston

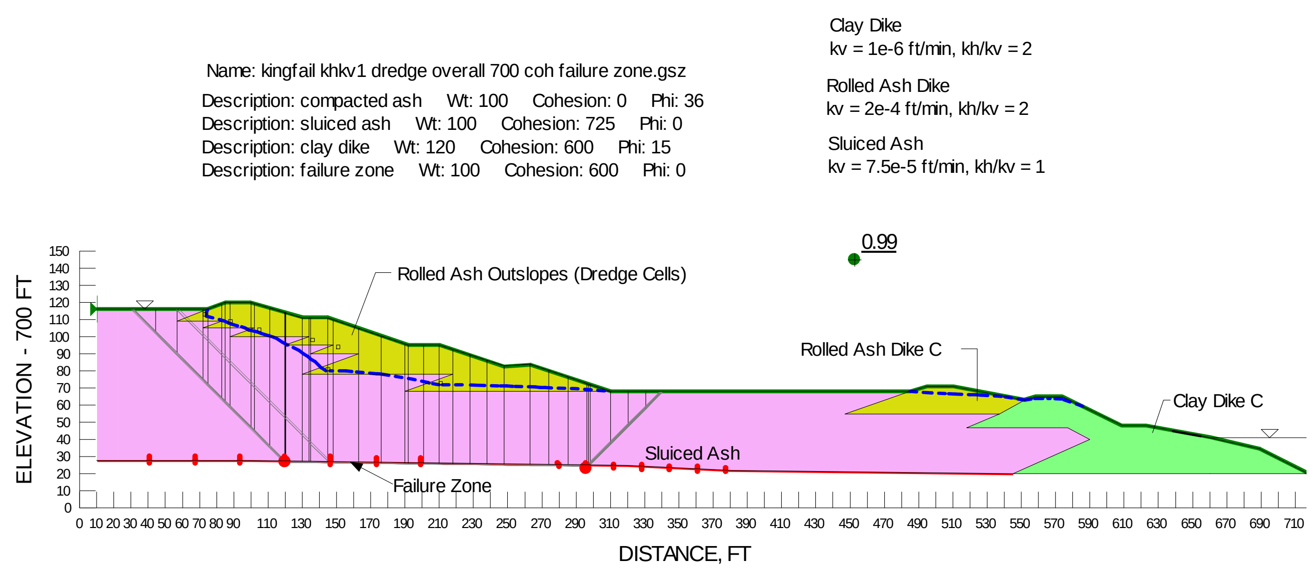

Figure 2. Results of stability analysis for weak bottom layer having undrained shear strength = 600 psf, remaining sluiced ash having undrained shear strength = 725 psf, and minimum factor of safety less than 1.0 according to the undrained failure theory suggested by TVA's forensic team at Kingston

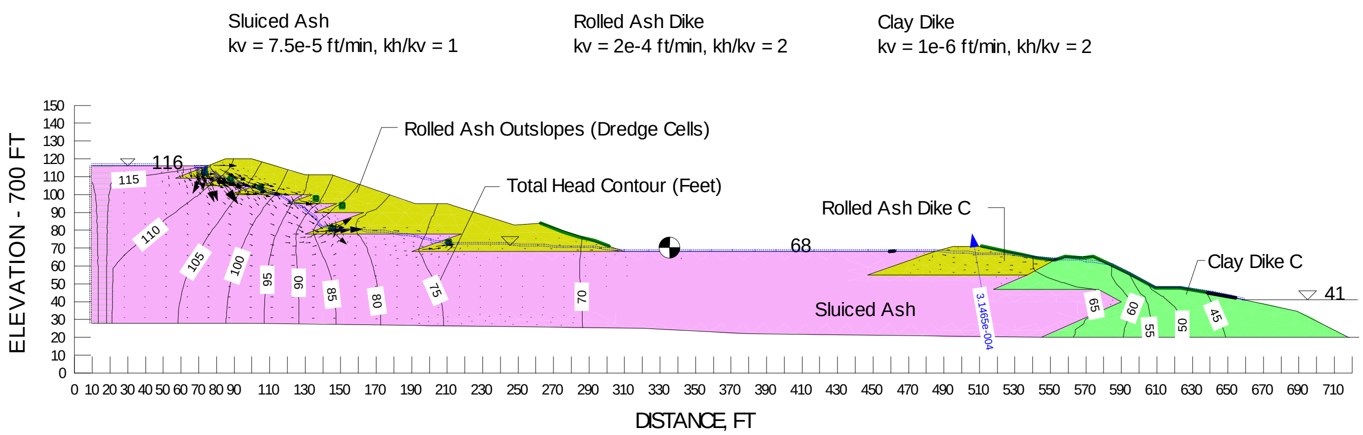

Figure 3. Results of finite element seepage analysis for northeast outslope of Kingston Dredge Cells and Dike C with $k_h/k_v = 1$ in sluiced ash

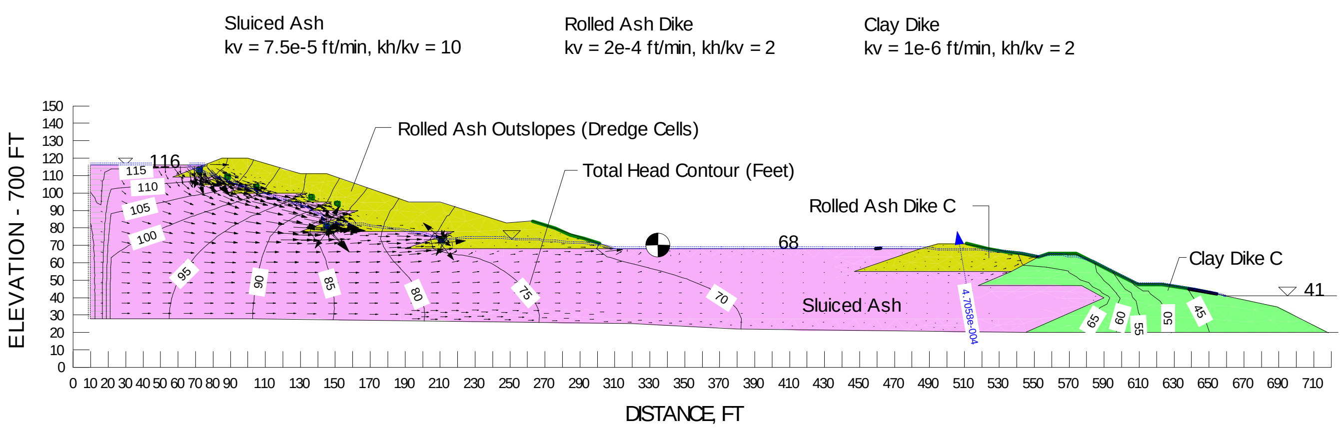

Figure 4. Results of finite element seepage analysis for northeast outslope of Kingston Dredge Cells and Dike C with $k_h/k_v = 10$ in sluiced ash

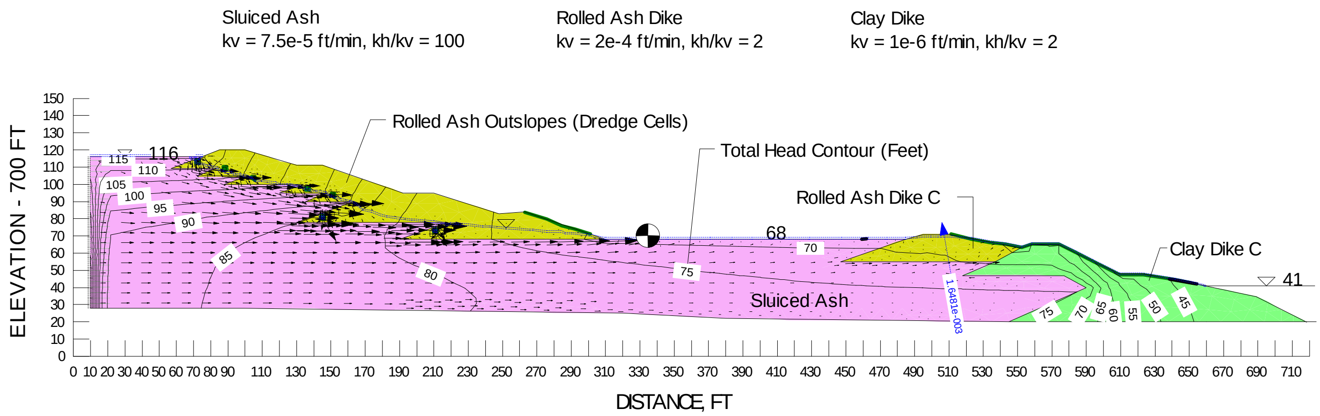

Figure 5. Results of finite element seepage analysis for northeast outslope of Kingston Dredge Cells and Dike C with k_h/k_v = 100 in sluiced ash

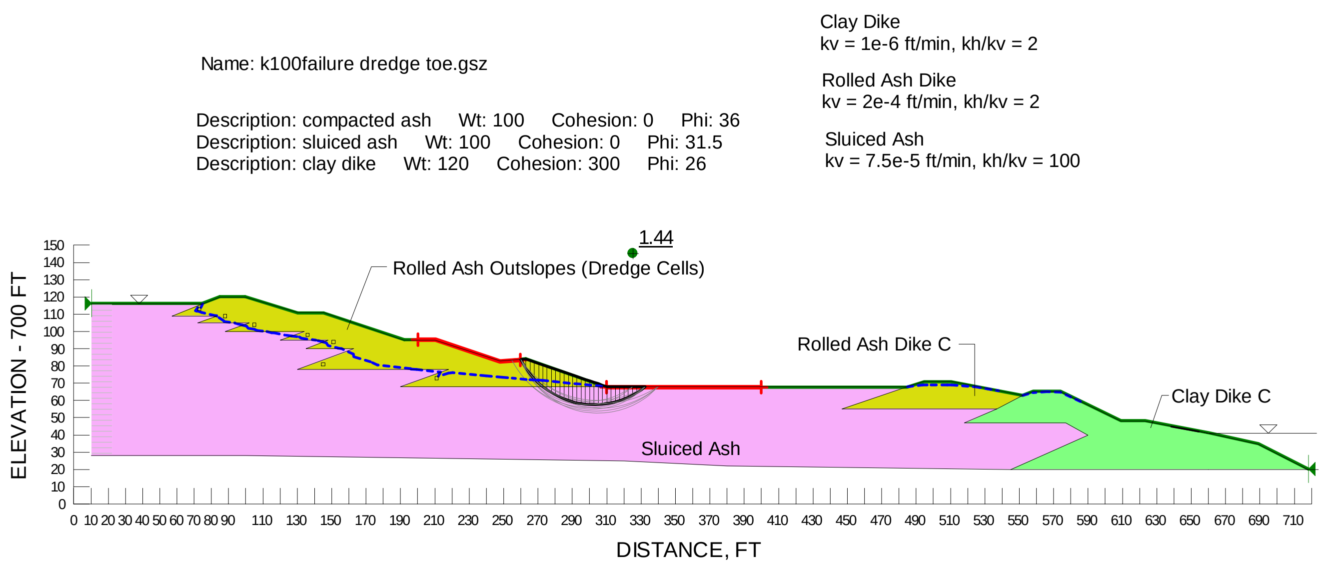

Figure 6. Results of stability analysis for effective stress condition with k_h/k_v = 100 in sluiced ash and minimum factor of safety = 1.44 (for lower toe of northeast outslope of Kingston Dredge Cells)

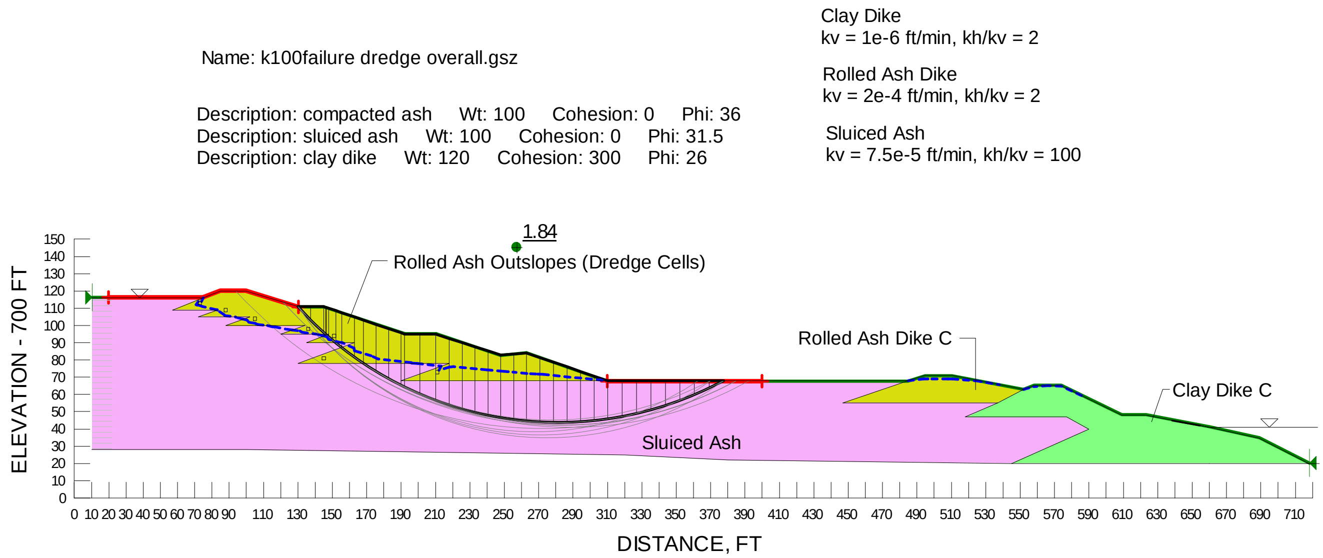

Figure 7. Results of stability analysis for effective stress condition with $k_h/k_v = 100$ in sluiced ash and minimum factor of safety = 1.84 (for mid-height to full-height case of northeast outslope of Kingston Dredge Cells)

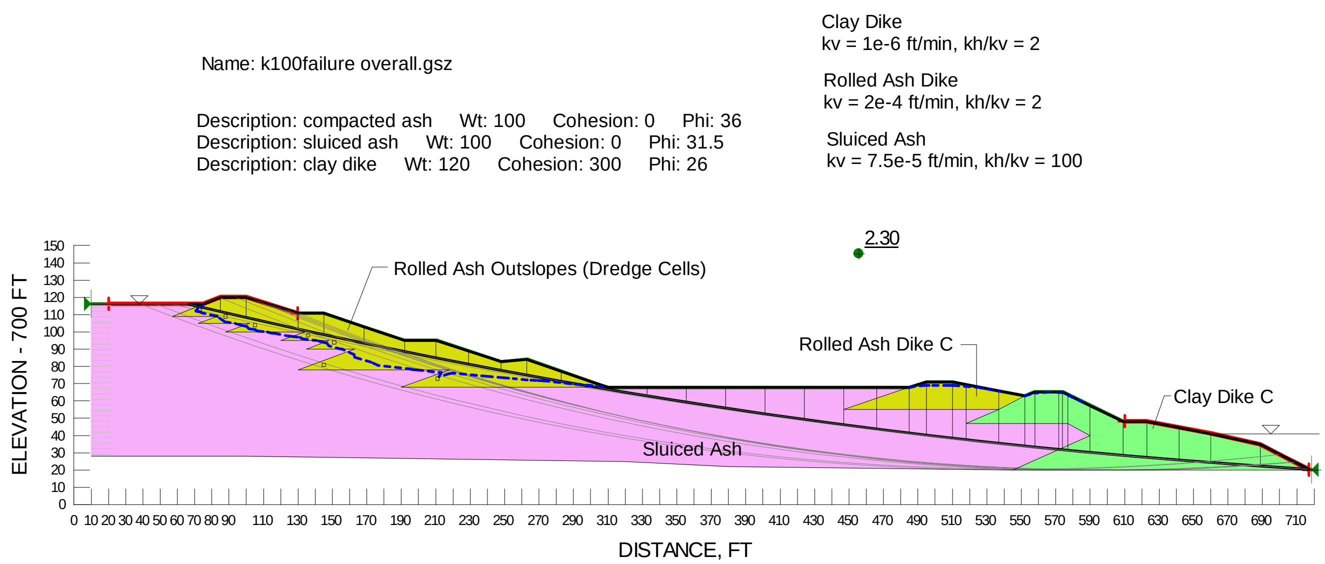

Figure 8. Results of stability analysis for effective stress condition with $k_h/k_v = 100$ in sluiced ash and minimum factor of safety = 2.30 (for full-height case of northeast outslope of Kingston Dredge Cells and Dike C)

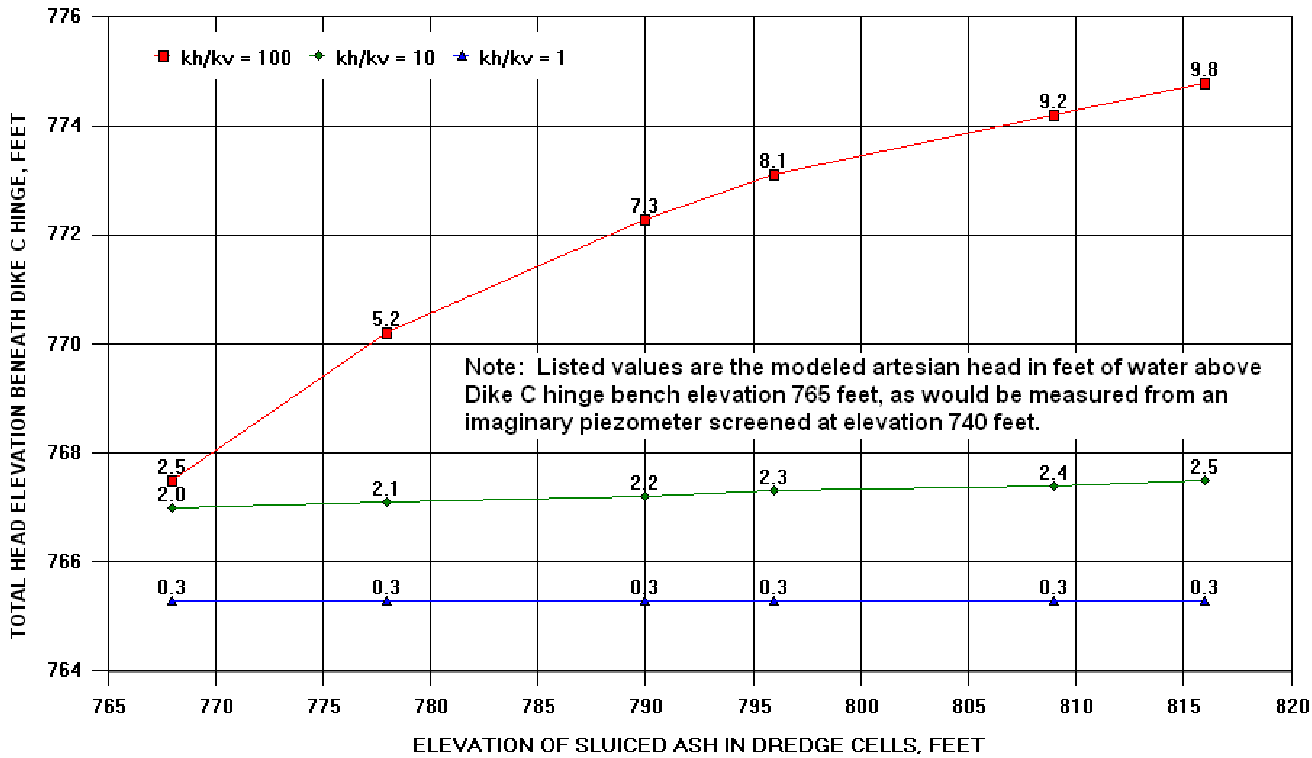

Figure 9. Results of steady-state seepage modeling, with k_h/k_v in sluiced ash ranging from 1 to 100, showing computed total water head elevations in sluiced beneath Dike C hinge area and artesian heads above the Dike C hinge bench at elevation 765

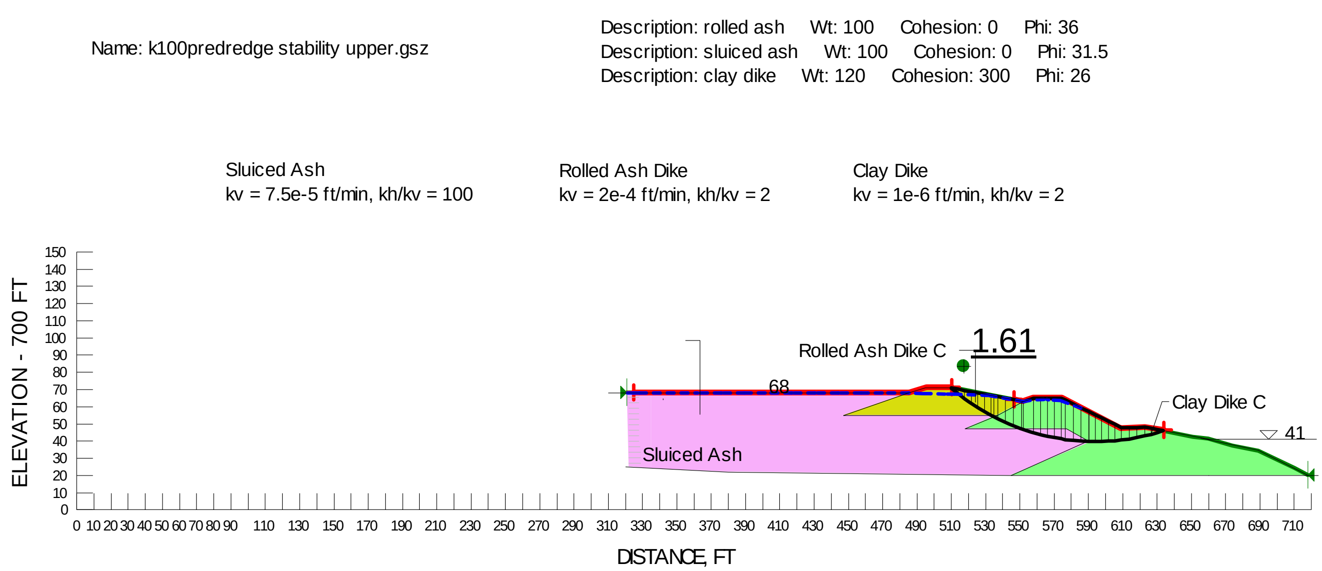

Figure 10. Results of stability analysis for effective stress condition with k_h/k_v = 100 in sluiced ash and minimum factor of safety = 1.61 for Dike C hinge area (for case with sluiced ash elevation = 768 feet before construction of Kingston Dredge Cells)

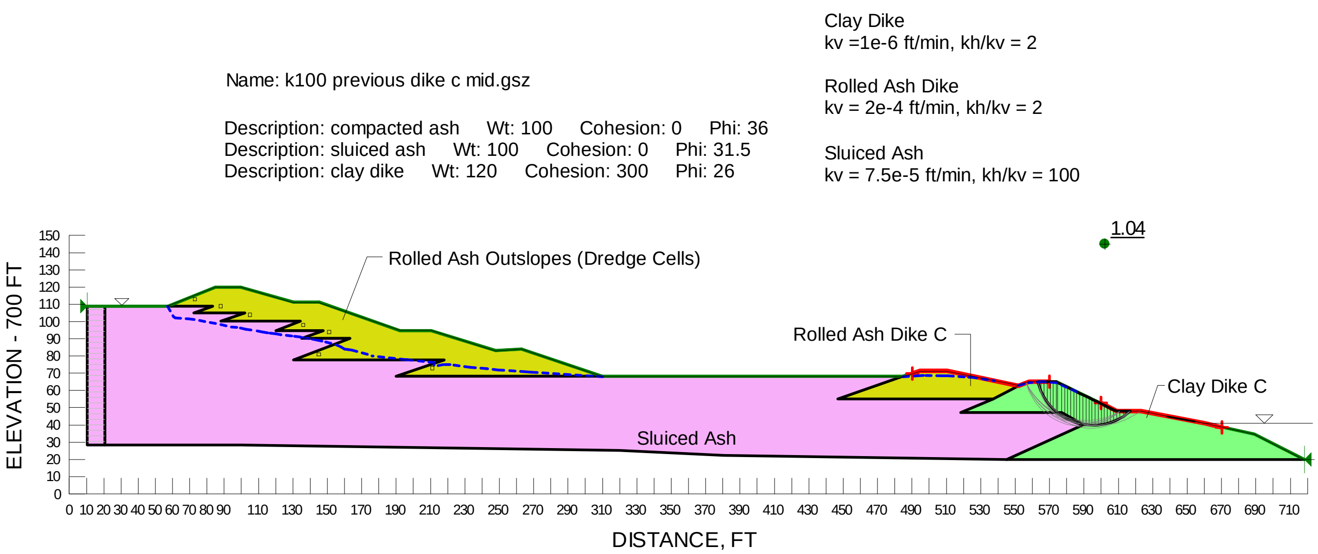

Figure 11. Results of stability analysis for effective stress condition with k_h/k_v = 100 in sluiced ash and minimum factor of safety = 1.04 for Dike C hinge area (for case with sluiced ash elevation = 809 feet in Kingston Dredge Cells)

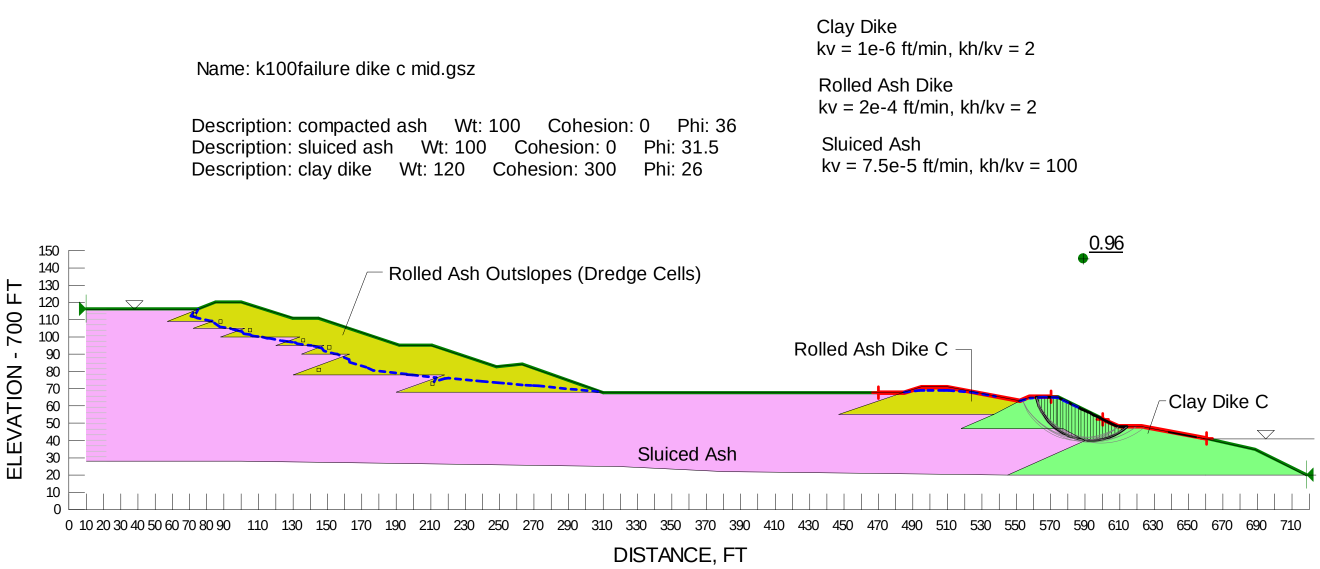

Figure 12. Results of stability analysis for effective stress condition with k_h/k_v = 100 in sluiced ash and minimum factor of safety = 0.96 for Dike C hinge area (for case with sluiced ash elevation = 816 feet in Kingston Dredge Cells, which coincides with condition at the time of failure on 22 December 2008)

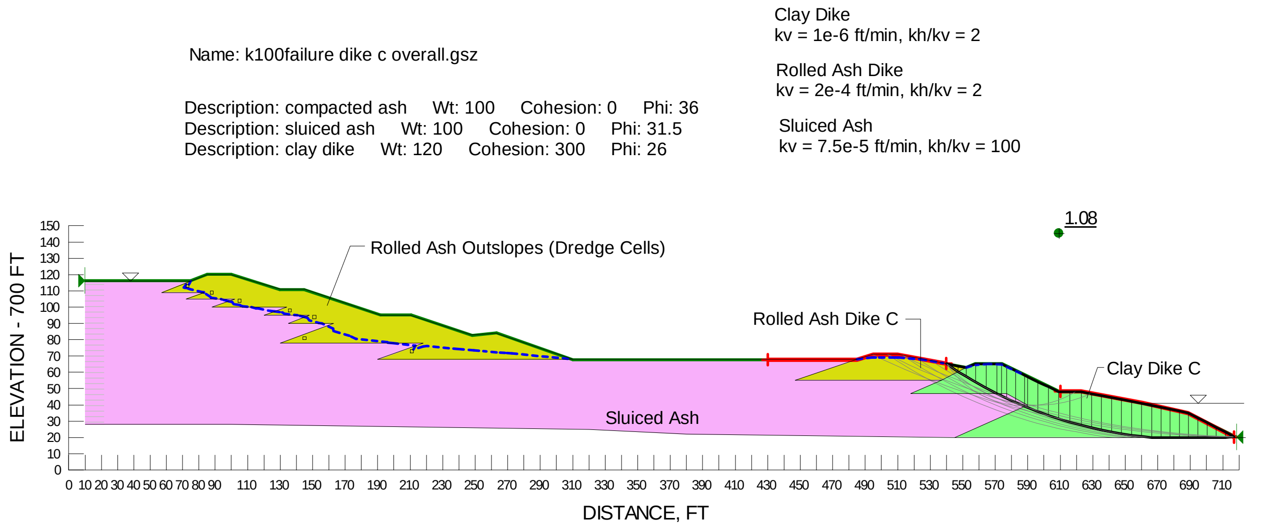

Figure 13. Results of stability analysis for effective stress condition with $k_h/k_v = 100$ in sluiced ash and minimum factor of safety = 1.08 for the overall slope of Dike C (for case with sluiced ash elevation = 816 feet in Kingston Dredge Cells), which indicates the entire Dike C structure is near collapse

Possible Scenario of Failures Triggered by Dike C Rupture

1. Undrained hinge of Dike C in area of original crest and first raising ruptured due to high pore water seepage pressure (X marks the spot).
2. Exposed sluiced ash with underlying bottom ash lenses under high pore water seepage pressure failed explosively through the breach, causing undermining of the toe of the dredge cells.
3. Progressive slope failure of dredge cells resulting from undermining of toe. Each slope failure disrupts seepage path of bottom ash lenses in sluiced ash, resulting in increased pore water pressure and failure of subsequent slope until angle of failed mass reaches equilibrium.

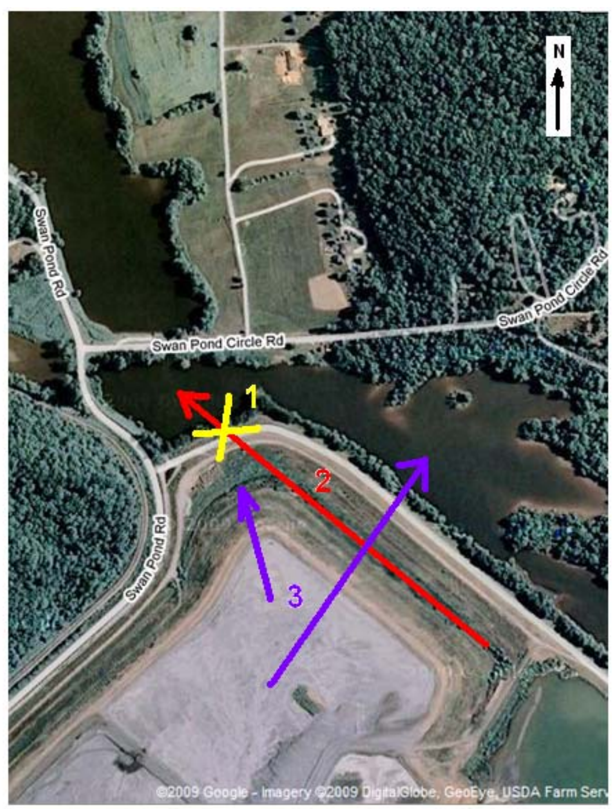

Figure 14. Illustration of possible scenario of failures, as triggered by Dike C rupture, based on results of steady-state seepage and stability analysis for effective stress conditions

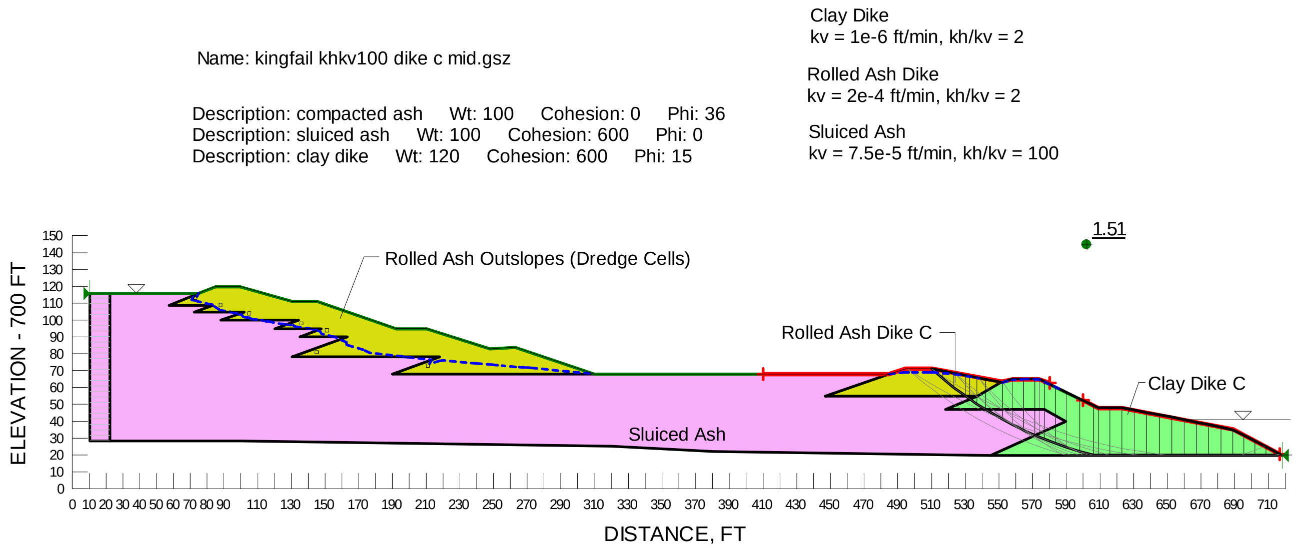

Figure 15. Results of stability analysis for Dike C using undrained strength as suggested by TVA's forensic team with computed minimum factor of safety = 1.51, which indicates stable condition, whereas steady-state seepage and stability analysis using effective stresses predicts failure.

WARNING: Do not use undrained shear strength to analyze stability of undrained hinges (i.e. analyzing the stability of undrained hinges using undrained shear strength is comparable to making chicken salad using chicken droppings… trust me, it won't work).