AECOM

Root Cause Analysis of TVA Kingston Dredge Pond Failure from December 22, 2008

Volume 4 – Seepage and Stability Analyses

Kingston Fossil Plant, Harriman, Tennessee

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4.1 Hydrology

4.1.1 Site Hydrology

The Kingston ash storage area is a confined watershed within the limits of the perimeter dikes of two dredge cells named Dredge Cell 1 and Dredge Cell 2. Since the failure likely started under the north side of Dredge Cell 2, a simplified watershed model was developed to estimate flood inflows and evaluate the flood capacity. The watershed modeling was performed with USDA Natural Resource Conservation Service (NRCS) TR-55 methodology using the HydroCAD computer software. Point precipitation frequency estimates were utilized from NOAA Atlas 14. The tailings basin was assigned a SCS Curve Number of 98 and a time of concentration of 60 minutes. There was reportedly no dredge pumping for 3 days prior to the failure event. The Dredge Cells had the capacity to store runoff from a 500-year, 24-hour duration storm event without considering any outflow from the decant. HydroCAD input and output are provided in Appendix 4A.

The Watts Bar Reservoir (former Emory River) is immediately adjacent to the ash storage area. The FEMA flood insurance rate map (FIRM) provides a 100-year flood elevation of 748 feet for the Emory River north of the ash storage area. A copy of the FIRM maps and the FEMA Flood Insurance Study (FIS) is provided in Appendix 4A.

The TVA operates several dams on the Clinch and Tennessee Rivers which are downstream of the Dredge Cells. The TVA records hourly headwater elevation at the Kingston intake. The recorded stage peaked at 0:00 am CST (1:00 a.m. EST) on December 22, 2008. A copy of the water levels at Watts Bar Dam and at the Kingston plant intake are included in Appendix 4A.

4.2 Seepage Analysis

The objective of the seepage analysis was to determine the position of the steady state phreatic surface in each cross-section, for later export to it the corresponding slope stability models. Seepage analyses were carried out on three (3) representative cross-sections: one cross-section on the north side of Dredge Cell 2 where the initial failure occurred, one cross-section on the west side of Dredge Cell 2, and one cross-section on the East side of the Phase 1 Emergency Cell.

The seepage analyses were performed using SEEP/W 2007 (Version 7.13) finite element flow model, a widely used program developed by Geoslope International. SEEP/W is able to model steady-state flow, considering unsaturated flow conditions and multiple boundary conditions such as total head, seepage face and internal drains. Most of the soil hydraulic conductivity parameters were chosen based on the field CPT dissipation testing, laboratory permeability and consolidation tests performed by AECOM as shown in Table 4.2_1. Selected parameters were adopted from studies performed by others at KIF.

TABLE 4.2_1
SUMMARY OF SEEPAGE MODEL PARAMETERS

Material	Test Method	Kh (cm/sec)	Kv (cm/sec)	Kh/Kv
Ash – Dike (Bottom Ash)	AECOM Lab – Permeameter AECOM Field - CPT_u Dissipation Parsons E&C/Geosyntec (2005)	NT **1.4E-05** 1.0E-04	NT NA -	- **2*** 2*
Ash – Sluiced (Fly Ash)	AECOM Lab – Permeameter AECOM Field - CPT_u Dissipation Parsons E&C/Geosyntec (2005)	**6.3E-05** 6.2E-06 3.7E-05	5.3E-05 NA -	**1** (1.2) - 2*
Laminated – Clay/Silt/Ash (Slimes)	AECOM Field - CPT_u Dissipation 09-104-CI@24ft.	**2.4E-07**	NA	**2***
Alluvium – Silt & Clay	AECOM Lab – Consolidometer AECOM Field – CPT_u Dissipation	- **1.5E-06**	1.9E-06 NA	**1** (0.8) -
Alluvium – Silty Sand	AECOM Field - CPT_u Dissipation	**9.1E-06**	NA	**2***
Compacted Clay (Dike C & Railroad Fill)	AECOM – Field/Lab Parsons E&C/Geosyntec (2005)	NS **5.0E-06**	NS -	- **2***
Shale Bedrock (Fractured)	Parsons E&C/Geosyntec (2005) Velesco & Bohac (1991-best fit) Freeze & Cherry (1979)	**1.0E-06** 3.3E-06 1.0E-05	- - -	**2*** - -

The as-built geometry, geological profile and output consisting of contour drawings of total head, porewater pressure and pressure head are included in Appendix 4B.

4.3 Stability Analysis

4.3.1 General

Slope stability analyses were performed using infinite slope methods, manual block failure calculations and commercial computer software (SLOPE/W 2007 - Version 7.13). SLOPE/W provides numerical tools to analyze the stability of embankments using limit equilibrium methods. Among the available methods, the Morgenstern-Price Method is considered to be mathematically rigorous, since it satisfies both force and moment equilibrium; thus, it was adopted for two-dimensional slope stability analysis. The SLOPE/W Optimization function was also selected for certain cases. The geometry of the sections used in the analysis were based on interviews and TVA mapping, as a best estimate of the topography just prior to failure.

4.3.2 Stability Model Parameters

Most of the index and shear strength parameters were chosen based on the field and laboratory tests performed by AECOM. Certain parameters were selected based on the work of others, as noted. The criteria used for the selection these parameters are discussed in Section 1.5.

The tables below summarize the strength parameters used for analysis (described in section 1.5).

Sluiced Ash

Unit Weight: 107 pcf

Analysis	Type	Units	Value
Drained	Friction Angle, ϕ'	degrees	30
Undrained (Peak)	Strength ratio, s_u/σ'_{vo}	dimensionless	0.3
Undrained (Residual)	Undrained Strength, $s_{u\text{-}ss}$	psf	100

Compacted Ash (Dikes)

Unit Weight: 120 pcf

Analysis	Type	Units	Value
Drained	Cohesion Intercept, c'	psf	0
	Friction Angle, ϕ'	degree	37

Slimes

Unit Weight: 90 pcf

Analysis	Type	Units	Value
Undrained	Undrained Strength, s_u	psf	600~1,500

Alluvium (Clays and Silts)

Unit Weight: 110 pcf

Analysis	Type	Units	Value
Undrained	Undrained Strength, s_u	psf	1,200

Alluvium (Silty Sands and Silts)

Unit Weight: 120 pcf

Analysis	Type	Units	Value
Drained	Cohesion Intercept, c’	psf	600
	Friction Angle, ϕ'	degree	30

Railroad Fill

Unit Weight: 120 pcf

Analysis	Type	Units	Value
AECOM Adopted Parsons (2004) Shear Strength Parameters	Cohesion Intercept, c’	psf	0
	Friction Angle, ϕ'	degree	37

Road Fill (Dike B)

Unit Weight: 114 pcf

Analysis	Type	Units	Value
AECOM Adopted Parsons (2004) Shear Strength Parameters	Cohesion Intercept, c’	psf	0
	Friction Angle, ϕ'	degree	37

Compacted Clay Fill

Unit Weight: 120 pcf

Analysis	Type	Units	Value
AECOM Adopted Parsons (2004) Shear Strength Parameters	Cohesion Intercept, c’	psf	600
	Friction Angle, ϕ'	degree	15

4.3.3 Limit Equilibrium Analysis

4.3.3.1 Dredge Cell No. 2 Northwest

The failure at the northwest corner of the Dredge Cell No. 2 on December 22, 2008 was evaluated in four stages, as the failure occurred in a progressive manner as discussed in Section 1.5 of this report. AECOM used TVA surveys to model the upstream dike geometry. These stages include:

1. Stage 1 - Evaluated the initial pre-failure failure conditions at the upstream dike area of the northwest dike section. The sluiced ash was assigned a drained strength state. The slimes were assigned an undrained strength state and were assumed to be in the initial stages of yielding (creep).
2. Stage 2 - Analyzed conditions after the sluiced ash goes to an undrained strength condition and the slimes continues to yield.
3. Stage 3 - Evaluated the stability of Dike C, before and after the upstream dikes failed and the sluiced ash liquefied and stacked up against Dike C.
4. Stage 4 - After the failure of Dike C, the stability of the remaining dredge cell south of the Cell 2 upstream dikes was evaluated for southward progressive failure.

For Stages 1 and 2, two cases for the undrained strength of the foundation slimes were modeled. Limit Equilibrium Method Analyses were used for Stages 1 though 4 for this section and a simplified sliding block analysis was conducted for Stage 2 (see Section 1.6).

4.3.3.2 Dredge Cell No. 2 Southwest

Two cases on this cross-section were analyzed (using TVA-defined slope geometry): Case 1 for the sluiced ash having drained shear strength with an effective friction angle of 30° and no cohesion and Case 2 for the sluiced ash having undrained shear strength versus effective overburden stress ratio of $S_{up}/\sigma_{v'}$ **=** 0.30. In Case 1, the potential slip surface runs through the laminated sensitive silts, touches the bottom of clay alluvium and drives out through Dike B. Case 2 indicates the potential slip surface runs through the sluiced ash and drives out along the upstream side of the Roadway Dike B.

4.3.3.3 Phase I Emergency Dredge Cell East

Using TVA-defined slope geometry, two cases on this cross-section were analyzed by grid and radius circular slip surface method: Case 1 for sluiced ash having drained shear strength with an effective friction angle of 30° and no cohesion and Case 2for sluiced ash having undrained shear strength of $S_{up}/\sigma v'$ = 0.30. Case 1 shows the potential slip surface runs through the sluiced ash. In Case 2, the potential slip surface runs through the bottom of clays alluvium (and top of silty sand alluvium).

The output from the Slope/W analyses are included in Appendix 4C and are discussed in Section 1.6.

4.3.4 Cell 2 Wedge Block Stability Analysis

A hand computation was completed, using wedge block stability methods described in Taylor (1948), Sowers (1979), Terzaghi, Peck and Mesri (1996) and active and passive wall movement theory from the Naval Facilities Design Manual (NAVFAC DM 7.2, 1982) to evaluate strain compatibility between ash and slime materials resisting instability in the active, central and passive blocks of Northwest Cell 2. This method assumes an active wall pushed against a central block on soft material and the passive wall that resists sliding.

The wedge block stability was computed separately for 5% and 20% lateral strain, in a slime layer assumed 6 inches thick. These deformations were matched with active and passive earth pressure coefficients consistent with 0.3 and 1.2 inches of lateral movement, respectively. Test results on the slimes show peak undrained shear strength at 5% shear strain and a decrease in strength of about 25% at 20% shear strain. Stability analyses indicate that when the strain in the slimes increases from 5% to 20%, the factor of safety decreases from 1.1 to 0.9.

4.3.5 Infinite Slope Analysis

An infinite slope analyses was performed for dry slope conditions, seepage parallel and below the slope surface, and seepage emerging from the slope, with reference to the U.S. Army Corps of Engineers Manual EM1110-2-1902, Figure E-7, and using the TVA design slope of 3H:1V for upstream ash dike construction. The resulting factors of safety were 2.3, 1.4 and 0.9 (see Section 1.6).

4.3.6 Flow Slide Analysis

With a maximum sluiced Cell No. 1 at El. 817 feet and a slide plane elevation of approximately 730 feet, portions of the 87-foot-tall body of wet ash exhibited a total displaced movement of about 4,600 feet from the Cell 1 / Cell 3 divider dike, northwest along Slough No. 2, and about 3,300 feet north then east to reach the Watts Bar Reservoir / Emory River. The final difference in elevation of the ash flow was about 11 to 18 feet over the above-referenced distances, which indicates an overall final angle of the repose of the ash estimated at 0.15 to 0.3 degrees (from horizontal).

From the work of Jeffries and Been (2006) and Lambe and Whitman (1968), the undrained steady-state shear strength (S_{us}) of the ash can be estimated as follows:

$$S_{us} = \sigma'_{vo} \quad (\gamma_t / \gamma') \quad \cos\theta \ \sin\theta$$

Using a total unit weight of 107 pcf, a submerged unit weight of 45 pcf, and an angle of repose (θ) of 0.15 to 0.3 degrees, S_{us} of the ash is estimated to be 25 to 50 psf. Since this analysis does not correct for failure momentum

effects, the results are expected to represent lower-bound values as compared to consolidated undrained triaxial compression test data measured by AECOM on loose ash laboratory samples (typically S_{us} of 100 to 250 psf).

4.4 Dredge Cell Filling Rates

Data on historical dredge cell filling rates (provided by TVA) and AECOM calculations from that data are discussed in Section 1.7 and are presented in Appendix 4D.

Appendices

Appendix 4A – Hydrology

Appendix 4B - Seepage Analysis

Appendix 4C – Stability Analysis

Appendix 4D – Dredge Cell Filling Rates

Appendix 4A - Hydrology

NOAA Precipitation Data

Kingston KIF Precipitation Data

HydroCAD Rainfall Model

Flood Insurance Rate Maps

FEMA Flood Insurance Study – Roane County

Historic River/Reservoir Levels

Water Level vs. Time – Watts Bar Reservoir

Water Level vs. Time – Kingston Plant Intake

Appendix 4B - Seepage Analysis

Appendix 4C – Stability Analysis

Limit Equilibrium Slope Stability Analysis

Wedge Block Slope Stability Analysis

Wedge Block Slope Analysis

Flow Slide Analysis

Appendix 4D – Dredge Cell Filling Rates

Calculations

Email Re: Dredge Cell Area

KIF Dredge Cell Records