Perils of Constructing a Gypsum Stack on Top of an Ash Pond
TVA Bull Run Fossil (BRF) Plant Gypsum Disposal Area 2A

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KEYWORDS: Slope stability, seepage control, instrumentation, byproduct

1. BACKGROUND

The Tennessee Valley Authority (TVA) Bull Run Fossil Plant (BRF) is located near Clinton, in Anderson County, Tennessee. The plant building complex is situated just east of Clinch River, while three of its coal combustion residual (CCR) storage areas lie along the banks of the river (Melton Hill Reservoir) and its tributary Bull Run Creek. Nearby cities include Oak Ridge, which is located approximately six miles to the west, and Knoxville, which is located approximately 12 miles to the southeast.

The three CCR storage facilities include the Bottom Ash Disposal Area (also known as Area 1), Gypsum Disposal Area (also known as Area 2A) and the Fly Ash Pond (also known as Area 2). The Fly Ash Pond includes a Stilling Pond. A fourth CCR storage facility is a Dry Ash Disposal Area located on the northeast side of the plant. The location of these facilities is provided in Figure 1.
The plant was constructed in the 1960s along with an initial CCR storage pond where ash was sluiced. The ash pond was created by constructing perimeter clay dikes next to the river/creek banks and lining the bottom with a geologic buffer. The pond was initially divided into two large areas, designated as Areas 1 and 2, separated by a drainage channel; in 1981 a divider dike was formed within Area 2 to create Area 2A. At first all areas received sluiced ash through pipes that started at the plant and ended at the northeast corner of Area 1. Later, bottom ash sluiced from the plant was allowed to dry and then stored on top of the sluiced ash in Area 1 and on portions of Area 2A,
while fly ash continued to be sluiced into Area 2 via a channel traversing along the east side of both areas. A divider dike was also constructed to form a stilling basin within the southwest portion of Area 2.

With scarce space to develop new CCR stacking facilities within reasonable distance of the BRF plant, TVA elected to construct a gypsum stack on top of Area 2A. Construction started in 2006 and the stack went online in 2008. At that time it was estimated that approximately 240,000 dry tons of scrubber gypsum would be produced each year and transported to Area 2A through wet sluicing.

Although dry ash had previously been stacked on top of ash ponds in other areas of the ash pond, this time TVA designed a stack with perimeter dikes located entirely within the ash pond where gypsum would be sluiced. This implied the impact of sluice water within the dikes had to be considered in the design of the gypsum disposal facility in addition to the challenges of constructing most of the stack on top of soft ash deposits.

2. DESIGN, CONSTRUCTION AND SLOUGHING OF GYPSUM DISPOSAL AREA 2A SOUTH SLOPE

DESIGN

The original dike forming the north side of Area 2 also forms the north side of Area 2A. The west, south and east sides of Area 2A were constructed within the ash pond starting at elevation 810 feet and using uniform 3:1 slopes up to elevation 835 feet, with a 20-foot wide intermediate bench at elevation 825 feet. The north slope varied in steepness from 1.5H:1V to 3H:1V and included an intermediate bench at elevation 818 feet. The east slope toes out along a sluice ditch channel; the south slope of Area 2A toes out into the Fly Ash Pond; the west slope toes out near the upstream side of the Area 2 west perimeter dike crest; and the north side toes out along a drainage channel. Two 30-inch CMP Spillway Outlets were designed to route discharge water through the south slope into the ash pond (Area 2).

The south dike of the gypsum disposal area was constructed over sluiced ash deposits that extend into the Fly Ash Pond (Area 2) well beyond the toe of the dike. In view of the foundations conditions, the design of the south dike included certain features to reduce the potential for settlement or failure due to pore pressure buildup within the soft sluice ash foundation deposits, and to control seepage across the dike. A review of the design, construction modifications and performance of the south dike is presented in the following paragraphs.

The design of this area called for vertical bottom ash columns to be constructed in a grid pattern within the sluiced ash deposits underneath most of the south dike. According to TVA, the columns were actually constructed with manufactured sand instead of bottom ash. The columns were to extend from elevation 810 feet down to native soil (approximate elevation 784 feet). The purpose for the columns was to reduce pore pressure buildup within the sluiced ash deposits during construction of the south dike through dissipation into the columns. In addition, a 5-foot thick blanket of bottom ash was constructed between elevations 810 feet and 805 feet, around the columns, to
further promote pore pressure dissipation near the top of the columns. The top of the 5-foot thick blanket of bottom ash was to be lined with geotextile fabric, which was to extend from the bottom (interior) of the disposal area toward the south toe of the exterior slope, and end about 5 feet from the face of the slope. In addition, the interior toe area design included a perimeter under-drain (6 inch perforated pipe wrapped in crushed stone and filter fabric traversing parallel to the slope contours) at elevation 810 feet. The perimeter under-drain was to include lateral outlet pipes (6 inch non-perforated pipe) at 100 feet intervals, as indicated on the detail reproduced below in Figure 2.

![Figure 2. Detail of South Slope Toe Area Design](image)

Design records also show a series of 4-inch diameter perforated under-drain pipes traversing across the entire bottom of the disposal area in a north-south direction, with the pipes installed at 50-foot intervals and discharging near elevation 815 feet. Additionally, the design called for another perimeter under-drain at elevation 825 feet with 6-inch lateral outlet pipes installed at 125-foot intervals, and perimeter drains at elevation 815 feet running parallel to the east and west sides of Area 2A, with lateral outlet pipes installed at 250 feet spacing intervals.

An October 2009 reconnaissance of the exterior dike slopes revealed the presence of two sets of lateral pipes exiting the south slope and one set exiting each the west and east slopes of the dike. On the south dike, one set consisted of six 6-inch diameter
lateral pipes daylighting slightly below elevation 825 feet at approximately 125-foot intervals. The second set daylighted near the toe of the slope, slightly above or below the normal ash pond pool elevation. A total of ten 4-inch diameter or 6-inch diameter lateral pipes were found daylighting near elevation 806 feet at 50-foot to 100-foot intervals.

CONSTRUCTION AND SLOUGHING OF SOUTH DIKE SLOPE

As the construction of the initial gypsum disposal area dike was being completed in 2007, seepage through the lower portion of the dike caused sloughs along the exterior toe of the south dike. In addition, erosion around the 30 inch CMP outlet of the spillway pipes was observed, presumably caused by leakage through openings encountered in pipe joints. The 2007 sloughing was repaired along with some modifications to the seepage control features as discussed in the next paragraphs. However, new sloughing developed along the toe of the south embankment and was observed on October 6, 2009. Information from slope inclinometers installed as part of this exploration (discussed in Section 3) show the bottom of the sloughing to be near elevation 803 feet. The photos below show both the 2007 (Figure 3) and 2009 (Figure 4) sloughing observed as described above.

Figure 3. Area 2A – Gypsum Disposal Area – 2007 South Slope Sloughing
According to project records, the original seepage problems were corrected by implementing several corrective measures along the toe of both the exterior and interior south dike slopes and the south side of the gypsum stack bottom. The 30 inch CMP spillway pipes were slip-lined using a 24 inch HDPE (butt-fusion weld), which included grouting the annular space between the two pipes. The erosion (voids) observed around the outlet of the spillway pipes was to be filled by grouting.

In addition, under-drainage through the 4-inch perforated HDPE pipe system installed through the south dike was cutoff. These under-drain pipes were connected to a 12-inch header pipe installed north (upstream) of the interior toe of the embankment slope. This header pipe included a penetration into each of the 24-inch spillway riser pipes, such that the water collected by the header pipe would drain into the spillway pipes.
The header pipe also included two 12-inch HDPE stand pipes immediately south of each stop log structure (weir). The under-drain perforated pipes (finger drains) through the dike were to be plugged, retaining the outer 20 feet (+/-) as a toe drain but blocking drainage from within the embankment. Repair details called for field plugging these under-drain pipes using bentonite pellets. Repair design notes called for a clay plug/shoulder to be constructed along the interior toe of the embankment, immediately south of the 12-inch header pipe.

Records also showed recommendations to change the bottom drainage system by installing drainage collection pipes wrapped in gravel and geotextile filter fabric. Figure 5 presents photos of the modified bottom drainage system.
Other changes to the original design included the construction of crushed stone pocket drains on 50-foot centers along the exterior toe of the embankment. These pockets were to be constructed by excavating approximately 14 feet into the slope at elevation 805 feet, with the top of the excavation located near elevation 812 feet. The pockets were to be lined with non-woven filter fabric and formed using #7 crushed stone.
Thirteen pocket drains installed at 50-foot centers were found near the toe of the south slope during observations made in October 2009. Each pocket was 2 to 3 feet wide and appeared to have been constructed using #7 or finer crushed stone. The toe areas between pocket drains appeared to have been repaired using ash and clay.

3. GEOTECHNICAL EXPLORATION OF THE AREA 2A SOUTH SLOPE

Stantec advanced a number of sampling and testing borings within Area 2A dikes and slopes in 2009, used the results of this work and project records to perform seepage and stability analyses, and developed recommendations to improve the stability of the dike slopes. Instrumentation consisting of piezometers and slope inclinometers were installed in selected borings to monitor subsurface movement and phreatic levels along the perimeter dikes of Area 2A. The description presented below pertains only to the south side of Area 2A.

The south dike of Area 2A, comprised primarily of compacted bottom ash material co-mingled with fly ash, was constructed over sluiced ash deposits. A 1 to 1½-foot thick capping layer consisting of red-brown, lean and fat clay (CL and CH) made up the surficial layer of the ash dike subsurface profile. In general, the bottom ash was gray to dark gray and dense to very dense with occasional pockets of loose relative density. As mentioned, the ash dike system was underlain by sluiced ash materials. According to N-values derived from standard penetration testing (SPT), the sluiced ash materials exhibited a very loose to loose relative density. The thickness of the sluiced ash deposits ranged from 14 feet to 26 feet; these deposits were underlain by alluvial clays and sands. Some alluvial silts were also noted in several borings at the interface between the sluiced ash and alluvial clays, and had soft to medium stiff consistencies according to the SPT blow counts.

Geotechnical engineering analyses included evaluations of strength and permeability parameters, seepage analyses, and slope stability analyses. Prior to performing the analyses, Stantec developed the dike geometry at three cross-sections of the south dike using survey data provided by TVA, design drawings, site observations, and the results of the drilling and lab testing programs conducted by Stantec. Once the geometries of the sections were determined, each section was reviewed and evaluated for potential slope failure and two cross-sections (designated as Sections K-K’ and L-L’) were selected for further analysis. The criteria for selecting the critical sections were based on the steepness of slopes, the geometry of the sections, the piezometric surface, and the subsurface conditions. Permeability and strength parameters were derived based on the results of the drilling and lab testing programs, supplemental in-situ testing, historical information from past explorations, and Stantec’s past experience with similar soils and CCR materials. Figure 6 shows among other information the location of cross sections K-K’ and L-L’ and the borings advanced on the south dike of Area 2A. Figures 7 and 8 show the two cross sections obtained from the results of the stability evaluation performed on the conditions existing in 2009.
Slope Stability

Cross-Section K-K'
Gypsum Disposal Area 2A
Bull Run Fossil Plant
Tennessee Valley Authority

Factor of Safety: 1.2
Center: (380, 855) ft
Radius: 53.802 ft
Minimum Slip Surface Depth: 10 ft

Material Type | Moist Unit Weight | Saturated Unit Weight | Cohesion | Friction Angle
---|---|---|---|---
Ash Dike | 100 | 105 | 0 | 30
Silty Sand to Sandy Silt (Aluminium) | 106 | 107 | 0 | 29
Leach Clay (Aluminium) | 123 | 123 | 0 | 31
Silt (Aluminium) | 108 | 109 | 0 | 28
Bluized Fly Ash | 100 | 105 | 0 | 25

Notes:
The results of the analysis shown were developed based on available subsurface information from a limited number of exploratory locations, with the assumption that the materials exposed fairly represent the subsurface conditions within the areas explored. No guarantee can be made regarding the subsurface conditions between the explored locations, and such unknown conditions could have an impact on the results of the analyses.

Figure 7. Slope Stability Cross-Section K-K' – Existing Conditions
An analysis of steady state seepage through the dike was needed to estimate the magnitude of seepage gradients (for the evaluation of potential piping) and pore water pressures within the soils (for the evaluation of slope stability). The numerical seepage model was developed using SEEP/W 2007 (Version 7.14), a finite element code tailored for modeling groundwater seepage problems in soil and rock. SEEP/W is distributed by GEO-SLOPE International, Ltd, of Calgary, Alberta, Canada. Figures 9 and 10 show the results of the seepage analysis performed on the two cross sections for the conditions existing in 2009.
Figure 9. Seep/W Analysis Cross-Section K-K' – Existing Conditions
Figure 10. SEEP/W Analysis Cross-Section L-L’ – Existing Conditions

SEEP/W Analysis

Cross-Section L-L’
Gypsum Disposal Area 2A
Bull Run Fossil Plant
Tennessee Valley Authority

February 2015
Method: Steady State Seepage
File Name: 2A.gz
(See Workplan for file name convention)

Note:
The results of the analysis shown were developed based on available subsurface information from a limited number of exploratory locations, with the assumption that the materials exposed fairly represent the subsurface conditions within the project area. No guarantee can be made regarding the subsurface conditions between the explored locations, and such unknown conditions could have an impact on the results of the analyses.

Piping Potential
Maximum Exit Gradient occurs at (400,800)
Total Head = 806.50 ft
At (399,44,736.39)
Total Head = 807.50 ft
dH = 1.0 ft
dL = 3.61 ft
i = 0.277
I crítica = 0.708
PSpiping = 2.6

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<th>Ksat</th>
<th>Kneto</th>
<th>Wsat</th>
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<td>Lime Fly Ash</td>
<td>4.500e+007</td>
<td>0.00</td>
<td>0.00</td>
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<tr>
<td>Slag Fly Ash</td>
<td>1.613e+025</td>
<td>0.00</td>
<td>0.00</td>
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<td>Ash Dike</td>
<td>6.5e-006</td>
<td>0.04</td>
<td>0.46</td>
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Existing Condition
In addition to the obvious sloughing failure observed along the toe of the slope, the results of the stability and seepage analyses indicated the presence of failure planes located well below the dike slope and extending into the ash pond with factors of safety of 1.2 at Section K-K’ and 1.1 at Section L-L’ against slope failure. These values were below the acceptable criteria.

4. STABILIZING A DIKE SLOPE CONSTRUCTED ON TOP OF SLUICED ASH

Existing conditions limited the options available to repair the slope and provide long term stability. The stack was actively receiving sluiced gypsum and although there would be occasional scheduled plant shutdowns, reworking the dike or constructing drainage features within the dike to improve its stability was not practical. On the other hand, any design of work located ‘in front’ of the dike -- to enhance the stability of the dike by adding resistance (massive or otherwise) to the sliding -- needed to address the very soft foundation conditions posed by the sluiced ash.

Historically, TVA was successful in stacking dry CCR materials on top of sluiced ash, as was the case in Area 1 and the other three sides of Area 2A. Similarly, Stantec had designed CCR stack expansions for other clients, where CCR material was placed successfully on top of sluiced ash in a controlled manner.

Further engineering analyses showed that constructing a buttress of reduced height in front of the south dike would provide the additional resistance needed to achieve factor of safety values matching or exceeding the minimum accepted criteria. Figures 11 through 14 show the results of the stability and seepage analyses for a condition where a rock buttress is added against the toe of the dike, with most of the buttress constructed on top of sluiced ash. An attempt was made to minimize the height of the buttress to reduce its stress influence on the sluiced ash deposits serving as foundation media. The top of the buttress was set at elevation 808 feet, which made the buttress as much as 14 feet thick in areas with a lower pond bottom. The design buttress crest width and out-slope were 80 feet and 3H:1V, respectively, and the buttress total length was approximately 1,230 feet. Figure 15 depicts a layout and typical sections of the buttress design.
Slope Stability

Cross-Section K-K'  
Gypsum Disposal Area 2A  

Bull Run Fossil Plant  
Tennessee Valley Authority

February 2010  
Method: Modified Spencer  
(File Name: 120spc)  
(See workplan for file name convention)

Note:  
The results of the analysis shown were developed based on available subsurface information from a limited number of exploratory locations, with the assumption that the materials exposed fairly represent the subsurface conditions within the areas explored. No guarantee can be made regarding the subsurface conditions between the explored locations, and such unknown conditions could have an impact on the results of the analyses.

Factor of Safety: 1.5  
Center: (363.4, 842.809) ft  
Radius: 42.693 ft  
Minimum Slip Surface Depth: 10 ft

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<th>Saturated Unit Weight</th>
<th>Cohesion</th>
<th>Friction Angle</th>
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<td>Ash Dike</td>
<td>100</td>
<td>105</td>
<td>0</td>
<td>33</td>
</tr>
<tr>
<td>Silty Sand to Sandy Silt (Alumina)</td>
<td>106</td>
<td>107</td>
<td>0</td>
<td>29</td>
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<tr>
<td>Lean Clay (Mixed)</td>
<td>123</td>
<td>123</td>
<td>0</td>
<td>31</td>
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<tr>
<td>Sil (Alumina)</td>
<td>109</td>
<td>109</td>
<td>0</td>
<td>28</td>
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<tr>
<td>Stabilized Fly Ash</td>
<td>100</td>
<td>105</td>
<td>0</td>
<td>25</td>
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<tr>
<td>Rip-Rap</td>
<td>115</td>
<td>115</td>
<td>0</td>
<td>40</td>
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Proposed Buttress and Gypsum Disposal Area 2A Pool raised to Elevation 830'

Figure 11. Slope Stability Cross-Section K-K' – Proposed Buttress
Slope Stability

Cross-Section L-L'  
Gypsum Disposal Area 2A

Bull Run Fossil Plant
Tennessee Valley Authority

Figure 12. Slope Stability Cross-Section L-L' – Proposed Buttress

Factor of Safety: 1.6
Center: (361.616, 843.444) ft
Radius: 46.343 ft
Minimum Slip Surface Depth: 10 ft

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<td>39</td>
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<td>Stained Fly Ash</td>
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<tr>
<td>Ash Dike</td>
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<td>0</td>
<td>33</td>
</tr>
<tr>
<td>Rip-Rap</td>
<td>115</td>
<td>0</td>
<td>40</td>
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Note:
The results of the analysis shown were developed based on available subsurface information from a limited number of exploratory locations, with the assumption that the materials exposed typically represent the subsurface conditions within the area explored. No guarantee can be made regarding the subsurface conditions between the explored locations, and such unknown conditions could have an impact on the results of the analyses.

Proposed Buttress with Gypsum Disposal Area Pool raised to Elevation 830'
**SEEP/W Analysis**

Cross-Section K-K’
Gypsum Disposal Area 2A

Bull Run Fossil Plant
Tennessee Valley Authority

February 2010
Method: Steady State Seepage
File Name: IC.css

Note:
The results of the analysis shown were
developed based on available subsurface
information from a limited number of exploratory
locations, with the assumption that the materials
exposed fairly represent the subsurface conditions
within the areas explored. No guarantee can be made
regarding the subsurface conditions between the
explored locations, and such unknown conditions
could have an impact on the results of the analyses.

<table>
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<th>Material Type</th>
<th>Kaiz (ft/s)</th>
<th>Kratio</th>
<th>Waist (ft/L/s)</th>
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<tr>
<td>Ash Ortholite</td>
<td>6.5e-008</td>
<td>0.04</td>
<td>0.48</td>
</tr>
<tr>
<td>Silt Sand to Sandy Silt (Aluminum)</td>
<td>6.95e-005</td>
<td>0.05</td>
<td>0.39</td>
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<tr>
<td>Lean Clay (Aluminum)</td>
<td>4.93e-007</td>
<td>0.05</td>
<td>0.41</td>
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<tr>
<td>Silt (Aluminum)</td>
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<td>0.05</td>
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<td>Slushed Fly Ash</td>
<td>1.607e-009</td>
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<tr>
<td>Rip-Rap</td>
<td>0.01</td>
<td>1</td>
<td>0.4</td>
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**Piping Potential**
Maximum Elevation Gradient occurs at (477.08, 804.66)
Total Head = 805.5 ft
At (476.90, 800.1)
Total Head = 806.85 ft
dh = 0.35 ft dl = 4.56 ft
i = 0.077 (Kicritical) = 0.708
FSpiping = 9.2

**Figure 13.** SEEP/W Analysis Cross-Section K-K’ – Proposed Buttress
Figure 14. SEEP/W Analysis Cross-Section L-L’ – Proposed Buttress

Note: The results of the analysis shown were developed based on available surcharge information from a limited number of exploratory locations, with the assumption that the materials exist true to represent the subsurface conditions within the areas explored. No guarantee can be made regarding the subsurface conditions between the explored locations; and such unknown conditions could have an impact on the results of the analysis.

Maximum Exit Gradient occurs at (475,14,755.12)
Total Head = 805.50 ft
At (475,87,759.51)
Total Head = -807.00 ft
dH = 0.5 ft
dL = 5.61 ft
i = 0.089 (critical) = 0.708
Fanning = 8.0

Material Type    ksat    ktrv   West
Leach Clay ( SWAT )  4.330e-06   0.05   0.41
Shocked Fly Ash  1.0e-09     0.02   0.46
Ash Dome       0.14e-06   0.04   0.46
Rip-Rap         0.01      1      0.4

Proposed Buttress with Gypsum Disposal Area raised to Elevation 830’
Figure 15. Buttress Plan
The reduced buttress height left the lower portion of the disturbed (sloughed) dike slope partially exposed. Consequently, the corrective design included armoring of the slope above the top of the buttress up to elevation 818 feet. The armoring consisted of removing the cap layer of clay and topsoil from the dike surface, placing a graded seepage filter over the CCR material used to build the dike, and placing a 12-inch thick surface layer of No. 2 stone over the graded filter. The plan view, typical cross sections and a detailed composition of the armoring is presented in Figure 16.
Figure 16. Slope Armoring Plan
Because of the weak foundation conditions, the design documents included a suggested construction sequence with emphasis on implementing the different activities at a considerably slower pace than when performed if working on firm ground. The primary purpose of these suggestions was to prevent pore pressure buildup within the sluiced ash deposits, which could result in excessive settlement or failure of the buttress. The suggested sequenced activities included the following:

i. Controlled dewatering of ash pond to water elevation 795 feet.
ii. Stabilizing of soft areas to create working surface
iii. Vegetation and clay cover removal along south slope receiving armoring.
iv. Excavation for armoring and buttressing
v. Stabilization of excavation bottom (to elevation 800 feet)
vi. Armoring of the slope
vii. Staged construction of buttress
viii. Surface Run-off ditch construction

The design included different methods to prepare the subgrade of the buttress for different areas and allowed modification of these methods during construction to address unforeseen conditions. Figure 17 shows the different details and areas where they would be applied. Low ground areas within the ash pond were to be filled first to establish a level subgrade surface across the entire footprint of the buttress. Once the level surface was attained, the design specified that a given layer of fill material needed to be placed across the entire buttress footprint before placing the next layer. This was to prevent instant differential settlement from developing between adjacent areas.
Figure 17. Subgrade Preparation Plan
5. CONSTRUCTION PHOTOS

The photos presented below depict several activities related to the buttress construction including subgrade preparation, adjustments to correct unforeseen conditions and equipment used to spread the different types of material.

Figures 18 through 21 show photos of the 2009 sloughing and toe failure along different locations of the dike slope.

Figure 18. Pre-Construction – Area 2A South Slope Toe Failure
Figure 19. Pre-Construction – Area 2A South Slope Toe Failure

Figure 20. Pre Construction – Area 2A South Slope Toe Failure
Figure 21. Pre-Construction – Area 2A South Slope Toe Failure

Figure 22 shows the preparation of the construction access road to the buttress area using Geogrid and crushed stone.

Most of the initial work had to be performed using an amphibious excavator that applied less ground pressure and would remain afloat while working on top of sluiced ash within...
the ash pond. Figures 23 through 26 show the use of the amphibious excavator working on top of sluiced ash deposits preparing the buttress subgrade.

Figure 23. Use of Amphibious Excavator to Prepare the Subgrade Working on Top of Sluiced Ash

Figure 24. Use of Amphibious Excavator to Prepare the Subgrade Working on Top of Sluiced Ash
Figure 25. Use of Amphibious Excavator to Prepare the Subgrade Working on Top of Sluiced Ash

Figure 26. Use of Amphibious Excavator to Prepare the Subgrade Working on Top of Sluiced Ash
Once an approximately level ground was prepared, fine crushed stone (No. 57) was spread in thin lifts to create a flat surface upon which a layer of Geogrid was laid. This was followed by placing a thicker layer of No. 57 stone using a small dozer. Figure 27 shows this operation.

![Figure 27. Spreading No. 57 stone over Geogrid using small Front End Loader](image)

As fill is spread over, the Geogrid tends to stretch and spread load over a larger subgrade area, eventually allowing the use of heavier equipment for subsequent placement and compaction, including riprap, as shown in Figures 28 and 29.
In isolated cases, saturated ash was observed migrating to the surface as layers of stone were being placed and compacted. These areas were reworked to make sure the
subgrade consistency was uniform throughout. Figures 30 through 33 show an area where this occurred and the work done to correct the problem.

Figure 30. Saturated ash migrating through the aggregate course at one location and breaking out onto the surface

Figure 31. Close-up view of saturated ash breaking through surface of the aggregate foundation pad
Figure 32. Saturated ash that migrated through aggregate foundation pad is removed.

Figure 33. A geogrid patch is placed over the location of ash removal and backfilled with clean aggregate.

Figures 34, 35 and 36 show overviews of the completed buttress in slope armoring.
Figure 34. Completed Buttress

Figure 35. Overview of the Completed Buttress
Figure 36. Aerial View of the completed south slope buttress
6. CONCLUSIONS

Storing CCR material on top of ash ponds has not been an uncommon practice in the fossil power industry. There are numerous sites where dry CCR material has been stored successfully on top of saturated fly ash deposits contained in ponds constructed for this purpose. The Area 2A dikes of this project were constructed either on top of dry ash placed atop sluiced ash deposits or directly on top of sluiced ash deposits. After implementing certain measures to modify the foundation conditions, the construction of the dikes was completed successfully except for the sloughing observed along the toe of the south dike. All indications are that the sloughing was caused by uncontrolled seepage through the south dike and not due to a foundation failure.

The challenge in this case was to prevent the sloughing from developing into a deep-seated failure of the dike. Because Area 2A was in active use, the options to implement corrective measures were limited. Constructing a rock buttress in front of the dike was considered a viable alternative as long as measures were taken to reduce pore pressure buildup within the saturated ash during subgrade preparation and fill placement. The design included different subgrade preparation details to provide the contractor with options that could be selected for implementation depending on the actual sluiced ash consistency encountered within the foundation footprint of the buttress. The design also specified a slow rate of construction to prevent rapid pore pressure buildup within the sluiced ash. This required the contractor to place material across the entire buttress footprint one layer of fill at a time. TVA also allowed daily monitoring of all construction activities by the engineering design team. Readings obtained since the end of construction from instrumentation installed before and after the project was completed show no subsurface movement and normal seasonal changes in piezometric levels, indicating that the rock buttress remains stable.