ATTACHMENT 46
Closure Cap Risk Analysis Study
Special Waste Landfill Permit
Big Sandy Plant – Ash Pond Closure
Lawrence County, Kentucky

The existing facility proposed for closure was designed and operated as surface water dam. Since its commissioning in 1970, it is currently still regulated and monitored under an individual KPDES permit, with a compliant operating history.

On June 21, 2010, the United States Environmental Protection Agency (US EPA) announced proposed rules for the final disposition of coal combustion by-products. The proposed rules have not been promulgated by US EPA, however; KDEP has requested closure of the impoundment as a special waste landfill in anticipation of their promulgation.

An analysis of the proposed closure cap is attached and prepared in accordance with the Kentucky Administrative Regulations (KAR), Energy and Environment Cabinet, Department for Environmental Protection (Title 401), Technical and Operating Requirements for Special Waste Landfills (401 KAR 45:110). The purpose of the analysis is to describe how the proposed closure cap system will meet the environmental performance standards of 401 KAR 30:031, Sections 4, 5, and 6 concerning surface and groundwater. The attached cap risk analysis will address each of the factors listed in 401 KAR 45:110, Sectin 5, as applicable.

KENTUCKY POWER

BIG SANDY PLANT

LAWRENCE COUNTY, KENTUCKY

COAL ASH POND CLOSURE
CLOSURE CAP RISK ANALYSIS STUDY

Prepared for

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Appendix A: Stability Calculation Set

1.0 INTRODUCTION

This Closure Cap Risk Analysis (Cap RA) has been prepared in accordance with the Kentucky Administrative Regulations (KAR), Energy and Environment Cabinet, Department for Environmental Protection (Title 401), Technical and Operating Requirements for special waste landfills (401 KAR 45:110). The purpose of this document is to describe how the proposed closure cap system will meet the environmental performance standards of 401 KAR 30:031, Sections 4, 5, and 6 concerning surface and groundwater. This document will address each of the factors listed in 401 KAR 45:110, Section 5, as applicable. This Closure Cap Risk Analysis has been developed in support of the coal ash pond closure at Kentucky Power's Big Sandy Plant in Lawrence County, Kentucky.

2.0 NARRATIVE DESCRIPTION OF THE FACILITY AND OPERATION

2.1 SITE LOCATION AND DESCRIPTION

Kentucky Power Company (KPCo), a wholly owned subsidiary of American Electric Power (AEP), owns and operates the 1,097 Mega Watt (MW) Big Sandy Plant on the west bank of the Big Sandy River, near Louisa in Lawrence County. Currently, coal combustion fly ash from the plant is disposed of in the Big Sandy Fly Ash reservoir, which is impounded by the Horseford Creek Dam located approximately 0.75 miles northwest of the plant. In expectation of future Federal Regulations pertaining to wet ash impoundments, the Project involves closure design of the Plant's existing 140-acre wet fly ash impoundment, which will no longer be needed for wet sluice disposal beginning in 2016. In an effort to effectively close the fly ash reservoir in accordance with expected but not-yet-promulgated Federal Regulations for wet CCP impoundments, it is AEP's desire to permanently close the facility by draining and capping the Big Sandy Fly Ash Pond.

2.2 DOCUMENT FORMAT

Information presented in this document has been organized and presented consistent with the requirements presented in 401 KAR 30:031, Sections 4, 5, and 6 and 401 KAR 45:110, Section 5. Sections within this document have been titled and enumerated consistent with the regulations to facilitate the review process. The following *italicized text* is copied verbatim from the aforementioned regulations. For clarity of discussion, each response is provided in **bold text**.

3.0 ENVIRONMENTAL PERFORMANCE STANDARDS

3.1 SURFACE WATERS

401 KAR 30:031; Section 4 (1) – *No waste site or facility shall:*

(1) Cause a discharge of pollutants into waters of the Commonwealth, including wetlands, that violate any requirements of KRS Chapter 224, or the surface water standards of 401 KAR Chapter10 or 8; or

(2) *Cause a discharge of dredge material or fill material to waters of the Commonwealth that is in violation of the requirements under 33 USC 1251 et. seq. (Section 404 of the Clean Water Act of 1977 as amended).*

As stated in Section 2 of this document, the facility is an existing wet fly ash impoundment. It is designed and operates as a surface water dam, preventing water from discharging into adjacent properties. The design allows for surface drainage to be redirected away from the Horseford Creek Dam (or Main Dam) to the Saddle Dam to decrease the drainage area to the significant hazard dam and direct water through the Saddle dam, which is a low hazard dam. The facility is currently being monitored under an Individual KPDES permit with prescribed effluent limitations for associated outfalls.

The Closure Cap System proposed includes a stabilized vegetative layer above a barrier layer. A healthy vegetated cap comprised of grass totaling 90 percent cover provides approximately 99 percent reduction of erosion, according to the Kentucky Erosion Prevention and Sediment Control Field Guide. The facility's Storm Water Pollution Prevention Plan (SWPPP) will be modified to ensure adequate controls for storm water discharges during construction activities are implemented and the facility meets the requirements of its KPDES permit.

Risk of impacting the surface water after final closure is minimal. For example, administrative and regulatory restrictions, already in place are reflected in the closure cap design. Additionally, a minimal profile slope (0.50%, typical) is used as part of the design to minimize flow velocity and scour. The use of turf reinforcement and rock dams is also prescribed in the design to promote vegetative growth and energy dissipation, respectively.

A surface water delineation was performed for the facility in 2012 and was submitted to the United States Army Corps of Engineers, Louisville District for a jurisdictional determination in 2013. In the event construction will result in a discharge of dredge or fill materials into waters of the United States, an application for a Section 404/10 permit and 401 Water Quality Certification will be submitted to the Corps and KDEP.

3.2 GROUNDWATER

401 KAR 30:031; Section 5 – *Groundwater. No waste site or facility shall contaminate an underground drinking water source beyond the point of compliance in excess of the maximum contaminant levels specified in 401 KAR Chapter 8.*

The facility was designed as a surface water dam and operated under KDEP, Division of Water regulations. A liner system was not required as part of initial construction of the facility. A Hydrogeologic Site Investigation (HSI) was performed for the facility and the report has been included in the Special Waste Landfill Permit Application as part of Appendix 30. Based on the HSI, the subsurface soil permeability varies throughout the facility, including sand, clay, silt and rock layers. A clay cut-off wall was incorporated into the dam during its original construction and subsequent raisings which impedes groundwater movement. Groundwater generally follows existing surface topography towards the Horseford Dam (down valley), roughly

mimicking the current process water flow. The basis of the dam is to restrict and retain water (surface and subsurface) from flowing from the facility.

The proposed closure cap is comprised of the following layers from the bottom to the top:

- **Contouring Fill – consisting of bottom ash and/or structural clay fill at varying thickness to the appropriate grade. Bottom ash material is generated as a byproduct of coal combustion power generation at the Big Sandy Plant;**
- **Low permeability Layer – consisting of either 18 inches of compacted clay or flexible membrane liner (FML). Clay is to be compacted to Proctor values to meet a maximum permeability of 1 × 10^{-5} cm/sec based on laboratory testing;**
- **Drainage Layer (where needed) – consisting of a geocomposite drainage layer;**
- **Protective Cover – Low permeability layer consisting of 18 inches of compacted clay with 6 inches of soil cover capable of supporting vegetation. Areas where the FML will be utilized will have 24 inches of protective cover with 6 inches capable of supporting vegetation.**

The installation of the closure cap system is designed to restrict water from percolating into the coal combustion product (CCP) mass by providing an engineered barrier for storm water.

3.3 LAND USE FOR FOOD CHAIN CROPS

401 KAR 30:031; Section 6 – *Application to Land Use for the Production of Food Chain Crops. No waste site or facility shall exist or occur that applies waste within three (3) feet of the surface of land used for production of food chain crops unless in compliance with all the requirements of (1) or (2) of this section.*

No food chain crops are anticipated to be produced on the facility after closure. Additionally, a notice indicating the property was used to store waste material will be recorded in the property deed.

4.0 CLOSURE AND POSTCLOSURE REQUIREMENTS

401 KAR 45:110; Section 5(2) – *A closure plan shall have a closure design prepared to specify the function and design of the final cover on the special waste landfill considering:*

(a) The type and amount of waste in the facility;
(b) The mobility and expected rates of migration of the waste or leachate constituents;
(c) The site location, topography, surrounding land use, and final site use;
(d) The climatic conditions in the area;
(e) The characteristics of the cover material including its chemical and physical composition, erodibility, slope stability, final surface contours, thickness, porosity, permeability, slope, length of run of slope, and type of vegetation on the cover; and
(f) The geology and soil profiles and surface and subsurface hydrology of the site.

4.1 TYPE AND AMOUNT OF WASTE IN THE FACILITY

The wet fly ash pond was designed and commissioned in 1969. The original facility covered approximately 97.1 acres, and had modifications constructed as recent as 1992. The facility covers approximately 130 acres of area. As of 2009, the total storage capacity of the facility is approximately 13.4 Myd^3.

4.2 MOBILITY AND EXPECTED RATES OF MIGRATION

As part of the Hydrogeologic Site Investigation (HSI) (provided in Attachment 30), seven monitoring wells were completed for hydraulic testing. Three wells were completed in sandstone units, one in the sandy clay alluvium, and three in various locations on the property. The resultant rate of groundwater flow at the facility is calculated to be 0.036 feet per day. As previously noted, a liner system was not required. The groundwater at and around the facility follows the surface topography, flowing into the facility and towards the Horseford Creek Dam.

The material in the facility consists of wet fly ash produced from the Big Sandy Plant. Typical chemical constituent data for fly ash are publically available through the US Environmental Protection Agency (US EPA) as EPA-HQ-RCRA-2009-0640-6300 or Electric Power Research Institute (EPRI) as EPRI Report 1012578, as part of the proposed rule published in the Federal Register on June 21, 2010. The report utilized 81 field leachate samples collected at 29 management sites thereby providing a large cross-section of data for various coal sources and the by-products from various air pollution controls. These reports generally indicate that these materials are non-toxic with low levels of inorganic constituents near or slightly above the MCL drinking water standard. In addition, the reports state that high readings from TCLP testing are not indicative of anticipated exposure levels in groundwater.

Site specific samples of the groundwater were taken and tested in 2012. Samples were taken from monitoring wells onsite and analyzed for the metals listed in 401 KAR 45:160, Section 7 (2). Arsenic was the only dissolved metal with reported concentrations above the MCL in the monitoring wells during each of the sampling events. The metal was found in the up-gradient wells MW-1010 and MW-1011, and the alluvial deposit well MW-1206. The results of the testing can be found in Tables 4.4a through 4.4d of the HSI. The cessation of the sluicing operation and the installation of the closure cap are expected slow the rate of recharge through the material mass, thereby further reducing migration from the facility.

4.3 SITE LOCATION, TOPOGRAPHY, AND LAND USE

The facility is the result of damming the valley of Horseford Creek, located in Lawrence County, Kentucky. Approximately 30 acres of the facility is inundated with water. Depths of surface water within the facility have been reported to be up to 42 feet. Process water outfalls from the facility discharges into Blaine Creek, which, in turn, discharges into the Big Sandy River.

Surrounding property is owned by AEP and is generally undeveloped. Access into the facility is limited by natural boundaries and gated access roads. Nearby facilities include an asphalt

manufacturing facility located south of the site in the adjacent Burke Branch valley and the power plant located approximately 3,000 feet to the southeast of the reservoir. The proposed closure cap system is designed to help prevent unintended contact with waste materials by anyone who intentionally or unintentionally accessed the site without the permission of AEP.

The closure cap is designed to provide a barrier from surface waters from infiltrating into groundwater through the material mass. The closure cap will force surface water to shed away from, and to avoid contact with the CCP material. Additionally, groundwater already isolated within the material mass may be treated prior to release, as required for any liquid that has percolated through or drained from waste.

4.4 CLIMATIC CONDITIONS

Historic climatic conditions were considered for the grading design of the closure cap. Precipitation data for the area was obtained from the National Oceanic and Atmospheric Administration (NOAA) and was incorporated into the storm water design for the cap grading. The analysis and calculations associated with the proposed storm water controls can be found in the Special Waste Landfill Permit Application as Attachment 23. The cap system proposed is designed to function with the anticipated climatic conditions of the site.

4.5 COVER MATERIAL

The cover material used for the closure cap is discussed in Section 3.2 of this document. The permeability of the cover material was selected based upon the permeability of the facility's underlying soil. As indicated above, the HSI resulted in subsurface materials with variable permeability. However, observations also indicate that groundwater followed the surface topography and the surface water ultimately flows towards the Horseford Creek Dam. The wells found to have the highest hydraulic conductivity ranged from 10^{-3} to 10^{-5} cm/sec.

Slope stability analyses of the critical areas, i.e., Horseford Creek and Saddle Dams, were analyzed to set baseline requirements. The analyses were performed following guidance provided in the US Corps of Engineers' (USCOE) EM-1110-2-1902 "*Slope Stability*" and Ohio Environmental Protection Agency's (OEPA) "*Geotechnical and Stability Analyses for Ohio Waste Containment Facilities*" as no other guidance is available from KDEP. The OEPA guidance utilizes infinite slope analysis, which is highly conservative. The critical sections are analyzed for deep-seated stability, shallow translational stability (i.e. cap system stability), and seismic stability.

The Kentucky Dam Safety permit allows for the storage of CCP materials to an elevation of 705 ft. msl at the Horseford Creek Dam. The final elevation of the CCP material at closure is highly dependent on the amount of coal burned between now and the time of closure. This is directly related to electricity demand, balancing of loads with other regional power plants, and the ash content of the coal burned. All of these factors will vary considerably throughout the remaining life of the Big Sandy Plant. The closure grades presented in Attachment 20 are based on estimates of the amount of ash to be produced between the most recent survey date

and the date of closure. As such, the grades depicted are intended to represent the general grading scheme at closure. Actual closure grades for portions of the facility footprint or the entire footprint may be raised or lowered 2 to 10 feet, as needed. It is intended that the majority of the site will have a typical grade of approximately 2%, a minimum grade of 0.5%, and a maximum cap grade of approximately 25%. These proposed grades, along with the proposed vegetation plan and planned storm water channel reinforcement are anticipated to provide a stable, low maintenance cap system for the closure of the ash pond.

4.6 SITE GEOLOGY/HYDROGEOLOGY

As discussed above, the site geology and hydrogeology are discussed in Attachment 30, the site is underlain by variable geology. Existing groundwater flows indicate that the groundwater at the site follows the existing grade toward the Horseford Creek Dam. The proposed groundwater monitoring system will monitor the site throughout the post-closure period. The cessation of sluicing and the installation of the proposed cap system is anticipated to decrease the mobility of constituents in the groundwater at the site.

APPENDIX A

STABILITY CALCULATION SET

AEP BIG SANDY POWER PLANT
ASH POND CLOSURE

GEOTECHNICAL CALCULATIONS SUPPORTING CLOSURE DESIGN

I. Introduction

This calculation package includes URS's geotechnical analyses performed in support of the design for the Ash Pond Closure project (drawings have been submitted under separate cover). Section II below provides a table of contents of the calculations that have been performed to date and are included herein, and Section III below provides a brief summary of results obtained for each of the calculations.

II. List of Calculations

I. **Cap Constructability/ Bearing Capacity Analyses** - Bearing capacity was evaluated for three cases for construction of the cap namely; 1) Geomembrane and cap system placed directly on sluiced fly ash; 2) Improvement of cap subgrade by undercut and placement of 2 feet of bottom ash; and 3) Improvement of cap subgrade by consolidation via placement of a pre-loading surcharge prior to cap construction.

II. **Settlement Analysis** – Analysis of total and differential settlement for key project elements was performed. The consolidation settlement evaluation included determination of total and differential settlement of the proposed pore water drain as a result of consolidation of the sluiced fly ash and the underlying native alluvium soils under the surcharge of new borrow fill placed over the pond as part of the closure. In addition, the strain developed in the cap system due to settlement across a critical cross-section was analyzed.

III. **Closure Cap Deep-Seated Slope Stability Analysis** – The majority of fills placed to shape grades for the closure cap will feature very shallow slopes, and global slope stability for most of the cap system should be adequate by inspection. The separator berm proposed to be installed north of the saddle dam, separating the upper pond from the lower pond represents the only permanent fill that has an appreciable slope. Deep-seated static and seismic slope stability (the latter evaluated using pseudostatic methods) was evaluated for this critical area of the cap system. Static analyses were performed for both long-term drained conditions and short term undrained conditions.

IV. **Closure Cap Shallow Translational Slope Stability Analysis** - Shallow translational slope stability was analyzed for the cover system by evaluating the veneer stability of the final cover system proposed for the facility. There are no prescriptive standards related to slope stability in the Kentucky Special Waste Solid Waste Regulations, however this calculation check was performed for completeness.

V. **Seismic Liquefaction Analysis** – Analysis to determine liquefaction potential of the very loose/soft sluiced fly ash that underlies the project was performed.

VI. **Hydrostatic Uplift Analysis –** The only significant area with the potential for cap system uplift is the storm water channel, which is excavated below the current fly ash surface elevation. When the cap system is placed above this area, there is potential for elevated water levels in the ash subgrade to apply uplift pressures to the cap system. Hydrostatic Uplift Analysis - The potential for development of hydrostatic uplift was addressed by designing a porewater drainage system to manage pore water inflow into the excavated channel area and to provide a means to lower the pore water elevation to below the cap system in the long-term. This system is designed to incorporate a 2 foot thick bottom ash layer throughout the channel area which drains to two 6-inch diameter perforated pipes. These pipes will convey the collected pore water to a pump station which will convey the water to a leachate storage lagoon. In addition, the pore water levels adjacent to this storm water channel area will be monitored during construction by a series of piezometers to be installed parallel to the channel. These piezometers will help identify if any adjustments to the pore water management system are necessary based on the rate of dissipation of the pore water level in the ash. Therefore, formal calculations for hydrostatic uplift were deemed as not required.

VII. **Transmissivity of the Final Cover System Geocomposite** – Analysis was performed to calculate the required transmissivity of the final cover system geocomposite.

VIII. **Main Dam Stability Analysis –** Slope stability analysis of the Main Dam, in its proposed lowered configuration (Note: This analysis has been submitted to AEP separately and is not re-attached herein.

III. Summary Results of Analyses

The supporting calculations are attached to this document, the purpose of this section is to present a brief summary of analyses only. The summary results of analyses are as follows:

I. **Cap Constructability/ Bearing Capacity Analyses** - The analyses indicated that placement heavy equipment directly on (unimproved) sluiced ash may result in bearing failure of the ash. To achieve a minimum factor of safety of 1.3 against bearing capacity failure, an undercut of 2.75 feet and replacement with bottom ash, or placement of a temporary surcharge of 7.5 feet of fill placed for an approximately a six month pre-loading duration (to allow consolidation and strength gain in the upper areas of the sluiced ash) are feasible solutions to allow construction of the cap on fly ash subgrade.

II. **Settlement Analysis –** The settlement analysis indicates that up to 8-inches of settlement may occur under the proposed drainage channel and pore water drain piping, with the highest settlements occurring mainly in the far western end of the channel alignment. The magnitude of the change in slope (both positive and negative) is generally not large and is anticipated to be accommodated without significant loss of discharge capacity and without inducing significant strain in the proposed HDPE piping. Furthermore, the maximum estimated strain in the geosynthetic components of the cap (post-settlement) are roughly 0.1%, which are well below allowable values.

III. **Deep-Seated Slope Stability Analysis –** Results of the slope stability analysis indicate factors of safety against slope stability of the separation berm exceed guidance values given in the U.S. Corps of Engineers' EM-1110-2-1902 "Slope Stability" for all cases considered.

IV. **Shallow Translational Slope Stability Analysis -** Using the methodology outlined in Koerner and Soong (2005), the analysis consisted of finding minimum strength parameters (friction angle Φ with assumed cohesion c=0) for a variety of conditions listed below with the corresponding minimum required factor of safety against translational failure for that particular condition

- Static Conditions (Peak Strength): FS ≥ 1.50
- Static Conditions (Residual Strength): FS ≥ 1.10
- Static Conditions (Full Drainage Layer): FS ≥ 1.10
- Static Conditions (Equipment Loads): FS ≥ 1.25
- Seismic Conditions: FS ≥ 1.00

Results of the shallow translational slope stability analysis indicate a minimum required peak interface friction angle of 17.8 degrees and a residual interface friction angle of 12.4 degrees.

V. **Seismic Liquefaction Analysis –** Under the seismic action of a 7.5 magnitude earthquake with a return period of approximately 2,500 years, based on a design earthquake event of 2% probability of exceedence in 50 years and site bedrock to consist of NEHRP Class B material, the analysis indicates minimum factors of safety against dynamic liquefaction in the range of 1.3 to 1.8 for the fly ash within 7.5 feet of the surface. Minimum factors of safety for fly ash below 7.5 feet increase to a range of 1.4 to 2.0. Application of the cap system soils is anticipated to improve the factors of safety for the fly ash within 7.5 feet of the surface. It is generally accepted that no liquefaction should be anticipated for materials that exhibit factors of safety greater than or equal to 1.4 under seismic activity. Therefore, liquefaction of the ash under the design earthquake is not anticipated.

VI. **Transmissivity of the Final Cover System Geocomposite –** The minimum transmissivity required to maintain drainage inside the geocomposite on the 4H:1V slopes is 6.22×10^{-4} m^2/sec and represents the minimum required value for the testing and manufacturer's specifications, (T_{spec}).

VII. **Main Dam Stability Analysis –** Slope stability analysis of the Main Dam, in its proposed lowered configuration indicates that all factors of safety meet or exceed typical guidance provided by the U.S. Army Corps of Engineers (as given in EM-1110-2-1902 "Slope Stability").

CAP CONSTRUCTABILITY/BEARING CAPACITY ANALYSIS

BIG SANDY POWER PLANT ASH POND CLOSURE PROJECT

GEOTECHNICAL CALCULATIONS

CAP CONSTRUCTION - BEARING ANALYSIS

- Current analysis is to estimate bearing capacity / stability for the placement of cover material (~ 2' thick) & a geomembrane liner over the prepared subgrade.
- In certain areas, especially Phase 3 portions near the saddle dam; less than 2 ft of cut/fill is required to prepare the subgrade thereby indicating that the cap/cover material may be placed directly on the sluiced flyash that forms the bottom subgrade in these areas. This represents the worst-case conditions as other areas needing larger cut/fill may have backfilled bottom ash (better subgrade) acting as the subgrade supporting the cap. The current calculation provides an estimate of bearing capacity of the subgrade layer to provide support and enable placement of cap/liner.
- To place cap/liner estimate a medium-grade crawler track-mounted dozer with a long-reach attachment is used. Assume a model such as a John DEERE 850K crawler dozer is used for excavating minor depths in subgrade/placement of cap

Per the attached spec. sheet for Deere 850 K dozer, (see sheet 9 of 16 attached); the undercarriage details indicate

Ground contact pressure = 8.13 psi = 8.13 × 144

= 1172.16 say 1175

$q_{app} \sim 1200$ psf

Ground contact area = 5233 in^2 = $\frac{5233}{144}$ ft^2 = 36.34 ft

Length of track on ground = 9 ft 1 in ~ 9 ft

∴ Width B of contact area = $\frac{36.34}{9.085}$ = 4 ft

NOTE :- In reality width 4' is actually composed of two sets of crawlers (each 2' wide) separated by a distance of ~ 8' width (width between the 2 crawlers) However, assuming concentrated width of 4' without any gap in between is conservative as it represents worst design case

∴ Width B of fdn = 4 ft

Length L of fdn = 9 ft

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		Sheet 3 of 16
Description	Computed by	Date
	Checked by	Date

Reference

Placement of cap/geomembrane will be performed using the dozer considered above. The cap/geomembrane will be placed directly on the fluidized bottom ash (worst case scenario without any cut/fill) or may be placed on a certain thickness of bottom ash layer used to achieve subgrade after a minimum 2-foot cut/fill especially in Phase 3 of Construction near the saddle dam.

To obtain engineering/design parameters for each layer use results for lab testing as summarized in the geotechnical report for the project dated November 2012

FLUIDIZED FLY ASH :- Based on lab results in App. B.2 (see sheet 10 of 16 attached)

Total Stress drained parameters $\phi = 15.5^\circ$ $c = 1.4$ psi $= 160$ psf

For bearing analysis using drained conditions, neglect c'

$\therefore$ Use $\phi = 15.5^\circ$ $\gamma_t = 107$ pcf

BOTTOM ASH :- Based on parameters presented in slope stability analyses dated Dec 2012; effective strength parameters are $\gamma_t = 100$ pcf and $\phi = 32^\circ$, $c = 0$ psf. Considering that placement of bottom ash will be in loose lifts (limited compaction possible on fluidized fly ash subgrade) use $\phi = 26°$ (conservative) considering bottom ash to be rapidly draining, use total & effective parameters as same

CAP MATERIAL:- Assume silty clay material used

for cap $\gamma_t = 130$ pcf $\phi = 26°$ $c = 0$

NOTE:- To be conservative, assume cap material & bottom ash material average properties for bearing analyses

∴ Use $\gamma_{avg} = 115$ pcf (100-130 pcf) and $\phi = 26°$ for bearing analysis. Consider two possible cases

CASE I:- NO BOTTOM ASH FILL PLACED / CAP PLACED DIRECTLY ON FLUIDIZED FLY ASH.

For this case; use Meyerhoff Bearing Capacity Analysis

$$q_{ult} = \cancel{c}^{\,0} N_c s_c d_c + \cancel{q}^{\,0} N_q s_q d_q + 0.5 \gamma B N_\gamma s_\gamma d_\gamma$$

For CASE 1; $c = 0$ $q' = 0$ since D = depth of embedment of footing $= 0$

$$q_{ult} = 0.5 \gamma B N_\gamma s_\gamma d_\gamma$$

An in-house spreadsheet was created to calculate q_{ult} based on the above equation (See sheet 11 of 16)

For CASE 1; bottom mat'l = fluidized fly ash $\phi = 15.5$, $c = 0$, $\gamma = 107$ pcf, $B = 4$ ft, $L = 9$ ft $q_{ult} = 287$ psf

q_{ult} = 287 psf << contact pressure = 1200 psf

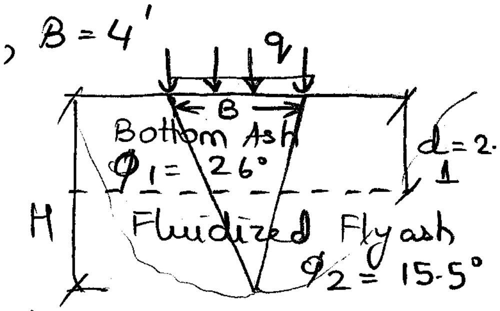
⇒ Cap material cannot be placed directly on fluidized flyash due to inadequate bearing capacity.

∴ Consider CASE II where undercut is performed for a depth of 2 feet and bottom ash is placed in the undercut area.

CASE II :- <u>Using 2' of undercut, replaced fluidized flyash with bottom ash subgrade</u>

This scenario consists of a two-layer analysis as described in "Foundation Analysis & Design" - Joseph Bowles 5th Ed. Ch-4, Sec 4.8 pp 251-255 (attached sheets 12-16/16)

Considering $\phi - c$ soils with $c = 0$, $B = 4'$



Distance d_1 = depth of bottom ash
= 2.0'

H = ht. of failure wedge = $0.5 B \tan\left(45 + \frac{\phi_1}{2}\right)$

where $\phi_1 = \phi$ for top layer (bottom ash) = 26°

$\therefore H = 0.5 * 4.0 * \tan\left(45 + \frac{26}{2}\right) = 3.2$ ft

$H > d_1 \Rightarrow$ modified $\phi = \dfrac{d_1 \phi_1 + (H - d_1)\phi_2}{H}$

(i.e. modify ϕ of top layer to obtain a weighted value of ϕ depending on contribution of lower, weaker ϕ based on amount of failure wedge in the weaker zone)

$$\therefore \text{mod } \phi = \frac{(2.0)(26^\circ) + (3.2 - 2.0)(15.5^\circ)}{3.2} \approx 22^\circ$$

Using spreadsheet $\phi_{mod} = 22^\circ \Rightarrow q_{ult} = 1027$ psf

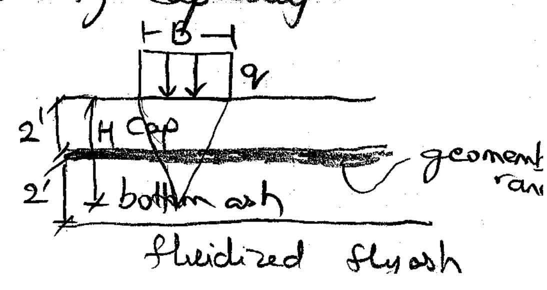
<u>marginal</u> for $q_{app} = 1200$ psf.

<u>NOTE</u>:- In reality, width $B = 4$ ft is composed of two tracks of width = 2 ft separated by a finite distance. Reduced width $\Rightarrow$ $H = 0.5 * B * 1.6$ $\Rightarrow$ H, height of failure wedge is reduced $\Rightarrow$ more of failure wedge is contained within stronger bottom ash layer with $\phi_1 = 26^\circ \Rightarrow$ mod. ϕ is larger $\Rightarrow q_u$ is higher, but only marginally as B is reduced while ϕ mod is higher.

Also, placement of cap mat'l will consist of a 4-5' high stockpile placed on certain stronger areas (access roads) with material being pushed/graded down to 2' $\Rightarrow$ bottom ash of 2' will be accompanied with 1-2' of cap as it is graded with dozer on top of the combined 2-4' thk layer.

NOTE:- Still consider FOS on bearing failure marginal and may have to use an observational approach & pay close attention during trial runs to ensure safety.

<u>For final condition with 2' cap material on top of geomembrane on top of 2' bottom ash subgrade followed by fluidized ash</u>, check available bearing capacity



For this scenario,

$d_1 = 4$ ft $\quad \phi_1 = 26^\circ \quad B = 4$ ft

$H = 0.5 * B * \tan(45 + \frac{26}{2}) = 0.5\, B * 1.6$

$= 0.8\, B = 3.2'$

Since $H = 3.2' < d_1 = 4'$; failure wedge is within the bottom ash zone $\Rightarrow \phi_{mod} = \phi_1 = 26°$

Using spreadsheet, for $\phi_{mod} = 26°$ $q_{ult} = 2{,}050$ psf

$q_{app} = 1200$ psf $\Rightarrow$ $F.S = \frac{2050}{1200} \approx 1.67$ OK

Check:- with 4' stockpile wt after spreading 2' cap; add'l surcharge remaining = 2 ft * 115 pcf = 230 psf $\Rightarrow q_{app} = 1200 + 230 = 1430$ psf

$\therefore FS = \frac{2050}{1430} \sim 1.4$ still acceptable

NOW, find minimum depth d_1 of bottom ash/cap reqd to obtain min FS = 1.3 for bearing

$q_{app} = 1200$ psf FS = 1.3 $\Rightarrow$ $q_{ult} = 1560$ psf reqd

Check with 2' of bottom ash and 1' thick first lift of cap mat'l

Thk = 3.0' $d_1 = 3.0$ H = 3.2 $\Rightarrow \phi_{mod} = \frac{3.0(26) + (3.2-3.0)(15}{3.2}$

$\therefore \phi_{mod} = 25.34°$

Using spreadsheet $\phi_{mod} = 25.3° \Rightarrow q_{ult} = 1830$ psf > 1560 psf reqd

$\therefore$ OK

$\therefore$ Minimum 3' of bottom ash + cap mat'l is required before track dozer can safely travel over it.

NOTE:- To obtain min $q_{ult} = 1560$ psf ; by trial and error; spreadsheet indicates ϕ_{mod} reqd = 24.5°

$\therefore$ Depth d_1 reqd to get $\phi_{mod} = 24.5°$ can be calculated as

$$\phi_{mod} = 24.5 = \frac{d_1(26) + (3.2 - d_1)\,15.5°}{3.2}$$

Solve for d_1 ⇒

$$24.5 = \frac{26 d_1 + 3.2 * 15.5 - 15.5 * d_1}{3.2}$$

$$\therefore (24.5 * 3.2) - (3.2 * 15.5) = (26 - 15.5)\,d_1$$

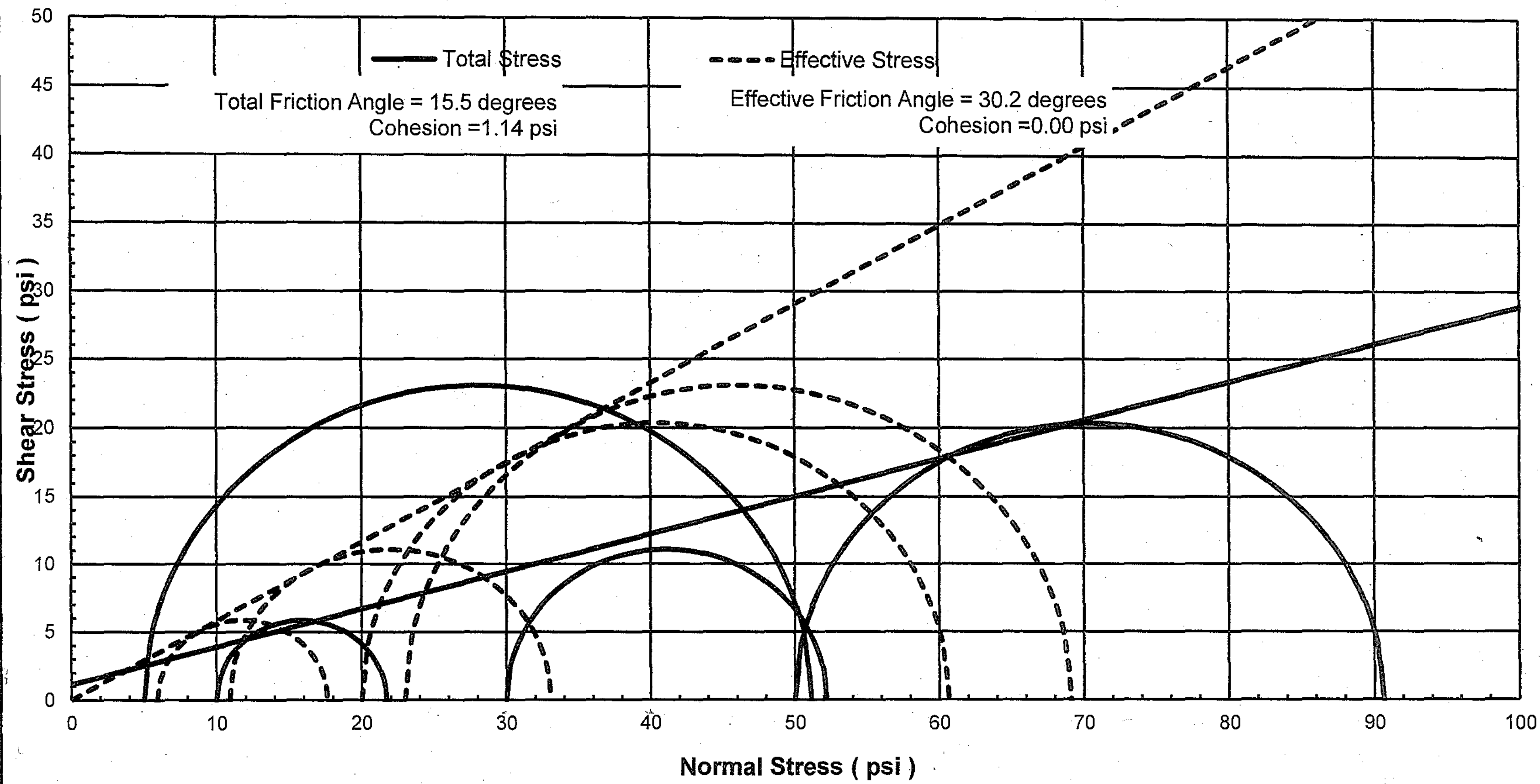
$$\therefore \quad d_1 = 2.75'$$

∴ Min depth required = 2.75' or 2'9" to achieve min FS = 1.3 for bearing analysis

Manufacturer	**John Deere**
Model	**850K**
DATE OF SPECIFICATION - 14 March 2012	
ENGINE	
Emission Rating	Interim Tier 4
Engine Make / Engine Model	John Deere / PowerTech PVX 6068
Net Power, kW (hp) @ Rated rpm	152 (205) @ 1800
Gross Power, kW (hp)	not published
Displacement, L (cu in.)	6.8 (415)
TRANSMISSION	
Type	Hydrostatic
Speed Range - forward	1
Speed Range - reverse	1
Maximum Speeds Forward, kph (mph)	9.7 (6)
Maximum Speeds Reverse, kph (mph)	9.7 (6)
FINAL DRIVES	
Type	Double Reduction
STEERING	
Type	No
HYDRAULIC SYSTEM	
Pump Type	Variable Displacement Piston
Pump Flow, L/min (gpm)	163 (43)
System pressure, kPa (psi)	24990 (3625)
UNDERCARRIAGE	
Track Gauge, mm (ft in)	1880 (6 ft 2 in)
Length of Track on Ground, mm (ft in)	2769 (9 ft 1 in)
Standard shoe type	Single Grouser - Moderate Service
Ground Contact Area, cm sq (sq in)	33760 (5233)
Ground Pressure, kPa (psi)	56 (8.13)
No. of Track Rollers	7
Track Pitch, mm (in)	203 (8)
Track Shoes, each side	40
Track Oscillation	Yes
DOZER	
Blade Width, mm (ft in)	3251 (10 ft 8 in)
Blade Height, mm (ft in)	1422 (4 ft 8 in)
SAE Blade Capacity, cu mt (cu yd)	5.6 (7.3)
Blade Lift Height, mm (ft in)	1151 (3 ft 9 in)
Digging Depth, mm (in)	599 (24)
Blade Width at Full Angle, mm (ft in)	not published
Blade Tilt, mm (ft in)	753 (2 ft 6 in)
DIMENSIONS	
Height w/ROPS, mm (ft in)	3161 (10 ft 5 in)
Length w/Dozer, mm (ft in)	not published
Ground Clearance, mm (in)	409 (16.1)
CAPACITIES	
Fuel Tank, L (gal)	356 (94)
Hydraulic Reservoir, L (gal)	106 (28)
WEIGHT	
SAE Operating Weight, kg (lb)	19295 (42538)

Specifications are based on published information at the time of publication.Specifications are subject to change without notice.

Sheet 10/16



Test at 5-psi effective consolidation stress excluded from Total Friction strength envelope

Project No. T13815141	AEP URS Corporation	Mohr Circles of Total and Effective Stresses at Peak CIU' Triaxial Test	Figure 2
TerraSense, LLC		Coarse Composite SUMMARY	June 2012

Exhibit 1

Ultimate Bearing Capacity Computation Using Myerhoff's Equations

Project: AEP Big Sandy, Louisa, Kentucky
Computed By: MSJ/VKG
Location: Bearing Analysis for cap construction

Input Information

Foundation Soil Properties	Input Set #1	Input Set #2	Input Set #3	Input Set #4	Input Set #5
Friction Angle ϕ (deg)	15.5	22	26	25.34	24.5
Cohesion c (psf)	0	0	0	0	0
Unit Weight γ (pcf)	107	115	115	115	115
Footing Width B (ft)	4	4	4	4	4
Footing Length L (ft)	9	9	9	9	9
Bearing Depth D (ft)	0	0	0	0	0
User input q'	0	0	0	0	0

Input Descriptions
Enter description of each calculation here

Input Set #	Description of Calculation
1	CASE I: no bottom ash, cap placed on fluidized fly ash, unit wt = 107 pcf.
2	CASE II: Use 2 ft of bottom ash in undercut, unit wt = 115 pcf, mod. Φ = 22
3	Option with 2 ft of bottom ash and 2 feet of cap placed (final condition)
4	Option with 2 ft of bottom ash and 1 feet of cap placed (intermediate condition)
5	Minimum depth of bottom ash and cap required to obtain approx. FS = 1.3

Calculations

Vertical Stress At Base (psf)	Set #1	Set #2	Set #3	Set #4	Set #5
$q' = \gamma D$ or user input value	0	0	0	0	0

Bearing Capacity Factors	Set #1	Set #2	Set #3	Set #4	Set #5
$N_q = e^{\pi \tan\phi} \tan^2\left(45 + \frac{\phi}{2}\right)$	4.13	7.82	11.85	11.05	10.12
$N_c = (N_q - 1)\cot\phi$	11.30	16.88	22.25	21.23	20.01
$N_\gamma = (N_q - 1)\tan(1.4\phi)$	1.25	4.07	8.00	7.16	6.22
$s_q = 1 + 0.1K_p \frac{B}{L}$ (=1 if ϕ<10)	1.08	1.10	1.11	1.11	1.11
$s_c = 1 + 0.2K_p \frac{B}{L}$	1.15	1.20	1.23	1.22	1.21
$s_\gamma = s_q$	1.08	1.10	1.11	1.11	1.11
$d_q = 1 + 0.1\sqrt{K_p}\frac{D}{B}$ (=1 if ϕ<10)	1.00	1.00	1.00	1.00	1.00
$d_c = 1 + 0.2\sqrt{K_p}\frac{D}{B}$	1.00	1.00	1.00	1.00	1.00
$d_\gamma = d_q$	1.00	1.00	1.00	1.00	1.00

Estimated Ultimate Bearing Capacity

$$q_{ult} = cN_c s_c d_c + q' N_q s_q d_q + 0.5\gamma B N_\gamma s_\gamma d_\gamma$$

	Set #1	Set #2	Set #3	Set #4	Set #5
q_{ult} =	287	1,027	2,050	1,830	1,584

JOSEPH E. BOWLES

FOUNDATION ANALYSIS AND DESIGN

FIFTH EDITION

3.5" HIGH DENSITY PC FORMAT DISK ENCLOSED

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Step 2. Obtain bearing-capacity factors for the Hansen equation using Tables 4-1 and 4-4. Do not compute ϕ_{ps}, since footing is square. For $\phi = 35°$ use program BEARING on your diskette and obtain

$$N_q = 33 \qquad N_\gamma = 34 \qquad 2\tan\phi\cdots = 0.255 \qquad \text{(also in Table 4-4)}$$

$$s_q = 1 + \frac{B'}{L'}\sin\phi = 1.57 \qquad s_\gamma = 1 - 0.4\frac{B'}{L'} = 0.6$$

$$d_q = 1 + 2\tan\cdots\frac{D}{B}$$

$$d_q = 1 + 0.255\frac{1.1}{2.5} = 1.11 \qquad d_\gamma = 1.10$$

From Table 4-1 and dropping any terms that are not used or are 1.0, we have

$$q_{ult} = \gamma D N_q s_q d_q + 0.5\gamma_e B' N_\gamma s_\gamma d_\gamma$$

Substituting values (note γ = soil above base), we see

$$q_{ult} = 1.1(18.10)(33)(1.57)(1.11) + 0.5(14.86)(2.5)(34)(0.6)(1.0)$$
$$= 1145 + 379 = 1524\ \text{kPa}$$
$$q_a = \frac{1524}{2} = \mathbf{762}\ \text{kPa (a very large bearing pressure)}$$

It is unlikely that this large a bearing pressure would be allowed—a possible maximum is 500 kPa (about 10 ksf). We might simply neglect the $\gamma_e B N_\gamma$ term to obtain q_a = 570 kPa (still large). If the latter term is neglected, the computations are considerably simplified; and doing so has little effect on what would normally be recommended as q_a (around 500 kPa in most cases).

////

4-8 BEARING CAPACITY FOR FOOTINGS ON LAYERED SOILS

It may be necessary to place footings on stratified deposits where the thickness of the top stratum from the base of the footing d_1 is less than the H distance computed as in Fig. 4-2. In this case the rupture zone will extend into the lower layer(s) depending on their thickness and require some modification of q_{ult}. There are three general cases of the footing on a layered soil as follows:

Case 1. Footing on layered clays (all $\phi = 0$) as in Fig. 4-5*a*.
- *a.* Top layer weaker than lower layer ($c_1 < c_2$)
- *b.* Top layer stronger than lower layer ($c_1 > c_2$)

Case 2. Footing on layered ϕ–c soils with *a, b* same as case 1.

Case 3. Footing on layered sand and clay soils as in Fig. 4-5*b*.
- *a.* Sand overlying clay
- *b.* Clay overlying sand

Experimental work to establish methods to obtain q_{ult} for these three cases seems to be based mostly on models—often with $B < 75$ mm. Several analytical methods exist as well, and apparently the first was that of Button (1953), who used a circular arc to search for an approximate minimum, which was found (for the trial circles all in the top layer) to give $N_c = 5.5 < 2\pi$ as was noted in Sec. 4-2.

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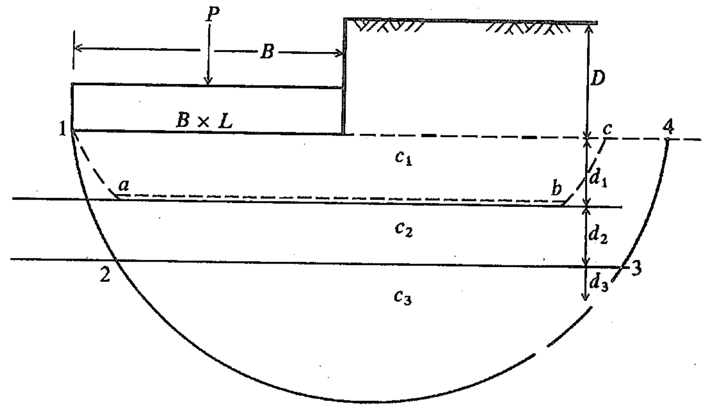

(*a*) Footing on layered clay soil. For very soft c_1 failure may occur along sliding block 1*abc* and not a circular arc and reduce N_c to a value less than 5.14.

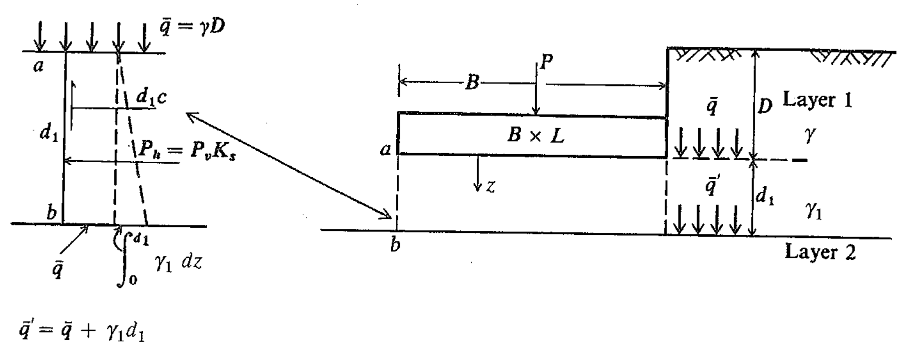

Figure 4-5 Footings on layered soil.

The use of trial circular arcs can be readily programmed for a computer (see program B-1 on diskette) for two or three layers using s_u for the layers. Note that in most cases the layer s_u will be determined form q_u tests, so the circle method will give reasonably reliable results. It is suggested that circular arcs be limited to cases where the strength ratio $C_R = c_2/c_1$ of the top two layers is on the order of

$$0.6 < C_R \leq 1.3$$

Where C_R is much out of this range there is a large difference in the shear strengths of the two layers, and one might obtain N_c using a method given by Brown and Meyerhof (1969) based on model tests as follows:

For $C_R \leq 1$

$$N_{c,s} = \frac{1.5d_1}{B} + 5.14C_R \qquad \leq 5.14 \qquad \text{(for strip footing)} \tag{4-5}$$

For a circular base with B = diameter

$$N_{c,r} = \frac{3.0d_1}{B} + 6.05C_R \qquad \leq 6.05 \qquad \text{(for round base)} \tag{4-6}$$

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When $C_R > 0.7$ reduce the foregoing $N_{c,i}$ by 10 percent.

For $C_R > 1$ compute:

$$N_{1,s} = 4.14 + \frac{0.5B}{d_1} \qquad \text{(strip)} \tag{4-7}$$

$$N_{2,s} = 4.14 + \frac{1.1B}{d_1} \tag{4-7a}$$

$$N_{1,r} = 5.05 + \frac{0.33B}{d_1} \qquad \text{(round base)} \tag{4-8}$$

$$N_{2,r} = 5.05 + \frac{0.66B}{d_1} \tag{4-8a}$$

In the case of $C_R > 1$ we compute both $N_{1,i}$ and $N_{2,i}$ depending on whether the base is rectangular or round and then compute an averaged value of $N_{c,i}$ as

$$N_{c,i} = \frac{N_{1,i} \cdot N_{2,i}}{N_{1,i} + N_{2,i}} \cdot 2 \tag{4-9}$$

The preceding equations give the following typical values of $N_{c,i}$, which are used in the bearing-capacity equations of Table 4-1 for N_c.

	$C_R = 0.4$		2.0		
d_1/B	Strip	Round	$N_{1,s}$	$N_{2,s}$	$N_{c,s}$
0.3	2.50	3.32	5.81	7.81	6.66
0.7	3.10	4.52	4.85	5.71	5.13
1.0	3.55	5.42	4.64	5.24	4.92

When the top layer is very soft with a small d_1/B ratio, one should give consideration either to placing the footing deeper onto the stiff clay or to using some kind of soil improvement method. Model tests indicate that when the top layer is very soft it tends to squeeze out from beneath the base and when it is stiff it tends to "punch" into the lower softer layer [Meyerhof and Brown (1967)]. This result suggests that one should check this case using the procedure of Sec. 4-2 that gave the "lower-bound" solution—that is, if $q_{\text{ult}} > 4c_1 + \bar{q}$ of Eq. (c) the soil may squeeze from beneath the footing.

Purushothamaraj et al. (1974) claim a solution for a two-layer system with ϕ–c soils and give a number of charts for N_c factors; however, their values do not differ significantly from N_c in Table 4-4. From this observation it is suggested for ϕ–c soils to obtain modified ϕ and c values as follows:

1. Compute the depth $H = 0.5B\tan(45 + \phi/2)$ using ϕ for the top layer.
2. If $H > d_1$ compute the modified value of ϕ for use as[5]

$$\phi' = \frac{d_1\phi_1 + (H - d_1)\phi_2}{H}$$

[5]This procedure can be extended to any number of layers as necessary, and "weighting" may be used.

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3. Make a similar computation to obtain c'.
4. Use the bearing-capacity equation (your choice) from Table 4-1 for q_{ult} with ϕ' and c'.

If the top layer is soft (low c and small ϕ) you should check for any squeezing using Eq. (c) of Sec. 4-2.

For bases on sand overlying clay or clay overlying sand, first check if the distance H will penetrate into the lower stratum. If $H > d_1$ (refer to Fig. 4-5) you might estimate q_{ult} as follows:

1. Find q_{ult} based on top-stratum soil parameters using an equation from Table 4-1.
2. Assume a punching failure bounded by the base perimeter of dimensions $B \times L$. Here include the $\overline{q}$ contribution from d_1, and compute q'_{ult} of the lower stratum using the base dimension B. You may increase q'_{ult} by a fraction k of the shear resistance on the punch perimeter $(2B + 2L) \times ks_u$ if desired.
3. Compare q_{ult} to q'_{ult} and use the smaller value.

In equation form the preceding steps give the controlling q'_{ult} as

$$q'_{ult} = q''_{ult} + \frac{pP_vK_s \tan\phi}{A_f} + \frac{pd_1c}{A_f} \quad \leq q_{ult} \tag{4-10}$$

where
q_{ult} = bearing capacity of top layer from equations in Table 4-1
q''_{ult} = bearing capacity of lower layer computed as for q_{ult} but also using B = footing dimension, $\overline{q} = \gamma d_1$; c, ϕ of lower layer
p = total perimeter for punching [may use $2(B + L)$ or $\pi \times$ diameter]
P_v = total vertical pressure from footing base to lower soil computed as $\int_0^{d_1} \gamma h\, dh + \overline{q}d_1$
K_s = lateral earth pressure coefficient, which may range from $\tan^2(45 \pm \phi/2)$ or use K_o from Eq. (2-18a)
$\tan\phi$ = coefficient of friction between P_vK_s and perimeter shear zone wall
pd_1c = cohesion on perimeter as a force
A_f = area of footing (converts perimeter shear forces to a stress)

This equation is similar to that of Valsangkar and Meyerhof (1979) and applies to all soils.

Note that there will not be many cases of a two- (or three-) layer cohesive soil with clearly delineated strata. Usually the clay gradually transitions from a hard, overconsolidated surface layer to a softer one; however, exceptions may be found, primarily in glacial deposits. In these cases it is a common practice to treat the situation as a single layer with a worst-case s_u value. A layer of sand overlying clay or a layer of clay overlying sand is somewhat more common, and the stratification is usually better defined than for the two-layer clay.

A possible alternative for ϕ–c soils with a number of thin layers is to use average values of c and ϕ in the bearing-capacity equations of Table 4-1 obtained as

$$c_{av} = \frac{c_1H_1 + c_2H_2 + c_3H_3 + \cdots + c_nH_n}{\sum H_i} \tag{a}$$

URS

Job AEP Big Sandy Pond Closure	Project No. 13815151	Page ___ of ___	
Description Bearing Capacity Analysis for cap construction - CALC 2	Computed by MSJ	Sheet 1 of 4	Date 3/12/13
	Checked by		Date

Reference

SUBGRADE IMPROVEMENT BY PRE-LOADING OPTION

- The purpose of this second set of calcs is to determine bearing capacity available for construction of cap at AEP Big Sandy Pond Closure project.
- This calculation is a follow-up to calculation set 1 prepared on 3/11/13.

As indicated on calc set 1, for construction of cap using Deere 850K tracked dozer or similar equipment,

$q_{app} = 1200$ psf

For minimum $F.S = 1.5$ $q_{reqd} = 1800$ psf.

Case I of calc set 1 had concluded that using drained parameters of $\phi' = 15.5°$ & neglecting c (conservative); cap placement directly on fluidized fly ash is not possible due to inadequate bearing capacity.

Consider a measure of adding surcharge (stockpile of bottom ash) onto fluidized fly ash to improve strength of flyash by pre-loading the layer, thereby increasing confining stress. Use the total strength envelope (attached sheet 3/4) for flyash obtained from lab testing.

Using the same spreadsheet as in calc set 1 using Meyerhoff Equation (see attached sheet 4/4)

$$q_{ult} = c N_c s_c d_c + q_1' N_q s_q d_q + 0.5 \gamma B N_\gamma s_\gamma d_\gamma$$

Using drained condition & total strength envelope; determine bearing capacity using c calculated from the total strength envelope for a normal stress "q" applied by pre-loading.

Use c & consider $\phi = 0°$; (also neglect help from q)

$q' = 0$ $\quad \phi = 0°$

$$\therefore q_{ult} = c N_c s_c d_c + \cancel{q' N_q s_q d_q} + \cancel{0.5 \gamma B N_\gamma s_\gamma d_\gamma}$$

By assuming values of c required to obtain $q_{ult} = 1800$ psf

using spreadsheet $\quad c_{reqd} \approx 350$ psf

$\therefore c_{reqd} = 350$ psf $= \dfrac{350}{144} = \underline{\underline{2.45}}$ psi

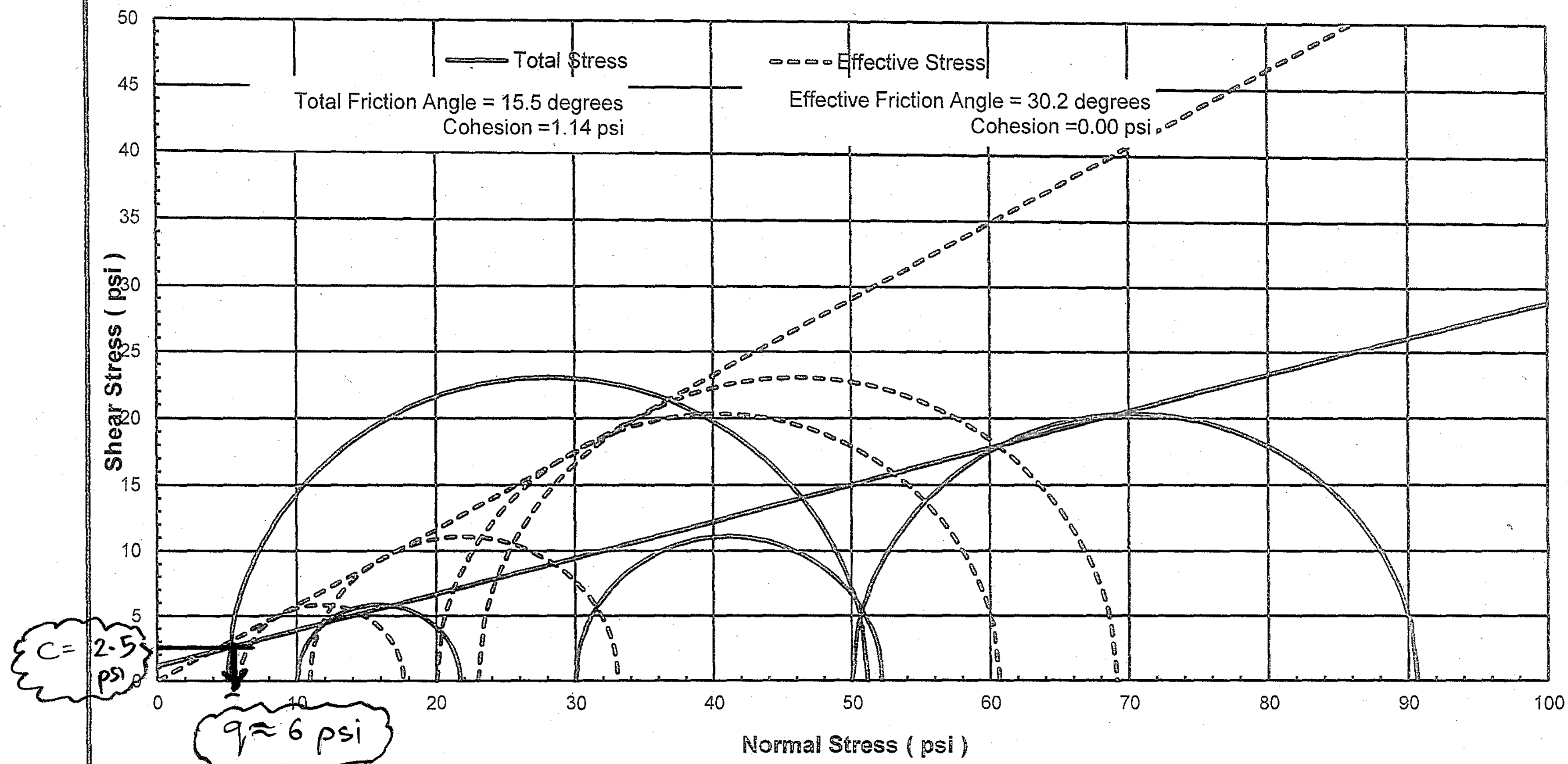
For $c' = 2.5$ psi ; $q_{reqd} \approx 6$ psi from CIU Triaxial Stress Envelope using Total Stress envelope

$\therefore q_{reqd} = 6$ psi $= 6 \times 144 = \underline{\underline{864}}$ psf

Assuming unit wt of bottom ash = 115 pcf ; ht of bottom ash surcharge reqd $= \dfrac{864}{115} \approx 7.5$ ft

$\therefore$ Surcharge of 7.5 feet of bottom ash placed for $\approx$ 6 months pre-loading duration may allow placement of cap on flyash subgrade.

Sheet 3/4



Test at 5-psi effective consolidation stress excluded from Total Friction strength envelope

Project No. T13815141	AEP URS Corporation	Mohr Circles of Total and Effective Stresses at Peak CIU' Triaxial Test Coarse Composite SUMMARY	Figure 2 June 2012
TerraSense, LLC			

Sheet 4/4

Exhibit 1

Ultimate Bearing Capacity Computation Using Myerhoff's Equations

Project: AEP Big Sandy, Louisa, Kentucky
Computed By: MSJ/VKG
Location: Bearing Analysis for cap construction - Calc set 2

Input Information

Foundation Soil Properties	Input Set #1	Input Set #2	Input Set #3	Input Set #4	Input Set #5
Friction Angle ϕ (deg)	0	0			
Cohesion c (psf)	350	300			
Unit Weight γ (pcf)	107	107			
Footing Width B (ft)	4	4			
Footing Length L (ft)	9	9			
Bearing Depth D (ft)	0	0			
User input q'	0	0			

Input Descriptions
Enter description of each calculation here

Input Set #	Description of Calculation
1	CASE I: Use assumed c= 350 psf for fluidized fly ash, unit wt = 107 pcf.
2	CASE II: Use assumed c= 300 psf for fluidized fly ash
3	
4	
5	

Calculations

Vertical Stress At Base (psf)	Set #1	Set #2	Set #3	Set #4	Set #5
$q' = \gamma D$ or user input value	0	0			

Bearing Capacity Factors	Set #1	Set #2	Set #3	Set #4	Set #5
$N_q = e^{\pi \tan\phi} \tan^2\left(45+\frac{\phi}{2}\right)$	1.00	1.00			
$N_c = (N_q - 1)\cot\phi$	5.14	5.14			
$N_\gamma = (N_q - 1)\tan(1.4\phi)$	0.00	0.00			
$s_q = 1 + 0.1K_p \frac{B}{L}$ (=1 if ϕ<10)	1.00	1.00			
$s_c = 1 + 0.2K_p \frac{B}{L}$	1.09	1.09			
$s_\gamma = s_q$	1.00	1.00			
$d_q = 1 + 0.1\sqrt{K_p}\frac{D}{B}$ (=1 if ϕ<10)	1.00	1.00			
$d_c = 1 + 0.2\sqrt{K_p}\frac{D}{B}$	1.00	1.00			
$d_\gamma = d_q$	1.00	1.00			

Estimated Ultimate Bearing Capacity

$$q_{ult} = cN_c s_c d_c + q' N_q s_q d_q + 0.5\gamma B N_\gamma s_\gamma d_\gamma$$

	Set #1	Set #2	Set #3	Set #4	Set #5
q_{ult} =	1,959	1,679			

SETTLEMENT ANALYSES

BIG SANDY POWER PLANT ASH POND CLOSURE PROJECT

GEOTECHNICAL CALCULATIONS

I. Purpose: The purpose of this analysis is to evaluate the primary consolidation settlement along the main porewater collection drain trunk line as well as the strain developed in geosynthetic components of the cap system as part of the Big Sandy Pond Closure project.

The settlement analyses summarized herein were performed to support the design of the Big Sandy Pond Closure project. The consolidation settlement analysis included determination of the total and differential settlement anticipated due to consolidation of the subsurface materials under the surcharge of new borrow fill to be placed over the pond as part of the closure. Total settlement at the elevation of the invert of the proposed main porewater collection drain line (referred to herein as the trunk line) was calculated at discrete points along its alignment, to determine change in slope due to differential settlement and ascertain the ability of the trunk line to maintain positive drainage from the upper pond to the lower pond. Total settlement along the drainage channel was calculated to ascertain the ability of the drainage channel to maintain positive drainage with limited potential for ponding. In addition, the strain developed in the cap system due to settlement across a critical cross-section was analyzed.

II. Project Description

The proposed project consists of closure of the 140-acre Big Sandy Fly Ash impoundment reservoir, which is used to dispose coal combustion product (fly ash) generated at the Big Sandy Plant owned and operated by American Electric Power. The Plant is located on the west bank of the Big Sandy River, near Louisa, KY with the ash reservoir located approximately ¾ mile northwest of the Plant.

In order to aid the drainage and achieve design grades necessary for placement of final cap, borrow fill material will be placed over the existing pond surface in four phases, starting at the upstream end and extending downstream to the main dam. The borrow fill will be contoured to promote surface water drainage into a central surface water channel, and the overall system (including the channel) will be sloped to drain from the west to north. A majority of the surface/storm water will be directed into the central surface channel (referred to as the main drainage channel in Phases 1 and 2, and the tributary drainage channel in Phase 3) to a new spillway constructed at the existing saddle dam, with a smaller portion directed to a new spillway at the existing main dam. Stormwater channels were centrally located based on the unique configuration of the pond and the distance from the western end to the proposed spillway. Slopes for the channels were set at 0.5% from the west draining to the existing saddle dam at the east and the cap system was graded at a cross-slope of 2% to drain toward the central channel.

The pore water management system will include a main trunk line and two lateral lines installed along the length of ash pond with its lowest point at the leachate pump station near the Saddle Dam. The trunk line will consist of two 6-inch diameter high density polyethylene (HDPE) pipes constructed in the middle of a 2-foot thick bottom ash subgrade layer, approximately 3-foot below the proposed invert of the central channel. The two 6-inch diameter pipes will be separated by a horizontal clear distance of 13 feet and have a 0.5% minimum slope.

The top of the borrow fill will be capped to complete the closure. Two final cap systems will be used to close the fly ash pond. Areas with 5 feet or more of contouring fill will receive a soil cap system. The soil cap system consists of 18 inches of re-compacted clay and 6 inches of top soil with vegetative cover. Areas with less than 5 feet of contouring fill will not allow for adequate compaction of the clay, and will therefore receive a geosynthetic cap system. The geosynthetic cap system consists of a flexible membrane layer (FML), a geocomposite drainage layer, and 24 inches of top soil with vegetative cover.

Over the majority of the closure footprint, final grades will be below existing grades, so no net surcharge will be placed over the sluiced ash in the pond. Significant settlements of the cap or associated features (such as the main drainage channel and pore water drain lines) are not anticipated in these areas.

A few areas of the closure will include permanent fills to raise grades for the cap system. These areas are subject to potential settlements induced by the surcharge created by the fill and include:

- Phase 1 of the closure, where new fill ranging in height from 24 to 26 feet will be required to meet the proposed grades.
- A small area at the upstream (west) end of Phase 2. This area includes placement of approximately 3 to 4 feet of new fill below the first 600 feet (Sta. 0+00 to Sta. 6+00) of the main trunk line.
- The northern end of Phase 3 and all of Phase 4. A pore water drain is not proposed in this area, and the main drainage channel occupies only a small portion of this area.

The above areas are the focus of this settlement analysis.

III. Settlement Analysis - Theory

Classical settlement theory was used herein. At the Big Sandy site, both the sluiced fly ash deposit and underlying alluvium are anticipated to be in a normally consolidated state, based on the results of 1D consolidation testing performed as part of the subsurface exploration (see further discussion below). Therefore, for the analyses presented herein, the following equation applies:

$$s_i = \frac{H_i C_c}{1+e_0} \log \frac{\sigma_0' + \Delta\sigma}{\sigma_p'} \qquad \text{where,}$$

s_i = Settlement of layer

H_i = Thickness of layer

C_c = Compression Index of layer

e_0 = Initial void ratio at layer

σ_0' = Effective overburden pressure at layer center

σ_p' = Effective preconsolidation pressure

$\Delta\sigma$ = Surcharge pressure at layer center, γz

Some analyses presented herein estimate the point-to-point strain in geosynthetic components of the cap system, as a result of differential settlement. For these calculations, strain was computed as follows:

$$E_T(\%) = \frac{L_f - L_0}{L_0} * 100 \quad \text{where,}$$

E_T = Tensile strain

L_f = Original distance separating two location points

L_0 = Final distance separating the same two points after settlement is complete

Methodology for determination of the various parameters involved in the above equations is described in subsequent sections.

IV. Settlement Analysis - Methodology

Settlement analysis was performed for cross-sections A-A, B-B, and C-C, as shown on Figure A-1 in Attachment A. These sections correspond to areas of the project where more significant fills will be placed and were selected and analyzed as follows:

- *Section A-A:* Currently, the primary geosynthetic component proposed as part of the cap system is a geocomposite drainage layer, proposed to be used as necessary, in the soil and geosynthetic cap system. In addition, a 40-mil PVC geomembrane liner will be used in the geosynthetic cap system proposed for the Closure Phases 2 and 3, where the depth of contouring fill is less than 5 feet.

  It is anticipated that, with this nominal thickness of new fill, settlements and strains of the cap system will be small. To provide a conservative analysis and interpretation of cap system strains, a cross-section within Phase 1 (Section A-A) was selected for analysis. Cross-section A-A is considered a critical section with respect to the potential for settlement-induced strain in the cap system for the following reasons: 1) It is located within Phase 1, which represents the maximum amount of borrow fill placement (24 to 26 feet in height) and maximum surcharge; 2) Cross-section A-A represents a minimal width across the fly ash pond wherein the calculated strains will be the largest for the same amount of differential settlement. A pattern of closely spaced analysis points, ranging in spacing from 10 to 50 feet was utilized for the settlement analysis along Section A-A.

Although the cap at this location will not include geosynthetics (Phase 1 is to have a soil cap), the results of the analysis at this section will provide a conservative interpretation of potential strains in the geosynthetic cap areas of the project.

- *Section B-B*: This section is located near the upstream origin of the pore water drain trunk line. As such, it is a conservative representation of the area of the small segment of trunk line that will be constructed over fill and has greatest potential for settlement. The maximum settlement of the trunk line was estimated using Section B-B. The reference point for the settlement analysis was taken immediately below the center of the main drainage channel. The resulting settlement value is used to provide a conservative indication of the change in slope of the trunk line and main drainage channel (due to settlement) that may occur between the location of the cross-section and points just to the east, where cuts rather than fills are proposed.
- *Section C-C*: This section is considered to be representative of the northern end of Phase 3, where fills are proposed. The reference point for the settlement analysis was taken immediately below the center of the tributary drainage channel. The resulting settlement value is used to provide a conservative indication of the change in slope of the Phase 3 main drainage channel (due to settlement) that may occur between the location of the cross-section and points to the southeast, where little to no fill is proposed.

The overall steps/elements of the settlement analysis are:

1. *Stratigraphic Profile for Settlement Analyses:* Establish the stratigraphic profile beneath the trunk line as well as Cross-section A-A at each settlement analysis point.

2. *Selection of Compressibility Parameters:* Select compressibility parameters for each major stratigraphic deposit.

3. *Determination of Net Surcharge Loading:* For each settlement analysis point, utilize the proposed final grade and existing grades to determine the thickness of new fill and corresponding surcharge loading at that point.

4. *Settlement Calculations:* Using the input established in Steps 1 through 3 above and the settlement equations given in Section II above, compute the settlement of each subsurface layer at each settlement analysis point. Sum the layer settlements to compute the total estimated settlement at each analysis point.

For Cross-section A-A, perform settlement calculations (per No. 4 above) at a number of reference points along the length of the cross-section. Then compare settlements of adjacent reference points and compute point-to-point strains as described above in Section III.

Detailed methodology of each step is described below:

1. Stratigraphic Profile for Settlement Analysis

A geotechnical exploration was conducted at the site in the Spring of 2012. The exploration has been described in detail in the Geotechnical Summary Report (under separate cover).

For the purposes of the settlement analyses herein that are conducted mostly within the pond area, the subsurface profile can be described to consist of very loose/very soft sluiced fly ash underlain by native alluvium consisting of interbedded loose to medium dense sands and soft to stiff clays. The alluvium is underlain by a thin layer of residuum and then by bedrock. As the residuum consists of dense granular or very stiff to hard cohesive soils, it has been excluded from the settlement analysis. Bedrock has been similarly excluded.

Stratigraphy for the major units included for the settlement analyses (sluiced fly ash and alluvium) was developed as follows:

1. *Fly Ash* – The elevation of the top of the fly ash deposit is well defined by the topographic and bathymetric survey information available to the project. The thickness of the fly ash stratum (or the bottom elevation of the stratum) is well defined at the discrete locations at which borings were performed within the pond, but is not accurately known at points in between, or more importantly for the settlement analysis along Section A-A. A historical USGS map (dated 1953) was available and provides some estimate of the existing grades within the stream valley prior to the construction of the main dam and the start of sluicing of ash within the pond. The contours shown on the USGS map were assumed herein to represent the elevation of the bottom of the fly ash stratum/top of alluvium. The USGS map was scaled and overlain on the project plans and the contours were digitized and made into a 3D surface. The resulting surface was then checked and adjusted to conform to the spot information corresponding to the boring locations, and the thickness of the fly ash deposit at any given point was established from the adjusted surface. The accuracy of the USGS topo used to establish the aforementioned surface is considered limited, and the fly ash thicknesses and elevation of the bottom of ash/top of alluvium in between discrete boring locations is considered to be approximate.

   For the settlement analysis along cross-section A-A, the top of fly ash across the transverse section was assumed to be constant at EL 572 (based on the elevation fly ash encountered in boring PB-2 along cross-section A-A). The bottom of the fly ash was evaluated using the surface established from the 1953 USGS map as described above.

For the analysis of Sections B-B and C-C (where settlement is computed for a single reference point), the profiles of borings PB-3 and PB-8 were used, respectively to establish the top and bottom of fly ash elevations.

2. *Native Alluvium* – For the settlement analysis along Cross-Section A-A, the thickness of the native alluvium across the transverse section was assumed to be constant at 17 feet (based on the alluvium thickness encountered in boring PB-2, located along the cross-section). It is anticipated that the thickness of alluvium will be reduced at the edges of the cross-section as the valley walls are approached. The thickness of the alluvium was thus estimated as 10 feet for settlement analysis points located at the edges of the cross-section.

   For Cross-Sections B-B and C-C, the thickness of alluvium was established from borings PB-3 and PB-8, respectively.

2. Selection of Compressibility Parameters

Based on the mechanism by which the fly ash was deposited, it was assumed to be a normally consolidated material in all settlement analyses herein. The results of consolidation testing on alluvium specimens indicated that the materials are normally or perhaps slightly overconsolidated. Herein, calculations were performed assuming that the alluvium is normally consolidated.

Material properties pertinent to the settlement analysis of normally consolidated materials include the soil total unit weight *(γ)*, initial void ratio *(e_0)*, and compression index (C_c). The table below summarizes the parameter values selected for use in the settlement analyses:

Layer	**γ(pcf)**	**e_0**	**C_c**
Fly Ash	109	0.923	0.095
Alluvium	126	0.633	0.142
Borrow fill	124	--	--

The basis for selecting these values is as follows:

- Two laboratory one-dimensional consolidation tests were performed using composite fly ash samples that were reconstituted using the fluviation procedure described in the Geotechnical report. The values of e_0, and C_c used for the fly ash are average values based on the lab results. The total unit weight was calculated using the average of the total unit weights obtained for the fine composite and the coarse composite sections of the fly ash as reported by the lab testing. Refer to the Geotechnical Summary Report (under separate cover) for a detailed presentation of the lab testing.

- As described previously, the alluvium consists of interbedded sand and clay soils. For the purposes of settlement analyses, the alluvium was modeled as wholly clay material. This is a conservative assumption, as it is anticipated that settlement will be larger in cohesive soils as compared to cohesionless soils under similar fill loads. Three laboratory one-dimensional consolidation tests were performed on the native alluvium soils. The high plasticity (CH) alluvium noted in only one boring (PB-4) was neglected and the average values of e_0, and C_c based on the lab results for the remaining two samples of alluvium were used in establishing the parameters given above. Again, refer to the Geotechnical Summary Report (under separate cover) for a detailed presentation of the lab testing.
- Proctor testing on proposed borrow source materials has been performed and is presented in detail in the Soil Borrow Study Report (under separate cover). The results of these tests were utilized herein to establish the unit weight of the borrow materials to be placed for the pond closure. For purposes of analysis, all borrow fill materials are assumed to have a total unit weight of 124 pcf. This value is based on an assumption of achieving 90% compaction for the lower 3 feet of borrow fill placed on top of the fly ash layer and 95% compaction for layers above that depth.

3. Determination of Net Surcharge Loading

Settlement under a given point will be induced by the difference in the effective vertical stress after completion of construction and the effective vertical stress currently existing at that location – i.e., the net surcharge applied.

The thickness of fill varies within the cross-section (generally highest at the edges of the closure, tapering to a low value immediately within the main drainage channel). The surcharge magnitude for the analysis of Sections B-B and C-C was calculated based on the average thickness of fill within 100 ft on each side of the centerline of the main drainage channel. The surcharge from this thickness was then assumed to be applied over a wide area to simulate an infinite surcharge loading to be conservative. For Section A-A, where several reference points are analyzed, the surcharge at any point along the cross-section was computed based on thickness of proposed fill at that point.

4. Settlement Calculations

A separate settlement analysis was performed for each settlement reference point as described above. The result at each point, i.e. the resulting settlement of all layers below the point, was then tabulated. Given the large number of computations required, the analyses were performed using in-house spreadsheets.

V. Results of Settlement Analysis

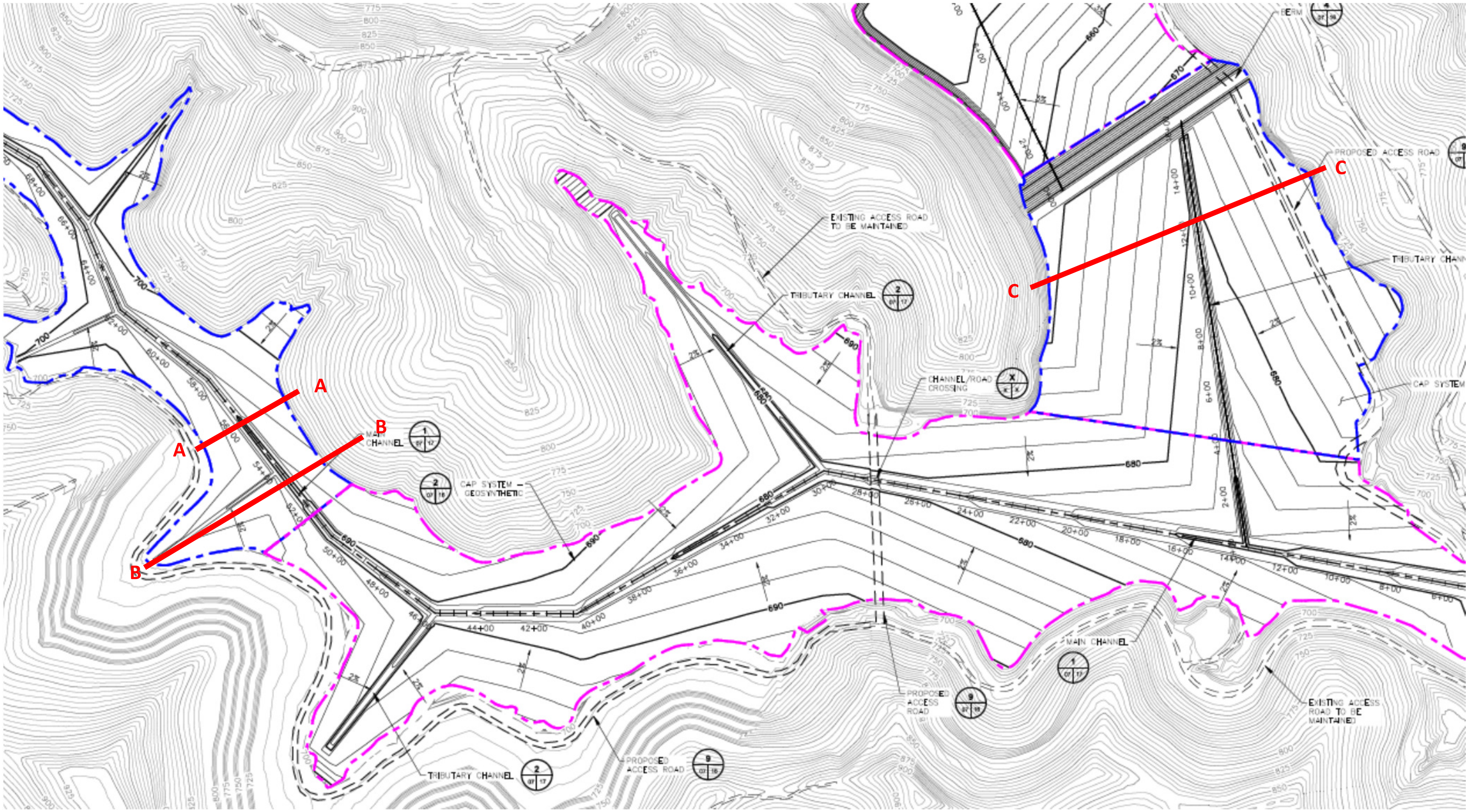
Pertinent spreadsheet and example settlement calculations for each cross-section, are provided in Attachment B of this memo.

The following points summarize the results of the analysis:

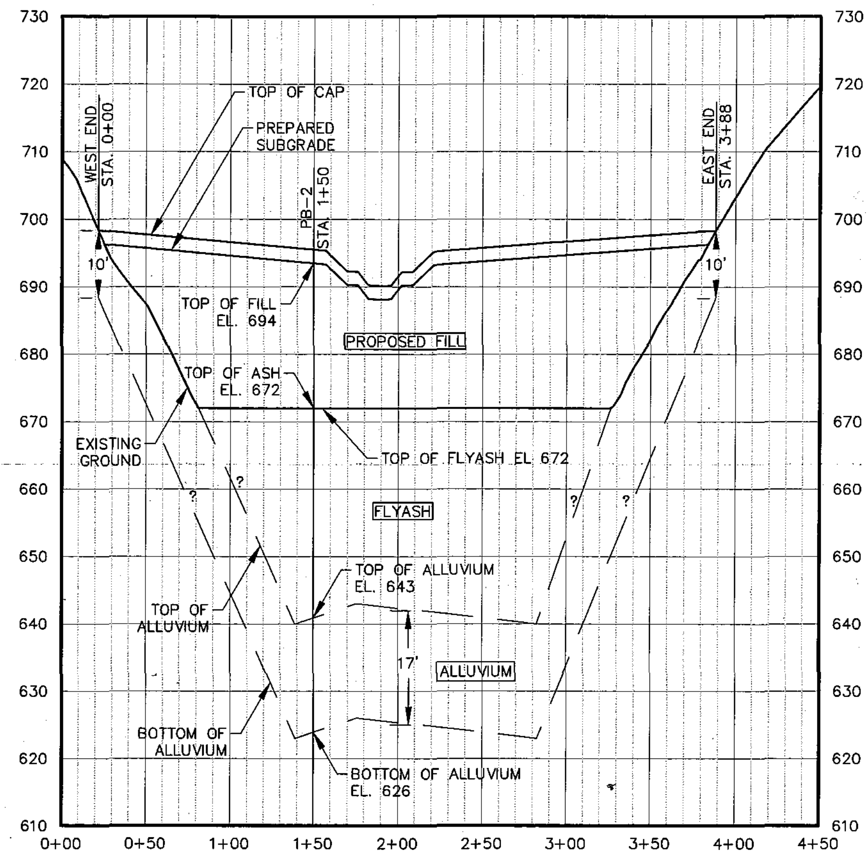
- *Section A-A*: The calculated settlement under the main drainage channel is approximately 20-inches. The calculations for differential settlement and geosynthetic strain along Cross-section A-A in Closure Phase 1 with a maximum fill height of 24 to 26 feet indicate a maximum induced strain of 0.1%. Typical allowable strains for geosynthetics are in the range of 1%-5%, much larger than the computed value. Therefore, it is concluded that no appreciable concerns exist with respect to geosynthetic strain due to settlement.
- *Section B-B*: The calculated settlement of the reference point at Section B-B was 19-inches. As described previously, the value represents the settlement of the pore water drain trunk line at its upstream origin (Sta. 0+00). Furthermore, around Sta. 6+00, the settlement is anticipated to drop to zero, as no new fill is proposed at or adjacent to the trunk line/main drainage channel at this station. Thus, an average change in slope of 19-inches over 600 ft, or 0.26% is anticipated over this length. This does not exceed the design slope of the trunk line/channel (which is 0.5%) and a localized reduction in slope over this relatively short distance is not anticipated to cause serviceability problems with surface water or subsurface drainage.
- *Section C-C*: The calculated settlement of the reference point at Section C-C was 15-inches. As described previously, this value represents the settlement of the main drainage channel at the northern end of tributary drainage channel (approximate Sta. 14+00). The settlement of the channel is anticipated to gradually reduce moving south through Phase 3, to approximately zero near Sta. 0+00, where little to no fill is proposed as part of the closure. Thus, an average change in slope of 15-inches over 1400 ft, or 0.09% is anticipated over this length. This is a small proportion of the design slope of the channel (which is 0.5%) and is not anticipated to cause serviceability problems with surface water drainage.
- Finally, as stated above the calculated settlements under the main drainage channel at Sections A-A and B-B are 20-inches and 19-inches, respectively. The length of channel between these two sections is approximately 250 ft, so the average change in slope of the channel is 1-inch over 250 ft, or 0.03%. Again, this is a small proportion of the design channel slope, so settlement is not anticipated to cause serviceability problems with surface water drainage in this area.
- The results indicate that settlements of the various project features should not result in serviceability problems. However, it is noted that settlements at the main drainage

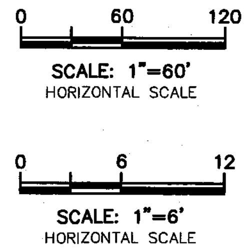
channel may be mitigated by placing the borrow fill within Phase 1, rough grading the channel and then allowing a wait time for settlement to take place prior to final capping and final grading of the channel. An estimate of the time required to realize a significant proportion of the anticipated ash and alluvium settlements has been performed and is estimated to be approximately 3 to 6 months.

ATTACHMENT A



SETTLEMENT ANALYSIS CROSS-SECTIONS





A–A' PROFILE STA. 0+00 TO STA 4+50

URS PROJECT NO. 13812901

URS
Cleveland • Columbus • Akron

AMERICAN ELECTRIC POWER

BIG SANDY PLANT

LOUISA KENTUCKY

SECTION A–A' PROFILE

DWG. NO.

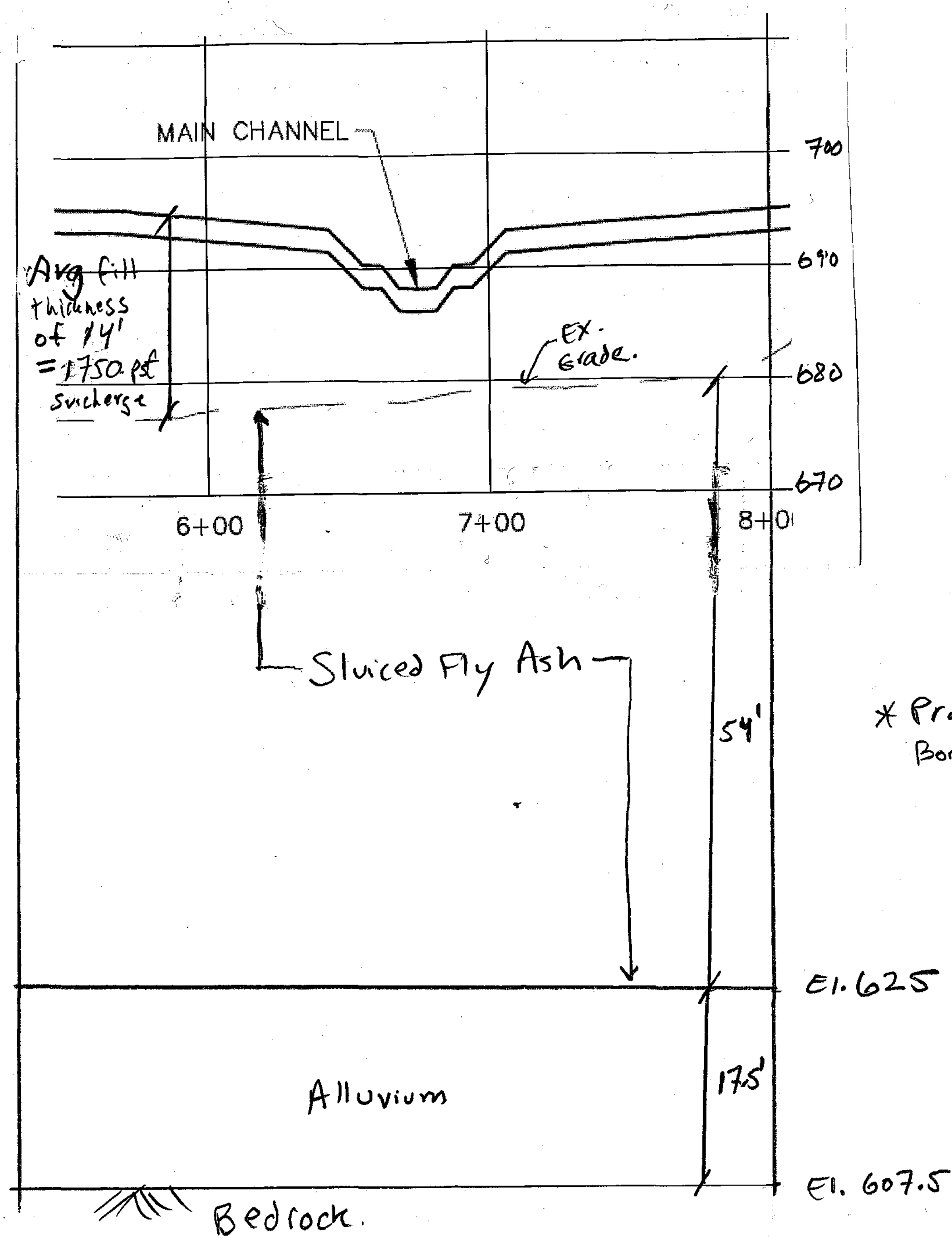
NTS		CIVIL ENGINEERING DIVISION
XXX	APPROVED BY	
XXX		
ENGR: XXX		
PROJ ENGR: XXX		
DATE: XX–XX–XX		

AEP AMERICAN ELECTRIC POWER

AEP SERVICE CORP.
1 RIVERSIDE PLAZA
COLUMBUS, OH 43215

URS

Job AEP Big Sandy Pond Closure | Project No. | Page of
Description Analysis Cross-Section B-B Subsurface Profile. | Computed by | Sheet of
Checked by | Date | Date
Reference



* Profile Based on Boring PB-3

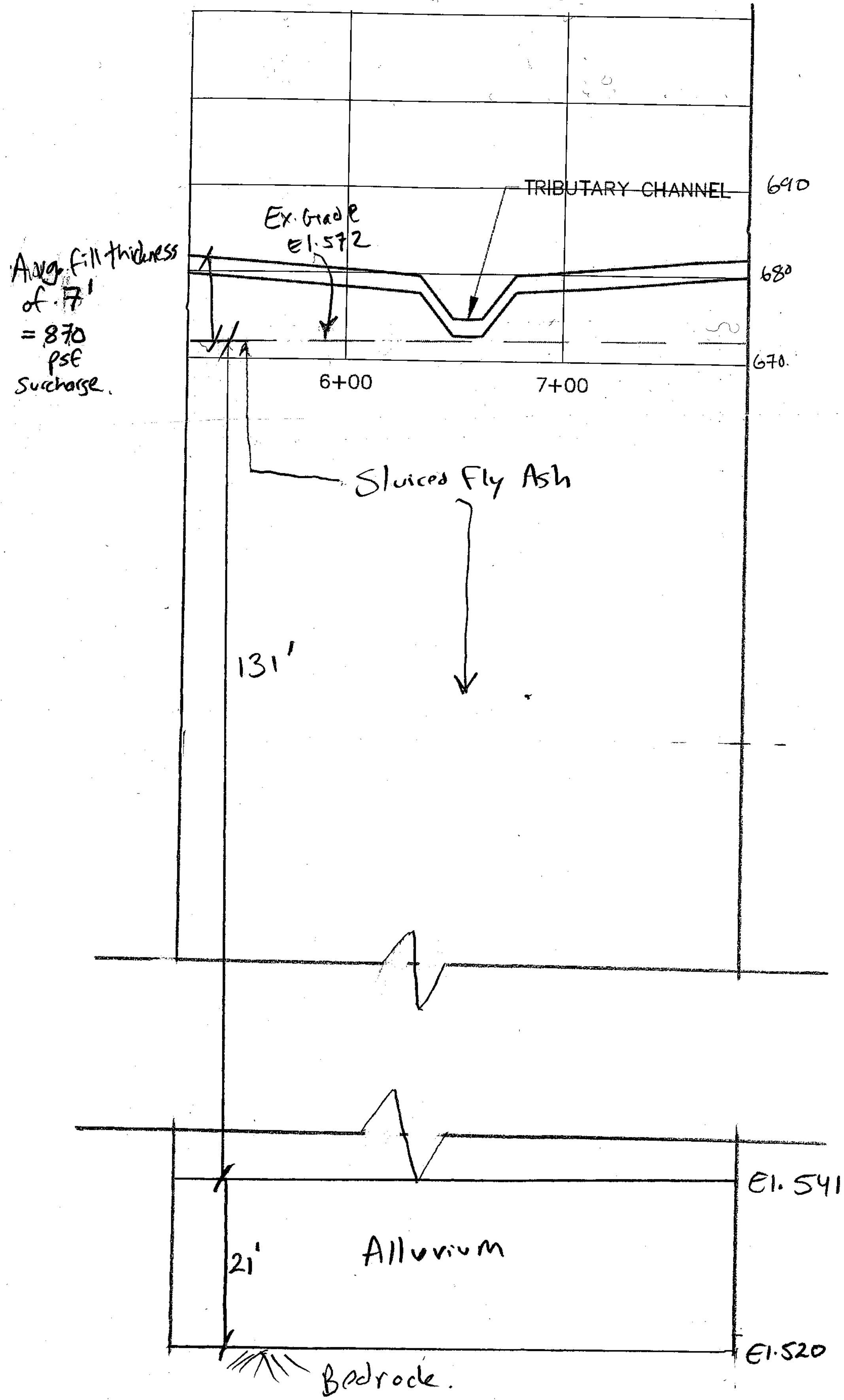
URS

Job AEP BIG SANDY POND CLOSURE, Project No. ______ Page ___ of ___ Sheet ___ of ___

Description Analysis Cross-Section C-C → Subsurface Profile. Computed by ______ Date ______

Checked by ______ Date ______

Reference



* Profile Based on Boring PB-8

ATTACHMENT B

CALCULATIONS FOR SECTION A-A

TABLE B.1
SUMMARY TABLE FOR SETTLEMENT ALONG CROSS-SECTION A-A

INPUT FOR SETTLEMENT SPREADHSEET

STATION	0*	10*	60	80	120	150	170	200	230	280	310	340*	360*	370*
PROP. FINAL GRADE (Top of fill) (EL)	698.5	698	697	696.5	696	692	690	694.5	695	696	697	697.5	698	698.5
TOP OF EXISTING GRADE (FLY ASH/ALLUVIUM*) (EL)	698.5	696	672	672	672	672	672	672	672	672	672	686	696	698.5
PROPOSED HEIGHT OF FILL (ft)	0	2	25	24.5	24	20	18	22.5	23	24	25	11.5	2	0
SURCHARGE DUE TO FILL (psf)	0	248	3100	3038	2976	2480	2232	2790	2852	2976	3100	1426	248	0
BOTTOM OF FLY ASH LAYER (EL)	N/A	N/A	671.9	660	640	643	642.45	641.6	640.8	652	671.9	N/A	N/A	N/A
HEIGHT OF FLY ASH LAYER (ft)	0	0	0.1	12	32	29	29.55	30.4	31.2	20	0.1	0	0	0
BOTTOM OF ALLUVIUM LAYER (EL)	688.5	685	654.9	643	623	626	625.45	624.6	623.8	635	654.9	672.5	685	688.5
HEIGHT OF ALLUVIUM LAYER (ft)	10	11	17	17	17	17	17	17	17	17	17	13.5	11	10

NOTE: * = EXISTING GRADE IS ALLUVIUM

OUTPUT FROM SETTLEMENT SPREADHSEET

STATION	0	10	60	80	120	150	170	200	230	280	310	340	360	370
SETTLEMENT IN FLY ASH (in)	0	0	7.8	8.2	14.9	13.3	13.1	14.1	14.6	14.9	7.8	0	0	0
SETTLEMENT IN ALLUVIUM (in)	0	5.1	8.1	10.4	6.7	6.7	6.5	6.7	6.8	5	8.1	10.3	5.1	0
TOTAL SETTLEMENT (in)	0	5.1	15.9	18.6	21.6	20	19.6	20.8	21.4	19.9	15.9	10.3	5.1	0
TOTAL SETTLEMENT (ft)	0	0.425	1.325	1.55	1.8	1.667	1.634	1.734	1.784	1.659	1.325	0.859	0.425	0
NET SETTLEMENT (ft)	0	0.425	0.9	0.225	0.25	0.133	0.033	0.1	0.05	0.125	0.334	0.466	0.434	0.425
ORIGINAL LENGTH (ft)	0	10	50	20	40	30	20	30	30	50	30	30	20	10
FINAL LENGTH (ft)	0	10.0091	50.0081	20.0013	40.0008	30.0003	20.0001	30.0002	30.0001	50.0002	30.0019	30.0037	20.0048	10.0091
STRAIN (%)	0.00	0.09	0.02	0.01	0.002	0.001	0.001	0.001	0.000	0.001	0.01	0.01	0.02	0.09

Station for Main
Drainage Channel

ATTACHMENT B: SAMPLE SETTLEMENT CALCULATION

<u>Cross-Section A-A, STA. 1+20</u>

<u>Input data:</u>

Based on the profile shown in "Figure B.2 – Profile along Cross-section A-A"

- Proposed final grade = EL 696.0
- Top of Existing Fly Ash layer = EL 672.0
- Bottom of Fly Ash layer = EL 640.0
- Top of Alluvium layer = EL 640.0
- Bottom of Alluvium layer = EL 623.0

The input data for settlement file (Page 2 of the calculation) is as follows:

- Height of fill layer at x= 120 is 696-672 = 24 feet.
  Total unit weight of borrow fill material = 124 pcf.
  Total surcharge load = 24*124 = 2976 psf.
  Create a table for distribution of surcharge using unit weight of 124 pcf and height of fill layer for each analysis point with x changing from 0 to 370 as presented in Summary Table B.1. Use x=120 for point under consideration.

- Height of fly ash layer = 672-640 = 32 feet.
  Use Layers 1-4 of 8 feet thickness each.
  Total fly ash height = 4*8 = 32 feet

- Height of alluvium layer = 640-623 = 17 feet.
  Use 3 layers for alluvium with Layers 5 and 6 of 6 feet and Layer 7 of 7 feet thickness.
  Total Alluvium height = 6+6+5= 17feet.

  Material properties for compressibility of flay ash and alluvium are assigned as per description in the memo.

<u>Output:</u>

Based on output file: Total Computed Settlement = 21.6 inches
Settlement in fly ash layer = 14.9 inches
Settlement in alluvium layer = 6.7 inches

One-Dimensional Settlement Analysis

Project Name and #:	**AEP Big Sandy**
Date:	**12/2/2012**
Description:	**Settlement Along Tranverse Section AA**
	Sta 1+20
Computed By:	**MSJ/ VKG**

INPUT DATA

Soil Profile Input

Water Table Depth: 0.01 ft

Enter up to 20 layers

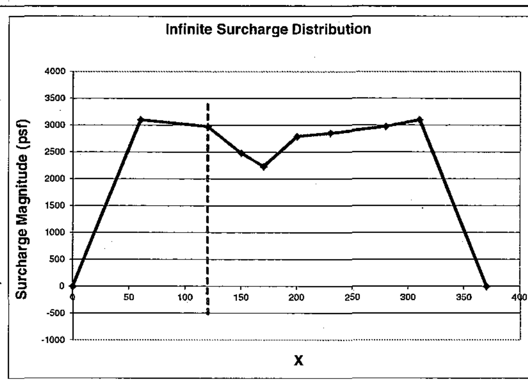
Layer Number	Layer Description	Layer Thickness (ft)	unit wt (pcf)	e_0 at layer	P_c* (psf)	C_r	C_c
1	Fly Ash	8	109	0.923	187.024	0.015	0.095
2	Fly Ash	8	109	0.923	559.824	0.015	0.095
3	Fly Ash	8	109	0.923	932.624	0.015	0.095
4	Fly Ash	8	109	0.923	1305.424	0.015	0.095
5	Alluvium	6	126	0.633	1682.624	0.018	0.142
6	Alluvium	6	126	0.633	2064.224	0.018	0.142
7	Alluvium	5	126	0.633	2414.024	0.018	0.142
8							
9							
10							
11							
12							
13							
14							
15							
16							
17							
18							
19							
20							

Surcharge Input

Infinite Surcharge Geometry (Up to 10 Points)

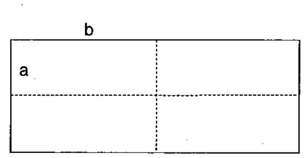
x-Coordinate of Settlement Point: 120

Point Number	x coordinate	surcharge value (psf)
1	0	0
2	60	3100
3	120	2976
4	150	2480
5	170	2232
6	200	2790
7	230	2852
8	280	2976
9	310	3100
10	370	0



Rectangular Loading

Include Rectangular Load?	n		
	Load 1	**Load 2**	**Load 3**
Enter Corner or Center (CO or CE) - Defaults to CE	CE	CE	CO
Enter magnitude, q (psf)			
Enter Width, a (ft)			
Enter Length, b (ft)			



Whole width = 2a; Whole length = 2b (for both CE and CO)

One-Dimensional Settlement Analysis

Project Name and #: AEP Big Sandy
Date: 12/2/2012
Description: Settlement Along Tranverse Section AA
Computed By: MSJ/ VKG

Settlement Anaysis Summary

Water Table Depth: 0.01 ft

Enter up to 20 layers

Layer Number	Layer Description	Layer Thickness (ft)	unit wt (pcf)	e_0 at layer	Overcons? (Y or N)	P_c* (psf)	C_r	C_c	Layer Center Depth (ft)	P_0 at layer Center (psf)	Layer* Surcharge (psf)	P_f at layer center (psf)	Layer Settlement (in)
1	Fly Ash	8	109	0.923	N	187.024	0.015	0.095	4	187.024	2957.6	3144.6	5.81
2	Fly Ash	8	109	0.923	N	559.824	0.015	0.095	12	559.824	2921.1	3480.9	3.76
3	Fly Ash	8	109	0.923	N	932.624	0.015	0.095	20	932.624	2885.1	3817.7	2.90
4	Fly Ash	8	109	0.923	N	1305.42	0.015	0.095	28	1305.424	2848.9	4154.4	2.38
5	Alluvium	6	126	0.633	N	1682.62	0.018	0.142	35	1682.624	2816.4	4499.0	2.67
6	Alluvium	6	126	0.633	N	2064.22	0.018	0.142	41	2064.224	2787.5	4851.7	2.32
7	Alluvium	5	126	0.633	N	2414.02	0.018	0.142	46.5	2414.024	2760.0	5174.0	1.73

TOTAL COMPUTED SETTLEMENT: 21.6 in

SETTLEMENT IN FLY ASH: 14.9 in

One-Dimensional Settlement Analysis

Project Name and #: 12/2/2012
Date: Settlement Along Tranverse Section AA
Description: MSJ/ VKG

Surcharge Load Summary

All Loadings in pounds per square foot

Layer Number	Layer Description	Center Depth	Rectangular Loading (psf)			Infinite Surcharge (psf)	TOTAL SURCHARGE (psf)
			Load 1	*Load 2*	*Load 3*		
1	Fly Ash	4				2958	2958
2	Fly Ash	12				2921	2921
3	Fly Ash	20				2885	2885
4	Fly Ash	28				2849	2849
5	Alluvium	35				2816	2816
6	Alluvium	41				2788	2788
7	Alluvium	46.5				2760	2760

CALCULATIONS FOR SECTION B-B

One-Dimensional Settlement Analysis

Project Name and #: AEP Big Sandy
Date: 6/11/2013
Description: Estimated Maximum Settlement Along Leachate Trunk Line Sta. 0+00 (Based on Section B-B)
Computed By: VKG

INPUT DATA

Soil Profile Input

Water Table Depth: 0.01 ft

Enter up to 20 layers

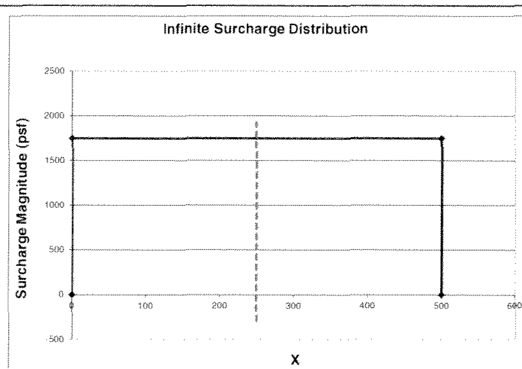
Layer Number	Layer Description	Layer Thickness (ft)	unit wt (pcf)	e_0 at layer	P_c* (psf)	C_r	C_c
1	Fly Ash	7	109	0.923	163.724	0.015	0.095
2	Fly Ash	7	109	0.923	489.924	0.015	0.095
3	Fly Ash	7	109	0.923	816.124	0.015	0.095
4	Fly Ash	7	109	0.923	1142.324	0.015	0.095
5	Fly Ash	7	109	0.923	1468.524	0.015	0.095
6	Fly Ash	7	109	0.923	1794.724	0.015	0.095
7	Fly Ash	6	109	0.923	2097.624	0.015	0.095
8	Fly Ash	6	109	0.923	2377.224	0.015	0.095
9	Alluvium	6	126	0.633	2707.824	0.018	0.142
10	Alluvium	6	126	0.633	3089.424	0.018	0.142
11	Alluvium	5.5	126	0.633	3455.124	0.018	0.142
12							
13							
14							
15							
16							
17							
18							
19							
20							

Surcharge Input

Infinite Surcharge Geometry (Up to 10 Points)

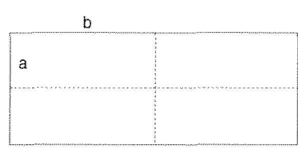
x-Coordinate of Settlement Point: 250

Point Number	x coordinate	surcharge value (psf)
1	0	0
2	0.001	1750
3	500	1750
4	500.01	0
5		
6		
7		
8		
9		
10		



Rectangular Loading

	Load 1	Load 2	Load 3
Include Rectangular Load?	n		
Enter Corner or Center (CO or CE) - Defaults to CE	CE	CE	CO
Enter magnitude, q (psf)			
Enter Width, a (ft)			
Enter Length, b (ft)			



Whole width = 2a; Whole length = 2b (for both CE and CO)

One-Dimensional Settlement Analysis

Project Name and #: AEP Big Sandy
Date: 6/11/2013
Description: Estimated Maximum Settlement Along Leachate Trunk Line
Computed By: VKG

Settlement Anaysis Summary

Water Table Depth: 0.01 ft

Enter up to 20 layers

Layer Number	Layer Description	Layer Thickness (ft)	unit wt (pcf)	e_0 at layer	Overcons? (Y or N)	P_c* (psf)	C_r	C_c	Layer Center Depth (ft)	P_0 at layer Center (psf)	Layer* Surcharge (psf)	P_f at layer center (psf)	Layer Settlement (in)
1	Fly Ash	7	109	0.923	Y	163.724	0.015	0.095	3.5	163.724	1750.0	1913.7	4.431
2	Fly Ash	7	109	0.923	Y	489.924	0.015	0.095	10.5	489.924	1749.9	2239.9	2.739
3	Fly Ash	7	109	0.923	Y	816.124	0.015	0.095	17.5	816.124	1749.7	2565.9	2.064
4	Fly Ash	7	109	0.923	Y	1142.32	0.015	0.095	24.5	1142.324	1749.3	2891.6	1.674
5	Fly Ash	7	109	0.923	Y	1468.52	0.015	0.095	31.5	1468.524	1748.5	3217.1	1.413
6	Fly Ash	7	109	0.923	Y	1794.72	0.015	0.095	38.5	1794.724	1747.4	3542.1	1.225
7	Fly Ash	6	109	0.923	Y	2097.62	0.015	0.095	45	2097.624	1745.8	3843.5	0.935
8	Fly Ash	6	109	0.923	Y	2377.22	0.015	0.095	51	2377.224	1744.0	4121.2	0.850
9	Alluvium	6	126	0.633	Y	2707.82	0.018	0.142	57	2707.824	1741.7	4449.5	1.350
10	Alluvium	6	126	0.633	Y	3089.42	0.018	0.142	63	3089.424	1739.0	4828.4	1.214
11	Alluvium	5.5	126	0.633	Y	3455.12	0.018	0.142	68.75	3455.124	1735.9	5191.0	1.015

TOTAL COMPUTED SETTLEMENT: 18.9 in

One-Dimensional Settlement Analysis

Project Name and #: 6/11/2013
Date: Estimated Maximum Settlement Along Leachate Trunk Line
Description: VKG

Surcharge Load Summary

All Loadings in pounds per square foot

Layer Number	Layer Description	Center Depth	Rectangular Loading (psf)			Infinite Surcharge (psf)	TOTAL SURCHARGE (psf)
			Load 1	*Load 2*	*Load 3*		
1	Fly Ash	3.5				1750	1750
2	Fly Ash	10.5				1750	1750
3	Fly Ash	17.5				1750	1750
4	Fly Ash	24.5				1749	1749
5	Fly Ash	31.5				1749	1749
6	Fly Ash	38.5				1747	1747
7	Fly Ash	45				1746	1746
8	Fly Ash	51				1744	1744
9	Alluvium	57				1742	1742
10	Alluvium	63				1739	1739
11	Alluvium	68.75				1736	1736

CALCULATIONS FOR SECTION C-C

One-Dimensional Settlement Analysis

Project Name and #: AEP Big Sandy
Date: 6/11/2013
Description: Estimated Maximum Settlement At Tributary Channel
Sta. 0+00 (Based on Section C-C)
Computed By: VKG

INPUT DATA

Soil Profile Input

Water Table Depth: 0.01 ft

Enter up to 20 layers

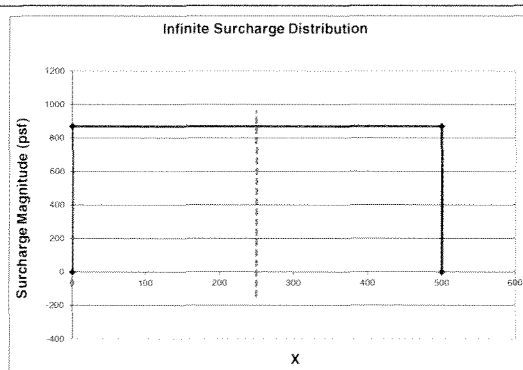
Layer Number	Layer Description	Layer Thickness (ft)	unit wt (pcf)	e_0 at layer	P_c* (psf)	C_r	C_c
1	Fly Ash	8	109	0.923	187.024	0.015	0.095
2	Fly Ash	8	109	0.923	559.824	0.015	0.095
3	Fly Ash	8	109	0.923	932.624	0.015	0.095
4	Fly Ash	8	109	0.923	1305.424	0.015	0.095
5	Fly Ash	9	109	0.923	1701.524	0.015	0.095
6	Fly Ash	9	109	0.923	2120.924	0.015	0.095
7	Fly Ash	9	109	0.923	2540.324	0.015	0.095
8	Fly Ash	9	109	0.923	2959.724	0.015	0.095
9	Fly Ash	9	109	0.923	3379.124	0.015	0.095
10	Fly Ash	9	109	0.923	3798.524	0.015	0.095
11	Fly Ash	9	109	0.923	4217.924	0.015	0.095
12	Fly Ash	9	109	0.923	4637.324	0.015	0.095
13	Fly Ash	9	109	0.923	5056.724	0.015	0.095
14	Fly Ash	9	109	0.923	5476.124	0.015	0.095
15	Fly Ash	9	109	0.923	5895.524	0.015	0.095
16	Alluvium	7	126	0.633	6327.824	0.018	0.142
17	Alluvium	7	126	0.633	6773.024	0.018	0.142
18	Alluvium	7	126	0.633	7218.224	0.018	0.142
19							
20							

Surcharge Input

Infinite Surcharge Geometry (Up to 10 Points)

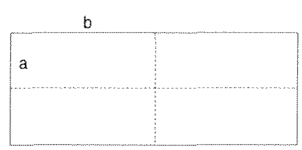
x-Coordinate of Settlement Point: 250

Point Number	x coordinate	surcharge value (psf)
1	0	0
2	0.001	870
3	500	870
4	500.01	0
5		
6		
7		
8		
9		
10		



Rectangular Loading

	Load 1	Load 2	Load 3
Include Rectangular Load?	n		
Enter Corner or Center (CO or CE) - Defaults to CE	CE	CE	CO
Enter magnitude, q (psf)			
Enter Width, a (ft)			
Enter Length, b (ft)			



Whole width = 2a; Whole length = 2b (for both CE and CO)

One-Dimensional Settlement Analysis

Project Name and #: AEP Big Sandy
Date: 6/11/2013
Description: Estimated Maximum Settlement At Tributary Channel
Computed By: VKG

Settlement Anaysis Summary

Water Table Depth: 0.01 ft

Enter up to 20 layers

Layer Number	Layer Description	Layer Thickness (ft)	unit wt (pcf)	e_0 at layer	Overcons? (Y or N)	P_c* (psf)	C_r	C_c	Layer Center Depth (ft)	P_0 at layer Center (psf)	Layer* Surcharge (psf)	P_f at layer center (psf)	Layer Settlement (in)
1	Fly Ash	8	109	0.923	Y	187.024	0.015	0.095	4	187.024	870.0	1057.0	3.567
2	Fly Ash	8	109	0.923	Y	559.824	0.015	0.095	12	559.824	870.0	1429.8	1.931
3	Fly Ash	8	109	0.923	Y	932.624	0.015	0.095	20	932.624	869.8	1802.4	1.357
4	Fly Ash	8	109	0.923	Y	1305.42	0.015	0.095	28	1305.424	869.5	2174.9	1.051
5	Fly Ash	9	109	0.923	Y	1701.52	0.015	0.095	36.5	1701.524	868.9	2570.4	0.956
6	Fly Ash	9	109	0.923	Y	2120.92	0.015	0.095	45.5	2120.924	867.9	2988.8	0.795
7	Fly Ash	9	109	0.923	Y	2540.32	0.015	0.095	54.5	2540.324	866.4	3406.7	0.680
8	Fly Ash	9	109	0.923	Y	2959.72	0.015	0.095	63.5	2959.724	864.4	3824.1	0.594
9	Fly Ash	9	109	0.923	Y	3379.12	0.015	0.095	72.5	3379.124	861.8	4241.0	0.526
10	Fly Ash	9	109	0.923	Y	3798.52	0.015	0.095	81.5	3798.524	858.7	4657.2	0.472
11	Fly Ash	9	109	0.923	Y	4217.92	0.015	0.095	90.5	4217.924	854.9	5072.8	0.428
12	Fly Ash	9	109	0.923	Y	4637.32	0.015	0.095	99.5	4637.324	850.5	5487.8	0.390
13	Fly Ash	9	109	0.923	Y	5056.72	0.015	0.095	108.5	5056.724	845.5	5902.2	0.358
14	Fly Ash	9	109	0.923	Y	5476.12	0.015	0.095	117.5	5476.124	839.9	6316.0	0.331
15	Fly Ash	9	109	0.923	Y	5895.52	0.015	0.095	126.5	5895.524	833.7	6729.2	0.306
16	Alluvium	7	126	0.633	Y	6327.82	0.018	0.142	134.5	6327.824	827.7	7155.5	0.390
17	Alluvium	7	126	0.633	Y	6773.02	0.018	0.142	141.5	6773.024	822.2	7595.2	0.363
18	Alluvium	7	126	0.633	Y	7218.22	0.018	0.142	148.5	7218.224	816.3	8034.5	0.340

TOTAL COMPUTED SETTLEMENT: 14.8 in

One-Dimensional Settlement Analysis

Project Name and #: 6/11/2013
Date: Estimated Maximum Settlement At Tributary Channel
Description: VKG

Surcharge Load Summary

All Loadings in pounds per square foot

Layer Number	Layer Description	Center Depth	Rectangular Loading (psf)			Infinite Surcharge (psf)	TOTAL SURCHARGE (psf)
			Load 1	*Load 2*	*Load 3*		
1	Fly Ash	4				870	870
2	Fly Ash	12				870	870
3	Fly Ash	20				870	870
4	Fly Ash	28				869	869
5	Fly Ash	36.5				869	869
6	Fly Ash	45.5				868	868
7	Fly Ash	54.5				866	866
8	Fly Ash	63.5				864	864
9	Fly Ash	72.5				862	862
10	Fly Ash	81.5				859	859
11	Fly Ash	90.5				855	855
12	Fly Ash	99.5				851	851
13	Fly Ash	108.5				845	845
14	Fly Ash	117.5				840	840
15	Fly Ash	126.5				834	834
16	Alluvium	134.5				828	828
17	Alluvium	141.5				822	822
18	Alluvium	148.5				816	816

DEEP SEATED STABILITY ANALYSIS

BIG SANDY POWER PLANT ASH POND CLOSURE PROJECT

GEOTECHNICAL CALCULATIONS

I. Purpose: The purpose of this analysis is to evaluate deep-seated (global) slope stability of the proposed separation berm, to be constructed between the main and lower ponds as part of the Big Sandy Pond Closure project.

The deep-seated stability analysis summarized herein was performed in support of the Big Sandy Pond Closure design. The subject of this analysis is the separation berm to be installed north of the saddle dam, and separating the upper main pond from the lower pond. This analysis is intended to evaluate the berm stability, which is considered to be the most critical permanent slope included in the pond cap system.

All analyses were performed with the SLOPE/W computer program. The program uses limit equilibrium theory and standard procedures (e.g. Spencer's, Bishop, Janbu, etc.) to determine factors of safety for circular and block (translational) failure surface geometries. The program searches for critical factors of safety based on user-input of entry and exit lines along the ground surface. Additional information on the program is available at http://www.geo-slope.com/.

II. Model Development

The various parameters used to construct the slope stability models are as follows:

- Model geometry - including slope configuration (cross-sections), as well as soil units and stratigraphy
- Material Properties – including strength and unit weight properties for each material
- Water Table Elevations – including pond/groundwater levels
- Surcharge Loading - due to seismic conditions. The surcharge loading due to seismic conditions was obtained using seismic site response analysis including a 1D SHAKE analysis that has been performed as part of the liquefaction evaluation (submitted under separate cover).

Model Geometry

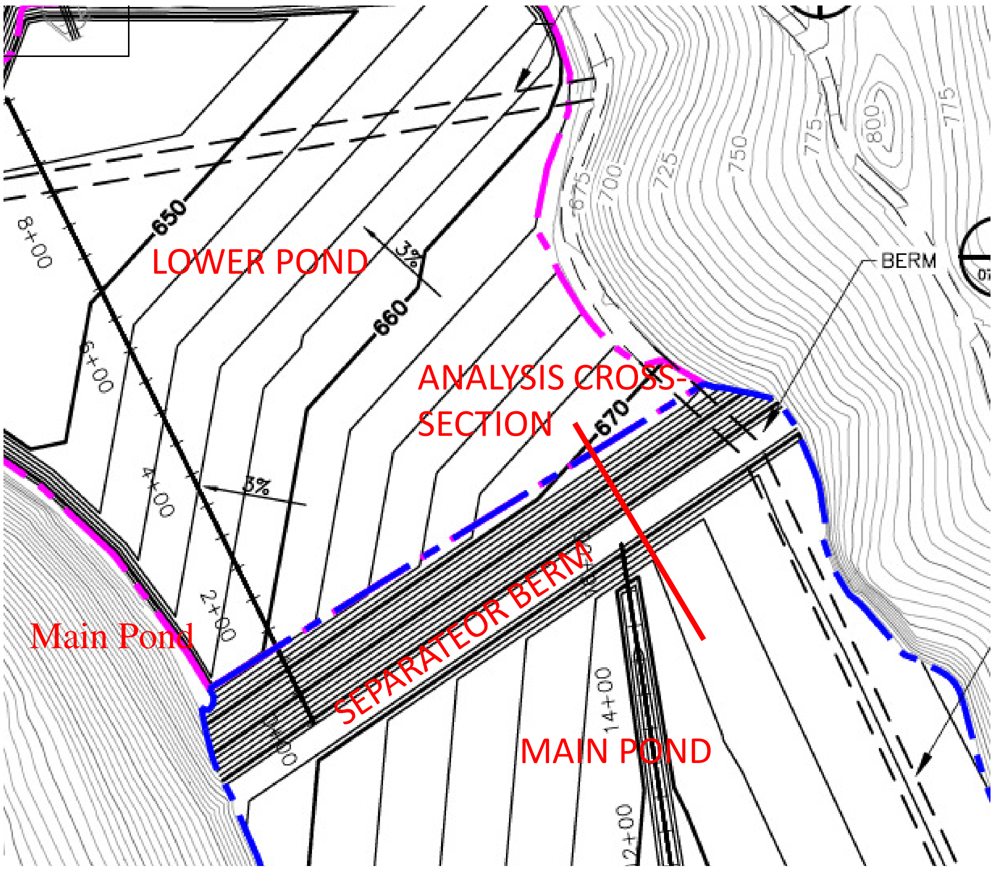
Cross-Section

The current Big Sandy Pond Closure design minimizes the amount of fill required to establish cap grades for the closure. As such significant permanently sloping ground or interim construction slopes are not proposed or anticipated on the project. Exceptions to this include:

1. Permanent cut slopes made at the existing valley walls, to obtain borrow fill for the pond closure.
2. A separation berm, to be constructed between the main pond and the lower pond that exists immediately behind the main dam, to accommodate the grade change that occurs between these two areas. This berm will have a height of 20 to 30 ft over the existing pond surface and will be constructed using new fill materials, placed above the pond surface.

Regarding No. 1 above, the permanent cuts proposed in the valley wall lie above the limits of the proposed cap system and as such are not anticipated to affect the finished cap. Furthermore, the majority of these cuts will be in rock, and limit equilibrium slope stability analyses are not pertinent.

The analysis performed herein is focused on the separation berm (No. 2 above). The cross-section analyzed is illustrated below. As the berm configuration and the existing pond grades within the berm footprint are relatively consistent, the cross-section location was chosen by inspection and is considered to be representative of the entire berm.



Summary of Subsurface Conditions

Pond boring PB-8 was performed on the existing splitter dike that currently occupies the area where the separation berm is to be constructed. The boring is located roughly at the center of the proposed separation berm's crest. This boring was utilized to establish the subsurface conditions for slope stability analysis. The boring encountered the following materials (from highest to lowest elevation):

- Loose Bottom ash (including both materials placed to make access for the boring and the material comprising the existing splitter dike) to El. 649.5;

- Very loose/very soft sluiced Fly Ash to El. 541;
- Alluvium (consisting of medium dense to dense silty sand);
- Sandstone bedrock.

As the alluvium and bedrock are at great depth in comparison to the height of the berm, these layers were excluded from the slope stability analysis.

Stratigraphy

Stratigraphy in the slope stability models was developed as follows:

1. *Separation Berm* – Configuration of the proposed separation berm (including crest width/elevation and sideslope) were based on the contours given in the design plans. It is assumed that the berm will be constructed of nominally compacted bottom ash. This was anticipated to be the most conservative assumption, as bottom ash will be modeled as a purely frictional material, with no cohesion. For simplicity, a separate layer for the cap system was not defined in the model. This is not considered to affect the results of the analysis.
2. *Splitter Dike* – The specific configuration of the existing splitter dike is unknown. Plans and construction records were not available for review at the time of this analysis. Therefore, the splitter dike configuration was assumed as described herein.

   The splitter dike crest elevation and horizontal position within the cross-section was established from the existing survey information. The base elevation of the dike was taken as the elevation of the base of bottom ash, as revealed in boring PB-8 (El. 649.5). Dike sideslopes were assumed to be 3H: 1V, which is consistent with the existing bathymetric slope north of the dike. The dike is presumed to be constructed entirely of loose bottom ash, as encountered in the boring.
3. *Sluiced Fly Ash* – Sluiced fly ash was assumed to make up the remainder of the cross-section's stratigraphy. Both upper and lower surfaces of the fly ash were assumed to be horizontal, consistent with the method of deposition by sluicing.

Material Properties

All materials were modeled with Mohr-Coulomb strength envelopes and assuming effective strength or total strength conditions, depending on the analysis. The material properties pertinent to this type of model include the total unit weight, γ, and strength parameters ϕ, friction angle, and c, cohesion.

Material parameters used in the slope stability analyses were as follows:

Effective Strength Parameters

Layer	γ(pcf)	ϕ(deg)	*c* (psf)
Separator Berm	100	34	0
Splitter Dike	80	32	0
Sluiced Fly Ash	107	30	0

.

Total Strength Parameters

Layer	γ(pcf)	ϕ(deg)	*c* (psf)
Separator Berm	Same as Effective Strength Parameters		
Splitter Dike	Same as Effective Strength Parameters		
Sluiced Fly Ash	107	15.5	164

Commentary:

- Material properties for the separator berm and splitter dike (both assumed to be comprised of bottom ash), were conservatively assumed as indicative of a lightly compacted and minimally compacted ash, respectively. As bottom ash is considered to be a rapidly draining material, both total and effective strength properties are assumed to be the same.
- Friction angle and cohesion intercept for the sluiced fly ash (both total and effective strength parameters) were established based on the results of laboratory isotropic consolidated-undrained triaxial testing with pore-pressure measurements. Choice test results are provided in the attachments. The total unit weight was taken as the approximate average of undisturbed piston samples collected during the subsurface exploration.

Water Table

The groundwater table was assumed at El. 670, which was roughly the water level observed in boring PB-8. This is considered to be a conservative representation of the long term water table, as water levels are expected to drop once the lower pond is drained for the closure.

III. Methodology

The analysis checked "deep-seated" circular failure surfaces, with a minimum thickness of 10 ft. SLOPE/W was utilized to calculate a large number of failure surface geometries, using the Entry and Exit routine of the program. Circular failure surface geometries were checked in this analysis.

Three separate analysis cases were checked, as described below:

Potential Failure Geometries:

Case 1: Effective Strength (Long Term) Conditions: This analysis represents the long term state of the berm. Pore pressures generated in the underlying sluiced fly ash during construction of the berm have come to equilibrium, and all materials are assigned drained (effective) strengths, as described in Section II. This was a static analysis (no seismic loading).

Case 2: Total Strength Conditions: This analysis assumed that the separator berm will be constructed very rapidly, and that the sluiced fly ash would not undergo any consolidation at the end of the construction process. The analysis assigns strength to the sluiced ash based on the total strength material properties given in Section II. Furthermore, the strengths are fully based on the pre-construction in-situ stresses – i.e., the vertical stresses induced by the separator berm weight were excluded when assigning shear strength to the sluiced ash. This was accomplished in SLOPE/W using the Shear Strength as a Function of Depth option. The initial cohesion value was equated to the total cohesion given in Section II, and the rate of shear strength increase with depth was established using the total friction angle and the effective unit weight (see Section II). This was also a static analysis. The analysis is considered conservative, as some consolidation (and corresponding strength gain) of the sluiced ash would be expected as the berm is constructed.

Case 3: Seismic Conditions: This analysis models the earthquake condition corresponding to an event with 2% probability of exceedence in 50 years and was implemented as a pseudostatic analysis with constant seismic coefficient in SLOPE/W. A seismic site response analysis including a 1D SHAKE analysis has been performed as part of the liquefaction evaluation (submitted under separate cover). That analysis indicated that dynamic liquefaction of the sluiced fly ash under the earthquake scenario is unlikely. Based on this result, effective strength parameters have been assumed for the fly ash. Furthermore, some results of the SHAKE analysis, specifically the peak ground acceleration at the surface of the pond computed by the program, were utilized herein. The computed PGA was approximately 0.09g. This acceleration was assumed at the base of the separator berm (which will be constructed at the surface of the pond). The acceleration at the crest of the separator berm is anticipated to be different, possibly higher than the basal acceleration, depending on the period of vibration of the berm structure and the response spectrum of the ground motions considered in SHAKE. For this analysis, the base acceleration of 0.09g was increased to 0.12g, and this value was used as the seismic coefficient in the pseudostatic analysis.

IV. Results of Analysis

The results of the analyses are given in the table below. SLOPE/W outputs depicting the critical failure surfaces are provided in the Attachment.

ANALYSIS CASE	CRITICAL FACTOR OF SAFETY
Case 1: Effective Strength Conditions	2.88
Case 2: Total Strength Conditions	1.37
Case 3: Seismic Conditions	1.85

V. Conclusions

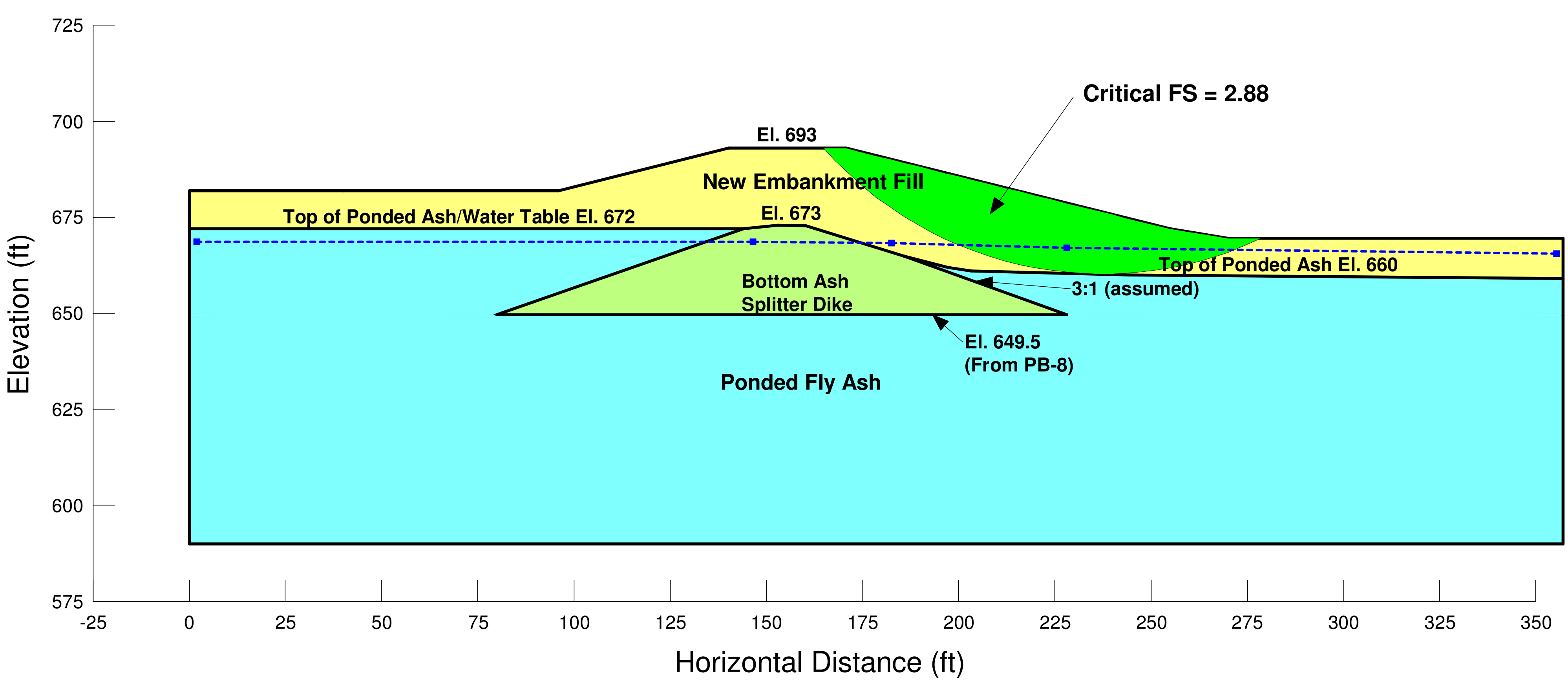
- The results of all cases indicate that factors of safety against deep-seated failures substantially exceed typical guidance values, including those provided in the Ohio EPA's "Geotechnical and Stability Analyses For Ohio Waste Containment Facilities", and the U.S. Corps of Engineers' EM-1110-2-1902 "Slope Stability".

ATTACHMENTS

AEP Big Sandy Pond Closure
Slope Stability Analysis - Separation Berm, North Pond
Effective Strength Static Conditions

Assumed Material Parameters

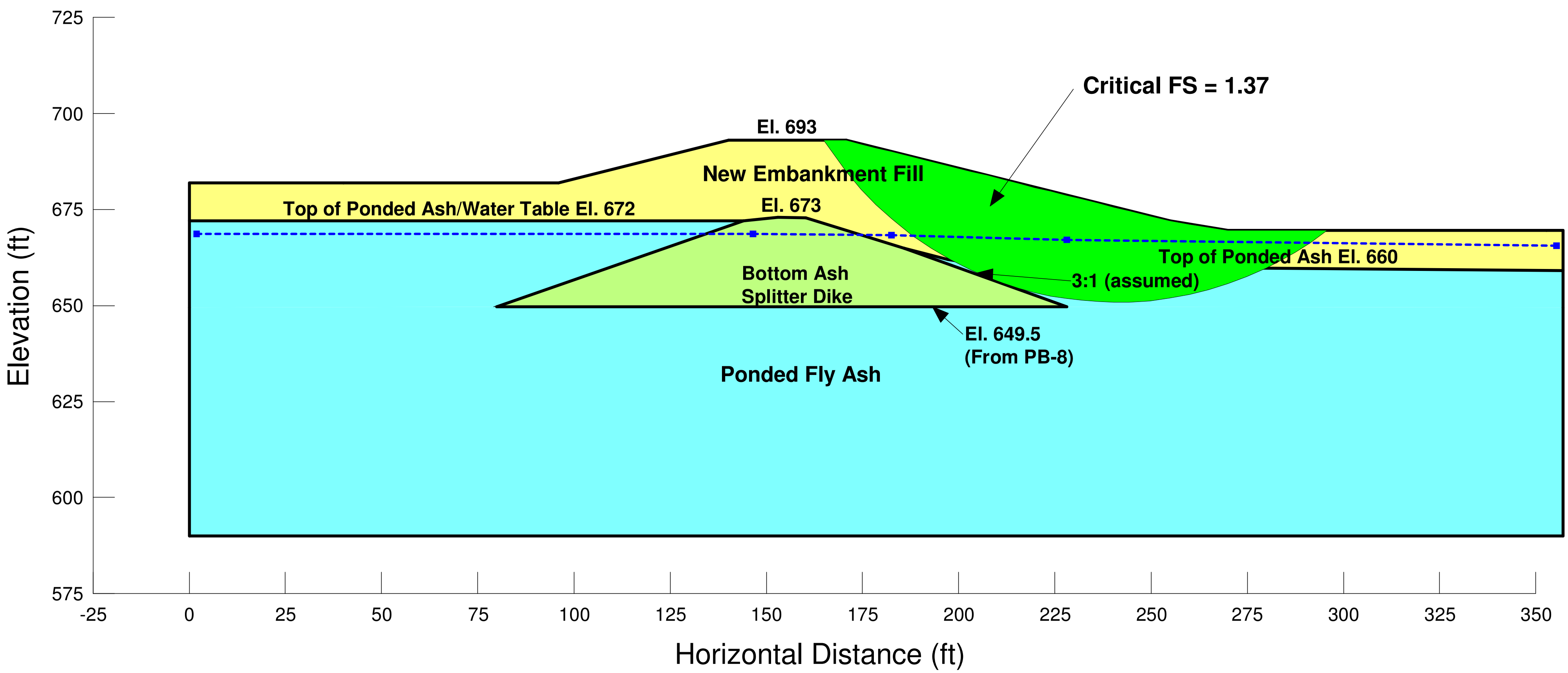
Name: Fill Embankment	Name: Ex. Bottom Ash Dike	Name: Ponded Fly Ash
Model: Mohr-Coulomb	Model: Mohr-Coulomb	Model: Mohr-Coulomb
Unit Weight: 100 pcf	Unit Weight: 80 pcf	Unit Weight: 107 pcf
Cohesion: 0 psf	Cohesion: 0 psf	Cohesion: 0 psf
Phi: 34 °	Phi: 32 °	Phi: 30 °
Piezometric Line: 1	Piezometric Line: 1	



AEP Big Sandy Pond Closure
Slope Stability Analysis - Separation Berm, North Pond
Total Strength Static Conditions

Assumed Material Parameters

Name: Fill Embankment	Name: Ex. Bottom Ash Dike	Name: Ponded Fly Ash
Model: Mohr-Coulomb	Model: Mohr-Coulomb	Model: S=f(depth)
Unit Weight: 100 pcf	Unit Weight: 80 pcf	Unit Weight: 107 pcf
Cohesion: 0 psf	Cohesion: 0 psf	C-Top of Layer: 164 psf
Phi: 34 °	Phi: 32 °	C-Rate of Change: 12.37 psf/ft
Piezometric Line: 1	Piezometric Line: 1	Limiting C: 1e+005 psf

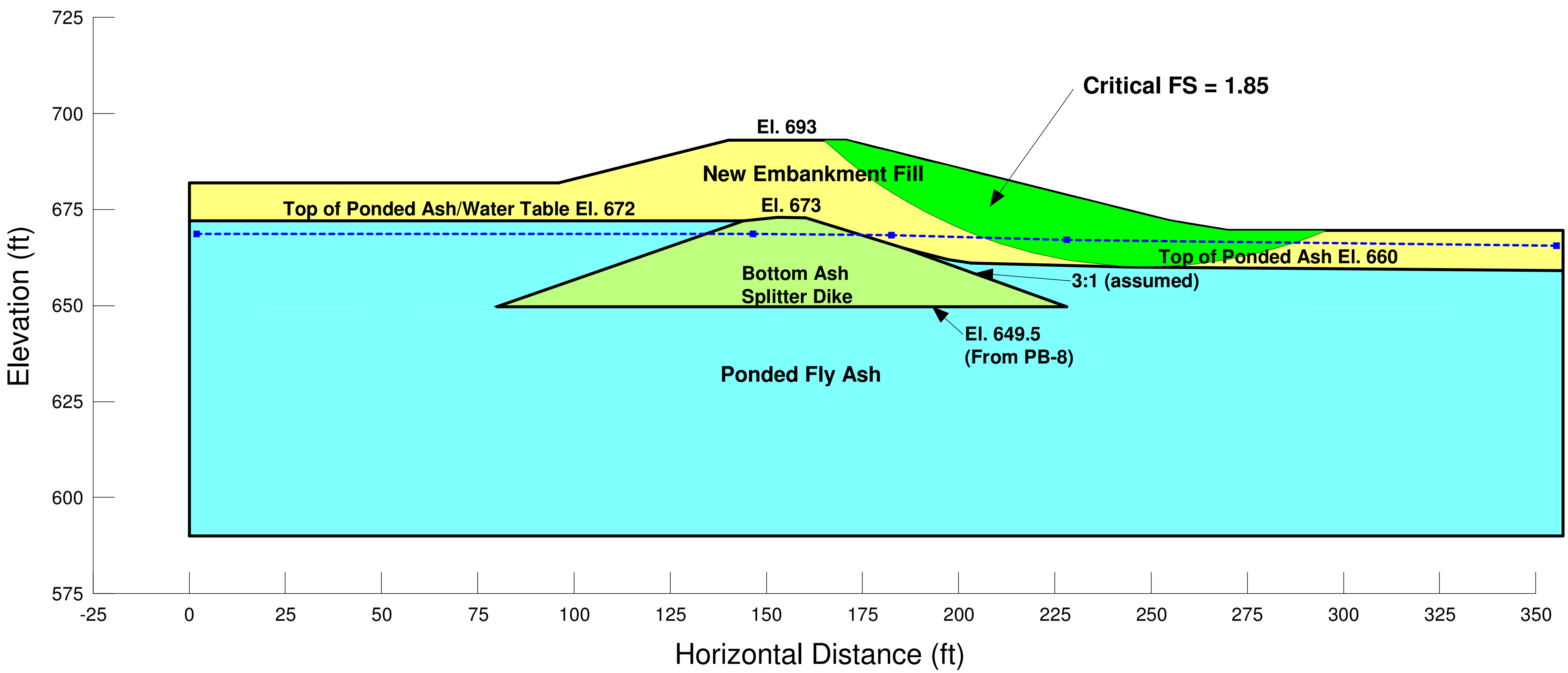


AEP Big Sandy Pond Closure
Slope Stability Analysis - Separation Berm, North Pond
Seismic Conditions

Assumed Material Parameters

Name: Fill Embankment	Name: Ex. Bottom Ash Dike	Name: Ponded Fly Ash
Model: Mohr-Coulomb	Model: Mohr-Coulomb	Model: Mohr-Coulomb
Unit Weight: 100 pcf	Unit Weight: 80 pcf	Unit Weight: 107 pcf
Cohesion: 0 psf	Cohesion: 0 psf	Cohesion: 0 psf
Phi: 34 °	Phi: 32 °	Phi: 30 °
Piezometric Line: 1	Piezometric Line: 1	

Pseudostatic Kh = 0.12g



Project: AEP Big Sandy Landfill Investigation	Log of Boring PB-8
Project Location: Louisa, KY	
Project Number: 13815141.10000	Sheet 1 of 7

Date(s) Drilled	4/20/12,4/23/12-4/25/12	Logged By	T. George	Checked By	
Drilling Method	HSA, Mud rotary	Drill Bit Size/Type	4 1/4" ID/8" OD HSA, 4" tricore mud-rotary	Total Depth of Borehole	153.0 feet
Drill Rig Type	CME 55 Rubber Track ATV, Remote control	Drilling Contractor	Pennsylvania Drilling	Surface Elevation	674.0 ft above msl
Borehole Backfill	2" SCH 40 PVC riser grouted in place	Sampling Method(s)	Piston/Split-spoon	Hammer Data	140#/30" Drop Auto
Boring Location		Groundwater Level(s)	3.1 ft ATD		

Elevation, feet	Depth, feet	SAMPLES Type	Number	Sampling Resist. Blows/6" OR CORE% RQD	Recovery, %	Pocket Penetrometer (tsf)	Graphic Log	MATERIAL DESCRIPTION	Water Content %	REMARKS AND OTHER DETAILS
										3.0' stickup
	0							Bottom ash splitter dike [BOTTOM ASH]		Grout
	1									SCH 40 PVC 2" diameter riser
	2									
	3								▽	*Drilled to 13' bgs without sampling*
670	4									
	5									
	6									
	7									
	8									
665	9									
	10									
	11									
	12									
	13							Very loose, wet, dark gray, bottom ash as coarse to fine SAND (SP-SM) trace gravel [BOTTOM ASH]		*Split-spoon @ 13' driven with 1 blow to 16'*
660	14		SS-1	1 0 0 0	100					*Bottom ash splitter dike*
	15									
	16									
	17									
	18									*~6" heave noted*
655	19		SS-2	1 0 1 0	100					
	20									

URS

Elevation, feet	Depth, feet	Type	Number	Sampling Resist. Blows/6" OR CORE% RQD	Recovery, %	Pocket Penetrometer (tsf)	MATERIAL DESCRIPTION	Water Content %	REMARKS AND OTHER DETAILS
	20								
	21								
	22						becomes loose		
	23	SS	SS-3	5 5 3 1	75				
							Very loose, wet, gray fly ash as fine silty SAND (SM) [FLY ASH]		*Bottom of splitter dike @ 23.5' bgs*
650	24								
	25								
	26								
	27								*Sample @ 27-29' fell 6" to 29.5' bgs*
	28	SS	SS-4	1 0 1 1					
645	29								
	30								
	31								
	32								
	33	P	P-1		88		becomes very loose, wet, gray, SILT (ML) with fine sand		
640	34								*Split-spoon @ 34'-36' WOR from 34'-41' bgs*
	35	SS	SS-5	WOR 0 0 0	0				
	36								
	37								
	38								
635	39								
	40								
	41								
	42						becomes light gray, interbedded with minor silty sand (SM)		*Split-spoon @ 42-44' 1 blow drives spoon 4 ft to 46' bgs*
	43	SS	SS-6	1 0 0	8				

Project: AEP Big Sandy Landfill Investigation
Project Location: Louisa, KY
Project Number: 13815141.10000

Log of Boring PB-8

Elevation, feet	Depth, feet	Type	Number	Sampling Resist. Blows/6" OR CORE% RQD	Recovery, %	Pocket Penetrometer (tsf)	Graphic Log	MATERIAL DESCRIPTION	Water Content %		REMARKS AND OTHER DETAILS
		SS	SS-6	0	8						
630	44										
	45										
	46							becomes mostly silty sand (SM) with interbedded sandy silt (ML)			
	47	SS	SS-7	WOR 0 0 0	100						*At 47-49' bgs rods fell 13' from 47-60' bgs*
	48										
625	49										
	50										
	51										
	52										
	53										
620	54										
	55										
	56										
	57										
	58										
615	59										
	60										
	61										
	62	P	P-2		92						
	63										
610	64	SS	SS-8	WOR 0 0 0	100						*Split-spoon @ 64-66' fell to 67' bgs*
	65										
	66										

Project: AEP Big Sandy Landfill Investigation	Log of Boring PB-8
Project Location: Louisa, KY	
Project Number: 13815141.10000	

Elevation, feet	Depth, feet	SAMPLES Type	Number	Sampling Resist. Blows/6" OR CORE% RQD	Recovery, %	Pocket Penetrometer (tsf)	Graphic Log	MATERIAL DESCRIPTION	Water Content %		REMARKS AND OTHER DETAILS
	67		SS-9	WOH, 0, 0, 0	33						Roller bit to 67'
605	69										
	70							Very loose, wet, dark gray bottom ash as coarse to medium SAND (SP-SM), trace gravel [BOTTOM ASH]			
	72		SS-10	1, 1, 1, 1	33			Very loose, wet, gray fly ash as fine silty SAND (SM) [FLY ASH]			
600	74										Drill rods clogged. Remove and flush.
	77		P-3		88						Bottom of piston tube is fly ash as sandy silt (ML)
595	79		SS-11	WOR, 0, 0, 0	0						
590	84										
	87		P-4		88						
585	89		SS-12	WOR, 0	0						Split-spoon @ 89-91' fell to 91.5' bgs
	90										

Elevation, feet	Depth, feet	Sample Type	Sample Number	Sampling Resist. Blows/6" OR CORE% RQD	Recovery, %	Pocket Penetrometer (tsf)	Graphic Log	MATERIAL DESCRIPTION	Water Content %	REMARKS AND OTHER DETAILS
			SS-12	0 0	0					
	91									
	92									
	93									
580	94									
	95									
	96									
	97									
	98		P-5		96					*Bottom of piston tube is fly ash as silty sand (SM)*
575	99							becomes loose, interbedded light and dark gray, medium to fine SAND (SP-SM) to silty SAND (SM), with minor interbedded sandy silt [FLY ASH]		
	100		SS-13	2 2 3 2	63					
	101									
	102									
	103									
570	104							becomes very loose, mostly sandy SILT (ML) to silty SAND (SM) with minor interbedded (SP-SM)		
	105									
	106									
	107									
	108		SS-14	1 0 0 1	79					
565	109									
	110									
	111									
	112							becomes mostly silty sand (SM) with minor interbedded sandy silt (ML)		
	113		SS-15	2 1 0	58					

Project: AEP Big Sandy Landfill Investigation
Project Location: Louisa, KY
Project Number: 13815141.10000

Log of Boring PB-8

Elevation, feet	Depth, feet	Type	Number	Sampling Resist. Blows/6" OR CORE% RQD	Recovery, %	Pocket Penetrometer (tsf)	MATERIAL DESCRIPTION	Water Content %	REMARKS AND OTHER DETAILS
560	114	SS	SS-15	1	58				
	115								
	116								
	117								
	118	P	P-6		96				
555	119						becomes mostly sandy silt (ML)		
	120	SS	SS-16	WOR 0 0 0	13				
	121								
	122								
	123								
550	124								
	125								
	126						becomes light gray with interbedded grayish brown mostly sandy SILT (ML) with minor interbedded silty sand, trace decayed plant stems		
	127								*Split-spoon at 127-129' fell to 131' bgs*
	128	SS	SS-17	WOR 0 0 0	88				
545	129								
	130								
	131								*Roller bit dropped to 132' when reinserted at 127'*
	132								
	133						Dense, wet, dark gray, medium to fine silty SAND (SM) with brown sandstone gravel [ALLUVIUM]		*Material is possibly a fill*
540	134								
	135								
	136								

Elevation, feet	Depth, feet	Type	Number	Sampling Resist. Blows/6" OR CORE% RQD	Recovery, %	Pocket Penetrometer (tsf)	Graphic Log	MATERIAL DESCRIPTION	Water Content %		REMARKS AND OTHER DETAILS
	137–139		SS-18	20, 17, 18, 21	79						
535	139										
	140							becomes moist, variably brown with gray mottling, trace greenish-gray, trace brownish-red			
	142–144		SS-19	19, 17, 26, 31	79						
530	144										
	147–149		SS-20	21, 2, 1, 11	25			No material in sampler representative of blow counts @ 147.5-149	14.3		Split-spoon at 147': 6" recovery appears the same as sample @ 142'. Blow counts may not be representative of material. %G=31.4 %S=49.3 %F=19.3
525	149										
	150										Drill change at 150'
	151							becomes with trace decayed vegetation			
	152–153		SS-21	15, 50/1"	100			Micaceous, silty sandstone, light gray, slightly weathered, weak to medium strong			Set PVC casing @ 152.5 ft bgs. Cement-bentonite grout placed using tremie pipe.
	153							End of Boring at 153' bgs			
520	154										
515	159										
	160										

SUMMARY FOR STATIC CIU' TRIAXIAL TESTS SPECIMENS

Test No	Boring No	Sample Section No	Depth / Elev ()	USCS Group Symbol / Gs	w_o / w_c (%)	$\gamma_{t,o}$ / $\gamma_{t,c}$ (pcf)	$\gamma_{d,o}$ / $\gamma_{d,c}$ (pcf)	$\sigma'_{c,max}$ (psi) / OCR	$\sigma'_{v,c}$ (psi) / $K_c = \frac{\sigma'_{v,c}}{\sigma'_{h,c}}$	$\varepsilon_{a,c}$ / $\varepsilon_{v,c}$ (%)	B factor (%) / ε_{rate} (%/hr)	at Peak Deviator Stress / at Peak Obliquity: ε_a (%)	$\frac{\sigma_1 - \sigma_3}{2}$ (psi)	$\frac{\sigma'_1 + \sigma'_3}{2}$ (psi)	σ'_1 / σ'_3	A factor	ϕ' for c'=0
T3274	Coarse	B		ML	37.8	101.24	73.47	5.00	5.00	0.6		18.7	23.07	46.05	3.01	-0.390	30.1
	Composite			(2.35)	36.9	107.55	78.55	1.0	1.00	6.5	1.2	2.0	0.36	0.46	7.57	6.879	50.1
T3275	Coarse	C		ML	37.9	102.81	74.58	10.00	10.00	0.8	99.5	18.8	5.85	11.77	2.98	0.349	29.8
	Composite			(2.35)	36.9	107.55	78.54	1.0	1.00	5.1	1.1	7.9	4.01	7.67	3.19	0.790	31.5
T3276	Coarse	D		ML	41.6	100.18	70.77	30.00	30.00	0.8	99.6	19.5	11.09	21.99	3.03	0.861	30.3
	Composite			(2.35)	39.6	106.09	76.00	1.0	1.00	6.9	1.1	9.0	8.82	16.82	3.20	1.247	31.6
T3273	Coarse	A		ML	41.9	100.83	71.04	50.00	50.00	2.1	99.2	19.0	20.33	40.33	3.03	0.738	30.3
	Composite			(2.35)	38.5	106.69	77.06	1.0	1.00	7.8	1.1	12.1	17.56	34.15	3.12	0.951	30.9

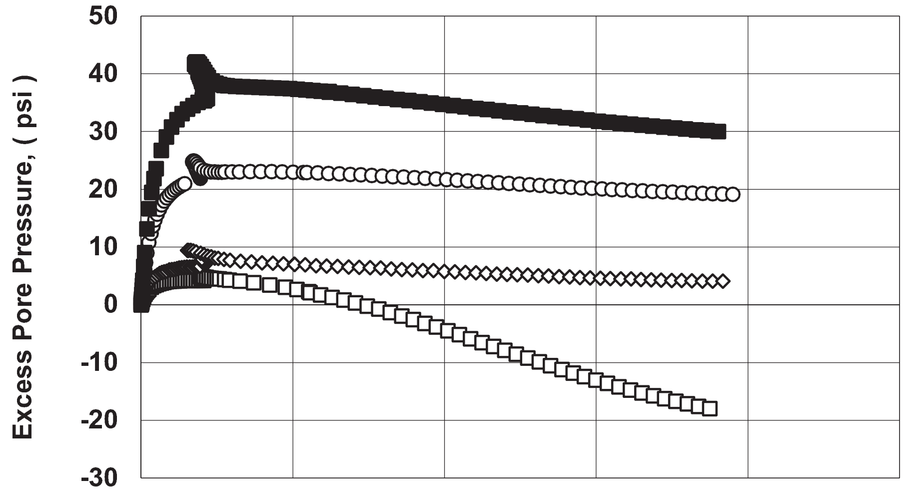
Test No	Description of Material Tested and Remarks
T3274	ML, Gray silt with sand
T3275	ML, Gray silt with sand
T3276	ML, Gray silt with sand
T3273	ML, Gray silt with sand
	Specimens unloaded at 2% strain then shear continued

Strength Envelope Summary

Test Series	Failure Criteria	ϕ' (deg)	c' (psi)	α' (deg)	a' (psi)	Correlation Coefficient
1	1	30.2	0.000	26.7	0.000	--
	2	30.8	0.152	27.1	0.131	1.000

Failure Criteria: 1 - Peak Deviator Stress; 2 - Peak Obliquity

Project No. T13815141	**AEP Big Sandy Landfill** **URS Corporation**	CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION with Pore Pressure Measurements Coarse Composite SUMMARY	June 2012
TerraSense, LLC			



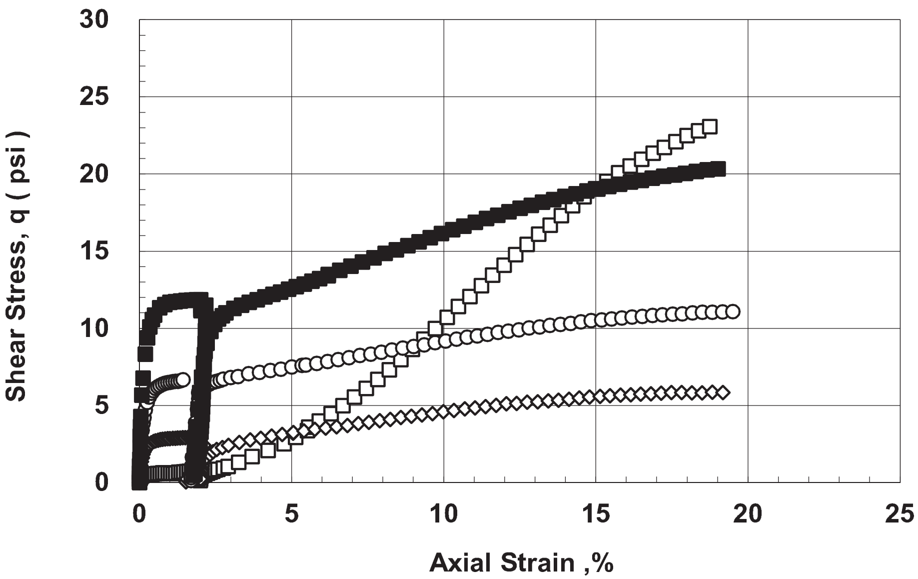
LEGEND AND SUMMARY INFORMATION

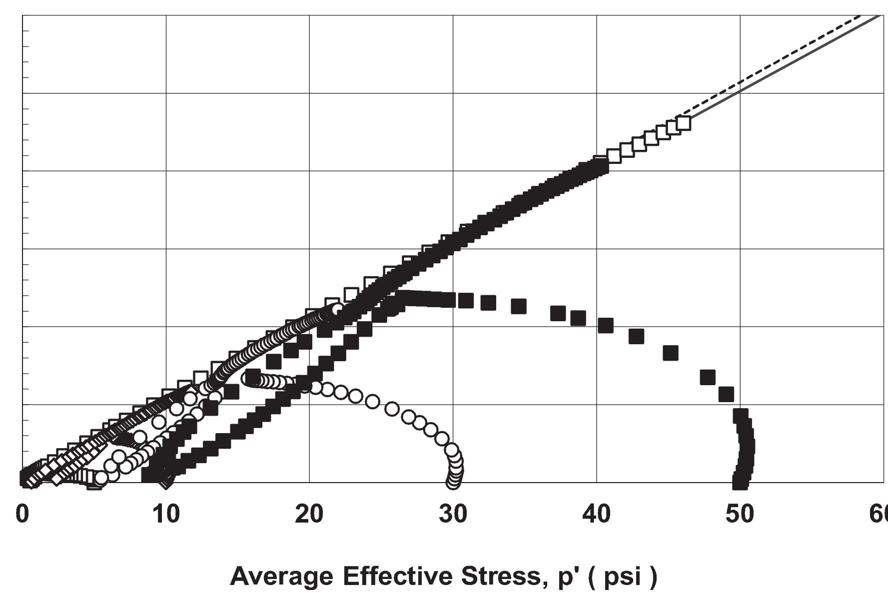
Symbol	Test	Sample	Specimen	w_o (%)	γ_{to} (pcf)	σ'_c (psi)
□	T3274	Coarse Composite	B	37.8	101.2	5.0
◊	T3275	Coarse Composite	C	37.9	102.8	10.0
○	T3276	Coarse Composite	D	41.6	100.2	30.0
■	T3273	Coarse Composite	A	41.9	100.8	50.0

Specimens unloaded at 2% strain then shear continued

SERIES SUMMARY

Notation	Failure Criteria	c' (psi)	Φ' (degrees)
solid line	Peak Deviator Stress	0.00	30.2
dotted line	Peak Obliquity	0.15	30.8

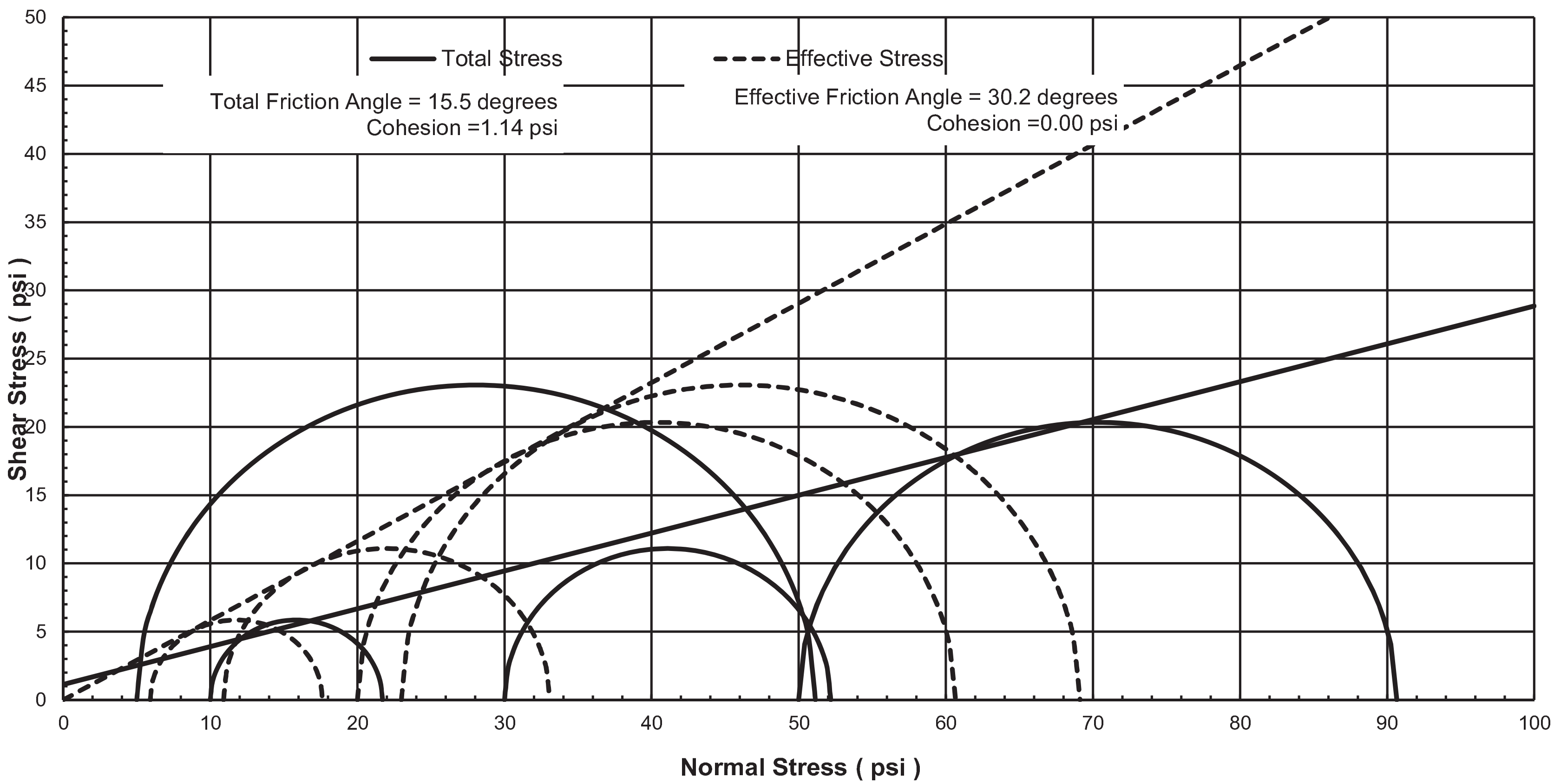




Prepared by: C. Jordan Checked by: G. Thomas	Project No. T13815141	**AEP** **URS Corporation**	CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION with Pore Pressure Measurements Coarse Composite SUMMARY	Figure 1 June 2012
	TerraSense, LLC			

Analysis File: Cu'sum4v4.xls

6/23/2012 Test: Cu'suma.xlsx



Test at 5-psi effective consolidation stress excluded from Total Friction strength envelope

Project No. T13815141	AEP URS Corporation	Mohr Circles of Total and Effective Stresses at Peak	Figure 2
TerraSense, LLC		CIU' Triaxial Test Coarse Composite SUMMARY	June 2012

CLOSURE CAP SHALLOW TRANSLATIONAL

SLOPE STABILITY ANALYSES

BIG SANDY POWER PLANT ASH POND CLOSURE PROJECT

GEOTECHNICAL CALCULATIONS

I. PURPOSE

The purpose of this analysis is to evaluate the veneer stability of the final cover system Big Sandy Fly Ash Pond by evaluating shallow translational failure potential of the cover system. There are no prescriptive standards related to slope stability in Kentucky Special Waste Solid Waste Regulations. These calculations are being provided to evaluate the stability of the cover system design proposed for the facility.

II. SITE AND PROJECT DESCRIPTION

The shallow translational failure analysis was performed as part of the Special Waste Permit Application for the closure of the Big Sand Fly Ash Pond located in Louisa, Kentucky. The proposed grades are generally shallow (approx. 2%). The slopes of the berm used to separate the revised Saddle Dam drainage area from the Main Dam drainage area incorporates the 4H:1V slope. This area is currently anticipated to receive a compacted soil cap system, but was analyzed for the geosynthetic cap system in the event that conditions during construction warrant the geosynthetic cap in lieu of the soil cap. The following sections summarize the methodology, assumptions, and results of the shallow translational failure analysis. For further detail on the specific calculations performed, refer to the corresponding data provided in the Attachments.

III. SHALLOW TRANSLATIONAL FAILURE ANALYSIS – METHODOLOGY

Analysis of the sliding potential of relatively thin cover soil layers (veneer) above both geosynthetic and natural soil liners, i.e. geomembranes (GM), geosynthetic clay liners (GCL) and compacted soil liners is important. This is because the underlying barrier materials generally represent a low interface shear strength boundary with respect to the soil placed above them and the geosynthetics are oriented precisely in the direction of potential sliding.

The method used in this analysis closely follows the methods outlined by Koerner and Soong (Koerner and Soong, 2005) and is performed by use of force equilibrium to balance the driving forces due to gravity pulling on the cover soils and the resistance to sliding due to friction between the underlying subsurface and cover material. Resistance to sliding is also due in part to the toe support (passive wedge) located at the base of the sliding mass. This is conceptually illustrated in **Figure 1** below.

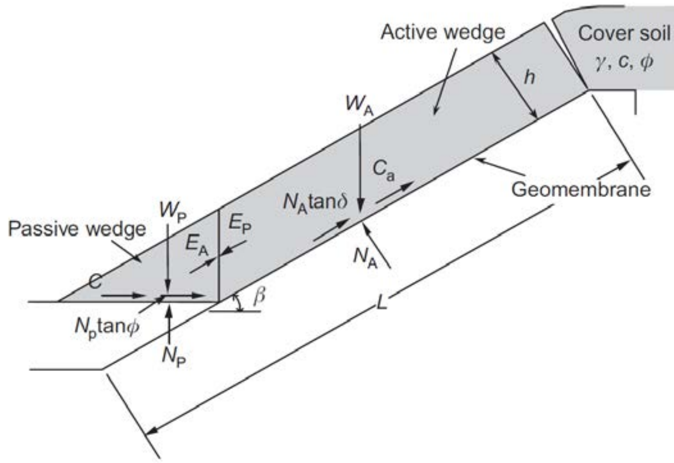


Figure 1. Conceptual Veneer Stability Analysis Cross Section/Free Body Diagram

Where,

W_A	=	Total Weight of the Active Wedge
W_P	=	Total Weight of the Passive Wedge
N_A	=	Effective Force Normal to the Failure Plane of the Active Wedge
N_P	=	Effective Force Normal to the Failure Plane of the Passive Wedge
γ	=	Unit Weight of the Cover Soil
t	=	Thickness of the Cover Soil
L	=	Length of Slope Measured Along the Geomembrane
β	=	Soil Slope Angle Beneath The Geomembrane
ϕ	=	Friction Angle Of The Cover Soil
δ	=	Interface Friction Angle Between Cover Soil and Geomembrane
C_A	=	Adhesive Force Between Active Wedge Cover Soil and Geomembrane
c_A	=	Adhesion Between Active Wedge Cover Soil and the Geomembrane
C	=	Cohesive Force Along The Failure Plane Of The Passive Wedge
c	=	Cohesion of the Cover Soil
E_A	=	Interwedge Force Acting on the Active Wedge from the Passive Wedge
E_P	=	Interwedge Force Acting on the Passive Wedge from the Active Wedge
FS	=	Factor of Safety Against Cover Soil Sliding on the Geomembrane

The shallow translational failure analysis is analyzed by fully satisfying the equilibrium of forces in the vertical and horizontal directions. By taking force summation parallel to the slope and comparing the resisting force with the driving or mobilizing force, a global factor of safety (FS) results:

$$\text{FS}=\frac{\sum \text{Resisting Forces}}{\sum \text{Driving Forces}}$$

As noted in the procedure contained in the Koerner and Soong paper, FS for veneer stability as depicted in Figure 2 is determined by solving the following quadratic equation:

$$FS = \frac{-b + \sqrt{b^2 - 4ac}}{2a}$$

Where,

$$a = (W_A - N_A cos\beta)cos\beta$$

$$b = -\begin{bmatrix} (W_A - N_A cos\beta)sin\beta tan\varphi \\ +(N_A tan\delta + C_a)sin\beta cos\beta \\ +sin\beta(C + W_P tan\phi) \end{bmatrix}$$

$$c = (N_A tan\delta + C_a)sin^2\beta tan\phi$$

When the calculated FS-value falls below 1.0, sliding of the cover soil on the geomembrane is to be anticipated.

For this analysis, the following conditions and factors of safety were used based on the anticipated probability and consequences of failure:

- Static Conditions (Peak Strength): FS ≥ 1.50
- Static Conditions (Residual Strength): FS ≥ 1.10
- Static Conditions (Full Drainage Layer): FS ≥ 1.10
- Static Conditions (Equipment Loads): FS ≥ 1.25
- Seismic Conditions: FS ≥ 1.00

The static/peak strength condition corresponds to a long term condition. Typically slope stability analyses require a static factor of safety between 1.4 and 2.0 (or potentially higher if conditions warrant it). A factor of safety of 1.5 is commonly used for landfill cap and liner system design for long term static conditions. The longest and steepest geometric slope will occur during interim conditions, therefore; a factor of safety of 1.3 was presented in the attached calculations in lieu of the long term evaluation. Mobilizing residual strength in the engineered components is unlikely, but the design was also analyzed for residual strength conditions where applicable. Because this condition represents a post-peak strength event and the strength is the reasonable low limit for strength of the interface a lower factor of safety. In addition, a residual FS of greater than 1.1 would allow the slope to restabilize after the unforeseen event, limiting further damage. The design was also analyzed for static conditions under high seepage forces in the drainage layer where the drainage layer is at full capacity. This condition is 7temporary based on the frequency of the design storm used; therefore, a lower factor of safety is warranted. Other loads accounting for equipment on the side slope and seismic forces were similarly analyzed representing temporary events warranting commensurate lower factors of safety.

Note that this analysis for equipment load only accounts for the weight of the vehicle and assumes very small and gradual acceleration and deceleration on the slope such that it can be neglected. It also assumes placement of the material beginning from the toe of slope progressing to the top.

IV. SELECTION OF PARAMETERS

For the final cover system, the following assumptions and design parameters were used. Slope lengths and angles used in all analyses correspond to the maximum (i.e. worst case) values corresponding to each respective system.

a. Slope Geometry

The final cover system will have a maximum slope of 4H:1V. Based on the final cover design grades, the maximum slope length is approximately 120 feet.

b. Layers

Layers and layer thicknesses for the cover system is anticipated as follows:

Table 1. Layer Summary for the Liner and Final Cover System

THICKNESS	LAYER
Final Cover	
6 in	Vegetative Cover Soil
18 in	Protective Cover Soil
n/a	Geocomposite Drainage Layer
n/a	PVC Geomembrane
Varies	Contouring Fill Material

c. Critical Interfaces

The critical interfaces analyzed represent preferential pathways for mass sliding. Critical interfaces in the cover or liner system are typically between adjacent geosynthetic materials or between geosynthetic and soil materials. The geosynthetics are of negligible thickness so the depth to the failure surface does not require adjustment for the individual components when they are stacked.

For the final cover system, the following interfaces represent the critical interfaces analyzed:

- Protective Soil/Geocomposite Drainage Layer
- Geocomposite Drainage Layer/PVC Geomembrane
- PVC Geomembrane/Contouring Fill

For each system, a minimum shear strength envelop was determined for each condition. The most conservative envelope developed from the various conditions is anticipated to be used to develop material specifications for use in construction.

d. Material Parameters

The various material parameters used in the veneer analyses are tabulated in the table below.

Table 2 Final Cover Variables

PARAMETER	VALUE	COMMENTS
Final Cover		
Dry Unit Weight of Veg. Cover Soil, $\gamma_{D\text{-}VC}$	100 pcf	Assumed (typical of veg. material)
Moisture Content of Veg. Cover Soil, $w_{F\text{-}VC}$	20%	Assumed (typical of veg. material)
Dry Unit Weight of Slope Cover Soil, $\gamma_{D\text{-}CS}$	100 pcf	Typical unit weight of in-situ lab samples
Moisture Content of Slope Cover Soil, $w_{F\text{-}CS}$	20%	Typical moisture of in-situ lab samples
Specific Gravity of Slope Cover Soil, G_S	2.7	Assumed (typical of clay)
Friction Angle of Slope Cover Soil, ϕ	25 deg	Conservatively assumed cover soil
Cohesion of Slope Cover Soil, c	100 psf	strength parameters
Equipment Loads		
Weight of Construction Equip (W_b)	39,918 lbs	CAT D6 LGP Dozer
Length of Equip. Track (w)	10.2 ft	CAT D6 LGP Dozer
Width of Equip. Track (b)	2.8 ft	CAT D6 LGP Dozer
Seismic Loads		
Seismic Coefficient (Cover)	0.09g	Seismic Coefficient Calc.

V. RESULTS OF ANALYSIS

The analysis consists of finding strength parameters (friction angle, ϕ and cohesion, c) for a given interface, to meet required factors of safety against translational failure. In the case of geosynthetic interfaces, the term adhesion is used instead of cohesion. Cohesion strictly relates to clay particle interaction, whereas adhesion refers to the physical interaction of geosynthetic surfaces. Mathematically, these terms are interchangeable.

The results of the analysis are summarized below. Detailed calculations are included as **Attachment A**. The minimum peak and residual shear strength parameters for each condition are included in graphical form in **Attachment B**.

a. Final Cover System Peak Shear Strength Parameters

The minimum required shear strength parameters for the final cover system are as follows assuming cohesion/adhesion is equal to zero. Additional minimum values where cohesion/adhesion are non-zero are included in **Attachment B.**

Table 3. Final Cover Minimum Calculated Required Peak Friction Angles

CONDITION	PEAK INTERFACE FRICTION, ϕ (DEGREES)*	TARGETED FACTOR OF SAFETY	CALCULATED FACTOR OF SAFETY
Static	17.8	1.50	1.50
Static (Full Drainage Layer)	17.8	1.10	1.33
Static (Equipment Loads)	17.8	1.25	1.46
Seismic	17.8	1.00	1.08

*Calculations of friction angles assume interface adhesion(c) is equal to zero.

b. Final Cover System Residual Shear Strength

The acceptable material specifications are anticipated to be based on peak strength of the materials. However, should the cover or liner system be temporarily acted upon by an outside force that causes the post peak strength to be mobilized, there may be some displacement of the cap system. Movement will occur along the interface with the lowest peak strength. This will then mobilize the residual/large-displacement strength of that particular interface. If residual strength for the interface with the lowest peak strength is above the minimum shear strength envelope as depicted in **Attachment B**, the cap system is anticipated to stabilize after the temporary loading condition ends.

Preconstruction testing with actual materials is anticipated to be conducted to verify that the materials used exhibit interface properties above the minimum shear strength envelope.

Preconstruction testing with actual materials is anticipated to be conducted to verify that the materials used exhibit interface properties above the minimum shear strength envelope.

The minimum shear strength parameters assuming residual strength is mobilized for the final cover is as follows assuming cohesion/adhesion is equal to zero. Additional minimum values where cohesion/adhesion are non-zero are included in **Attachment B.**

Table 4. Bottom Liner Minimum Calculated Friction Angles

CONDITION	RESIDUAL INTERFACE FRICTION, ϕ (DEGREES)*	FACTOR OF SAFETY
Final Cover Static Residual	12.4	1.10

*Calculations of friction angles assume interface adhesion (c) is equal to zero.

VI. REFERENCES

Koerner, R. M. (2005) Designing with Geosynthetics, 5th edn. Prentice Hall, Englewood Cliffs, NJ, 801 pp.- 1445.

Koerner, R. M. and Soong, T.-Y. (2005), "Analysis and Design of Veneer Cover Soils,". Geosynthetics International, Vol. 12, No. 1, pp. 28-49.

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Richardson, G. and Zhao, A., Design of Lateral Drainage Systems for Landfills, Tenax Corp., Baltimore, MD, 2000.

ATTACHMENT A

URS Corporation	American Electric Power	JOB	13815151		
564 White Pond Drive		SHT NO	1	OF	11
Akron, OH 44320	VENEER STABILITY (FINAL COVER)	CALC BY	NSG	DATE	05/01/13
Tel. (330) 836-9111	Big Sandy Ash Pond Closure	CHK BY	JLM	DATE	5/7/13
Fax (330) 836-9115		SCALE	NA		

Objective:

Determine the veneer stability of the cover system at the Big Sand Ash Pond Closure

Method:

Use methods outlined in the paper by Koerner and Soong, Analysis and *Design of Veneer Cover Soils* published in **Geosynthetics International,** 2005, 12, No.1.

Procedure:

Determine the static stability of the veneer cover system to evaluate the minimum required interface friction angle for all engineered components of the cover system. Balance the forces as shown in **Figure 1** and the required factor of safety (FS) then solve for minimum interface shear strength parameters.

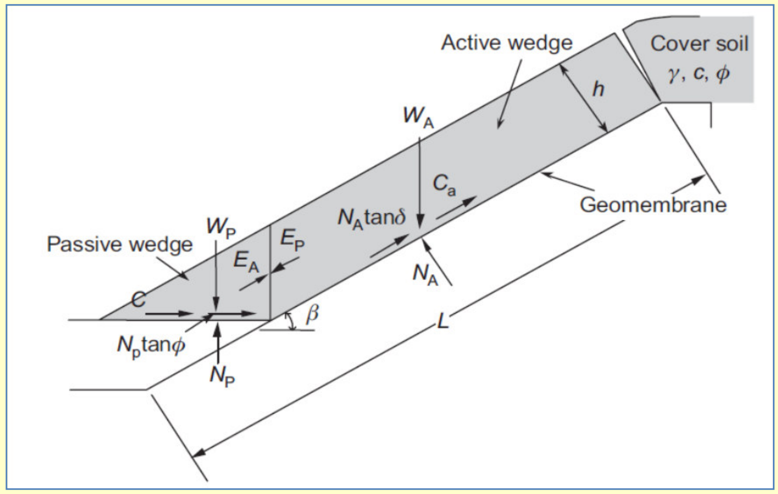

Figure 1. Limit equilibrium forces involved in a finite length slope analysis for a uniformly thick cover soil

Determine: Static factor of safety for cover system based on gravitational forces only and peak strength.

Assumptions:

1. No geosythetic reinforcements
2. No interface adhesion for geosynthetic components.
3. No tension allowed in geosynthetics
4. Minimum cohesion for multilayered systems
5. Weighted average friction angle for multilayered systems in passive wedge.

Veg. Cover Soil (VC)/Prot. Cover Soil (CS) and Slope Parameters	VC	CS	Utilized		Source
Thickness (h)	0.5	1.5	2.0	ft	Design - see Sheet 18
Dry Unit Weight (γ_D)	100.0	100.0	100.0	pcf	Assumed values based on lab testing
Mois. Cont. (field cond.) (w_F)	20.0	20.0	20.0	%	Assumed values based on lab testing
Avg Field Unit Wt (γ)	-	-	120.0	pcf	
Reference Stress	-	-	**232.8**	**pcf**	
Min. Friction Angle (ϕ)	25.0	25.0	25.0	deg	Assumed (conservative for clay)
Min. Cohesion (c)	100.0	100.0	100.0	psf	Assumed (conservative for clay)
Slope Angle Beneath the Geom. (β)	-	-	14.03	deg	Design - see Sheet 10
Ht. of Slp. Meas. Along Geom. (H_L)	-	-	30.00	ft	Design - see Sheet 10
Lng. of Slp. Meas. Along Geom. (L)	-	-	123.75	ft	
Inter. Frict. Angle for DL & Geom. (δ)	-	-	17.8	deg	Min. req. shear strength parameters - see CQA Plan for testing reqs.
Adhesion for DL & Geom. (c_a)	-	-	0.0	psf	
Required Factor of Safety (FS_R)	-	-	1.50		Min. req. FS for long term conditions

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Active Wedge Calculations

Determine the total weight of the active wedge (W_A),the effective force normal to the failure plan of the active wedge (N_A), the adhesive force between the cover soil of the active wedge and the geomembrane (C_a), and the interwedge force acting on the active wedge from the passive wedge (E_A) using the following eqs:

$$W_A = \gamma h^2\left(\frac{L}{h} - \frac{1}{\sin\beta} - \frac{\tan\beta}{2}\right)$$ = **27,659 lbs**

$$N_A = W_A \cos\beta$$ = **26,834 lbs**

$$C_a = c_a\left(L - \frac{h}{\sin\beta}\right)$$ = **- lbs**

$$E_A = \frac{(FS)(W_A - N_A\cos\beta) - (N_A\tan\delta + C_a)\sin\beta}{\sin\beta(FS)}$$ = **962 lbs**

Passive Wedge Calculations

Determine the total weight of the passive wedge (W_P),the effective force normal to the failure plan of the passive wedge (N_P), the cohesive force along the failure plane (C), and the interwedge force acting on the passive wedge from the active wedge (E_P) using the following eqs:

$$W_P = \frac{\gamma h^2}{\sin 2\beta}$$ = **1,020 lbs**

$$C = \frac{ch}{\sin\beta}$$ = **825 lbs**

$$E_P = \frac{C + W_P\tan\phi}{\cos\beta(FS) - \sin\beta\tan\phi}$$ = **969 lbs**

$$N_P = W_P + E_P\sin\beta$$ = **1,255 lbs**

Static Factor of Safety (Solved for Iteratively)

Determine the calculated Factor of Safety (FS_A) using a quadratic equation relationship where the constants are defined as follows:

$$a = (W_A - N_A\cos\beta)\cos\beta$$ = **1,577 lbs/ft**

$$b = -\begin{bmatrix}(W_A - N_A\cos\beta)\sin\beta\tan\phi \\ +(N_A\tan\delta + C_a)\sin\beta\cos\beta \\ +\sin\beta(C + W_P\tan\phi)\end{bmatrix}$$ = **(2,525) lbs/ft**

$$c = (N_A\tan\delta + C_a)\sin^2\beta\tan\phi$$ = **236 lbs/ft**

Results Have Converged		
FS_R =	**1.50**	
FS_A =	**1.50**	
Min. Peak δ	**17.8**	**deg**
Min. Peak c_a	**0.0**	**psf**

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Determine: Static factor of safety for cover system based on additional seepage forces.

Procedure:

Determine the static stability of the veneer cover system to determine the minimum required interface friction angle for all engineered components of the cover system. Balance the forces as shown in Figure 1 and the required factor of safety (FS) then solve for minimum interface shear strength parameters. Account for seepage forces in drainage layer as noted in **Figure 2**.

Assumptions:

In addition to the static case assumptions:

1. Seepage is parallel to slope
2. The drainage layer is sized such that liquid not build up beyond the thickness of the drainage layer.
3. Drainage layer has adequate capacity to handle maximum surface water flow
4. If geocomposite is used - it is less than 0.75 inches thick.
5. Max accumulation of up to 1 foot head on top of FML barrier to account for drainage aggregate in lieu of geocomposite.

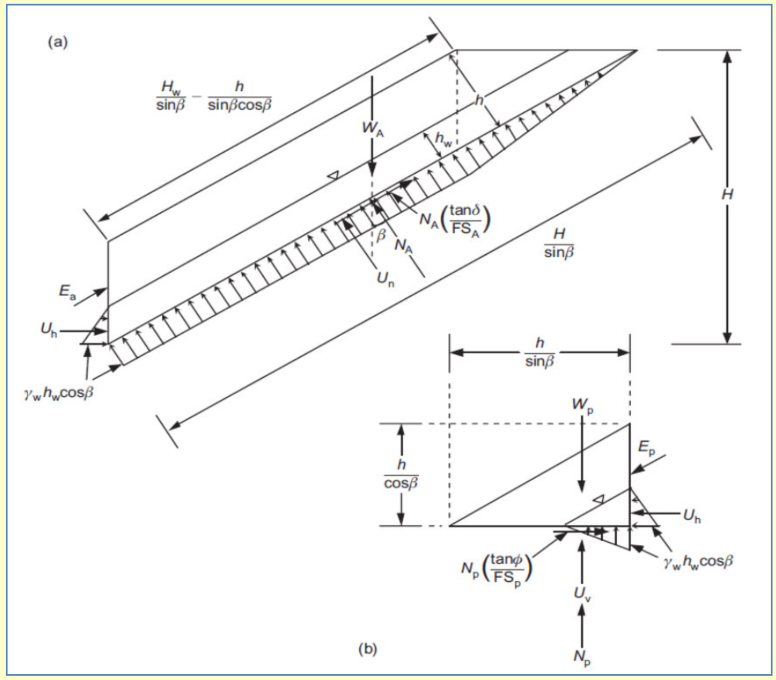

Figure 2. Limit equilibrium forces involved in finite-length slope of uniform cover soil with parallel-to-slope seepage build-up: (a) active wedge; (b) passive wedge

Veg. Cover Soil (VC)/Prot. Cover Soil (CS) and Slope Parameters	VC	CS	Utilized		Source
Thickness (h)	0.5	1.5	2.0	ft	Design - see Sheet 18
Dry Unit Weight (γ_D)	100.0	100.0	100.0	pcf	Assumed values based on lab testing
Mois. Cont. (field cond.) (w_F)	20.0	20.0	20.0	%	Assumed values based on lab testing
Avg Field Unit Wt (γ)	-	-	120.0	pcf	
Specfic Gravity of the (G_S)	2.70	2.70	2.70		**Assumed conservative**
Unit Weight of Water (γ_W)	-	-	62.4	pcf	
Saturated Unit Weight (γ_{SAT})	-	-	125.4	pcf	
Min. Friction Angle (ϕ)	25.0	25.0	25.0	deg	Assumed (conservative for clay)
Min. Cohesion (c)	100.0	100.0	100.0	psf	Assumed (conservative for clay)
Slope Angle Beneath the Geom. (β)	-	-	14.0	degrees	Design - see Sheet 10
Ht. of Slp. Meas. Along Geom. (H_L)	-	-	30.00	ft	Design - see Sheet 10
Lng. of Slp. Meas. Along Geom. (L)	-	-	123.7	ft	
Depth of Water in DL (h_W)	-	-	0.08	ft	**Max. thickness of DL (conservative)**
Inter. Frict. Angle for DL & Geom. (δ)	-	-	17.8	deg	Min. req. shear strength parameters - see CQA Plan for testing reqs.
Adhesion for DL & Geom. (c_a)	-	-	0.0	psf	
Required Factor of Safety (FS_R)	-	-	1.10		**Min. req. FS for temporary conditions**

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Active Wedge Calculations

Determine the total weight of the active wedge (W_A), resultant of the pore pressures acting on the interwedge surfaces (U_h), resultant of the pore pressures acting perpendicular to the slope (U_n), the effective force normal to the failure plan of the active wedge (N_A), and the interwedge force acting on the active wedge from the passive wedge (E_A) using the following eqs:

$$W_A = \frac{\gamma_D (h - h_W)[2H\cos\beta - (h + h_W)]}{\sin 2\beta} + \frac{\gamma_{SAT}(h_W)(2H\cos\beta - h_W)}{\sin 2\beta}$$ = **24,160 lbs**

$$U_h = \frac{\gamma_w h_W^2}{2}$$ = **0.22 lbs**

$$U_n = \frac{\gamma_w (h_w)(\cos\beta)(2H\cos\beta - h_w)}{\sin 2\beta}$$ = **623 lbs**

$$N_A = W_A \cos\beta + U_h \sin\beta - U_n$$ = **22,816 lbs**

$$E_A = W_A \sin\beta - U_h \cos\beta - \frac{N_A \sin\delta}{(FS)}$$ = **(484) lbs**

Passive Wedge Calculations

Determine the total weight of the passive wedge (W_P), resultant of the vertical pore pressures acting on the passive wedge (U_V), and and the interwedge force acting on the pass wedge from the active wedge (E_P) using the following eqs:

$$W_P = \frac{\gamma_D (h^2 - h_w^2) + \gamma_{SAT} h_w^2}{\sin 2\beta}$$ = **850.7 lbs**

$$U_V = U_h \cot\beta$$ = **0.9 lbs**

$$E_P = \frac{U_h (FS) - (W_P - U_V \tan\phi)}{\sin\beta \tan\phi - \cos\beta (FS)}$$ = **891 lbs**

Static Factor of Safety w/ Seepage Forces (Solved for Iteratively)

Determine the calculated Factor of Safety (FS_A) using a quadratic equation relationship where the constants are defined as follows:

$$a = W_A \sin\beta \cos\beta - U_h \cos^2\beta + U_h$$ = **5,682 lbs/ft**

$$b = -W_A \sin^2\beta \tan\phi + U_h \sin\beta \cos\beta \tan\phi - N_A \cos\beta \tan\delta - (W_P - U_V)\tan\phi$$ = **(8,165.4) lbs/ft**

$$c = N_A \sin\beta \tan\delta \tan\phi$$ = **828 lbs/ft**

Allow Exceeds Req'd - OK		
FS_R =	1.10	
FS_A =	1.33	
Min. Peak δ	17.8	deg
Min. Peak c_a	0.0	psf

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Determine: Static factor of safety for cover system based on additional equipment loads.

Procedure:

Determine the static stability of the veneer cover system to determine the minimum required interface friction angle for all engineered components of the cover system. Balance the forces as shown in Figure 1 and the required factor of safety (FS) then solve for minimum interface shear strength parameters. Account for equipment loads (W_b) as final cover is placed as noted in Figure 3.

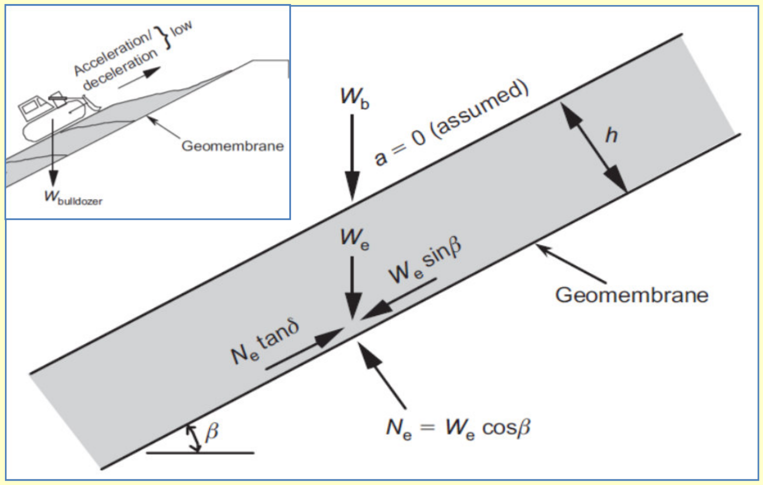

Figure 3. Additional load due to construction equipment moving on cover soil.

Assumptions:

In addition to the static case assumptions:
1. The equipment pushes material up slope leaving a toe buttress behind.
2. The equipment accelerates slowly with no sudden starts or turns to minimize additional loads besides the weight of the machine.

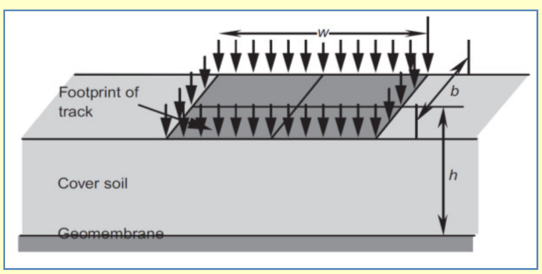

Figure 4. Illustration of stress distribution from overlying equipment.

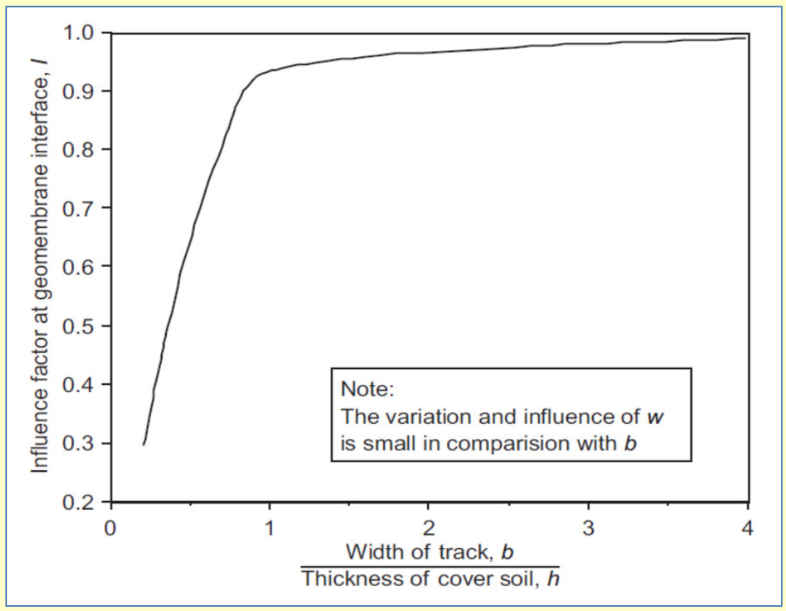

Figure 5. Values of influence factor I to dissipate surface force through cover soil to geomembrane interface (after Poulos and Davis 1974)

Equipment Parameters			Source
Equiv. Equipment Load per Unit Width (W_e)	6797	lbs	W_e = qwl
Influence Factor at the Geom. Interface (I)	0.95		See Figure 5 above.
Track Width to Cover Soil Thickness Ratio (b/h)	1.40		
Distributed Equipment Load (q)	699	psf	q = W_b/(2 × w × b)
Weight of Equipment (W_b)	39,918	lbs	**Typical weight of CAT D6 dozer**
Length of Equipment Track (w)	10.20	ft	**Typical track dimensions of CAT D6 dozer**
Width of Equipment Track (b)	2.80	ft	

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Veg. Cover Soil (VC)/Prot. Cover Soil (CS) and Slope Parameters

	VC	CS	Utilized		Source
Thickness (h)	0.5	1.5	2.0	ft	Design - see Sheet 18
Dry Unit Weight (γ_D)	100.0	100.0	100.0	pcf	Assumed values based on lab testing
Mois. Cont. (field cond.) (w_F)	20.0	20.0	20.0	%	Assumed values based on lab testing
Avg Field Unit Wt (γ)	-	-	120.0	pcf	
Min. Friction Angle (ϕ)	25.0	25.0	25.0	deg	Assumed (conservative for clay)
Min. Cohesion (c)	100.0	100.0	100.0	psf	Assumed (conservative for clay)
Slope Angle Beneath the Geom. (β)	-	-	14.0	deg	Design - see Sheet 10
Ht. of Slp. Meas. Along Geom. (H_L)	-	-	30.00	ft	Design - see Sheet 10
Lng. of Slp. Meas. Along Geom. (L)	-	-	123.75	ft	
Inter. Frict. Angle for DL & Geom. (δ)	-	-	17.8	deg	Min. req. shear strength parameters - see CQA Plan for testing reqs.
Adhesion for DL & Geom. (c_a)	-	-	0.0	psf	
Required Factor of Safety (FS_R)	-	-	**1.25**		**Min. req. FS for temporary conditions**

Active Wedge Calculations

Determine the total weight of the active wedge (W_A),the effective force normal to the failure plan of the active wedge (N_A), the adhesive force between the cover soil of the active wedge and the geomembrane (C_a), and the interwedge force acting on the active wedge from the passive wedge (E_A) using the following eqs:

$$W_A = \gamma h^2\left(\frac{L}{h} - \frac{1}{\sin\beta} - \frac{\tan\beta}{2}\right) + W_e = \textbf{34,457 lbs}$$

$$N_A = W_A \cos\beta = \textbf{33,429 lbs}$$

$$C_a = c_a\left(L - \frac{h}{\sin\beta}\right) = \textbf{- lbs}$$

$$E_A = \frac{(FS)(W_A - N_A\cos\beta) - (N_A\tan\delta + C_a)\sin\beta}{\sin\beta(FS)} = \textbf{(233) lbs}$$

Passive Wedge Calculations

Determine the total weight of the passive wedge (W_P),the effective force normal to the failure plan of the passive wedge (N_P), the cohesive force along the failure plane (C), and the interwedge force acting on the passive wedge from the active wedge (E_P) using the following eqs:

$$W_P = \frac{\gamma h^2}{\sin 2\beta} = \textbf{1,020 lbs}$$

$$C = \frac{ch}{\sin\beta} = \textbf{825 lbs}$$

$$E_P = \frac{C + W_P\tan\phi}{\cos\beta(FS) - \sin\beta\tan\phi} = \textbf{1,183 lbs}$$

$$N_P = W_P + E_P\sin\beta = \textbf{1,307 lbs}$$

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Static Factor of Safety w/ Equipment Load (Solved for Iteratively)

Determine the calculated Factor of Safety (FS_A) using a quadratic equation relationship where the constants are defined as follows:

$$a = (W_A - N_A \cos\beta)\cos\beta$$ = **1,965 lbs/ft**

$$b = -\begin{bmatrix} (W_A - N_A \cos\beta)\sin\beta\tan\phi \\ +(N_A \tan\delta + C_a)\sin\beta\cos\beta \\ +\sin\beta(C + W_P \tan\phi) \end{bmatrix}$$ = **(3,069) lbs/ft**

$$c = (N_A \tan\delta + C_a)\sin^2\beta\tan\phi$$ = **294 lbs/ft**

Allow Exceeds Req'd - OK

FS_R =	1.25	
FS_A =	1.46	
Min. Peak δ	17.8	deg
Min. Peak c_a	0.0	psf

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Determine: Static factor of safety for cover system based on additional seismic loads.

Procedure:

Determine the static stability of the veneer cover system to determine the minimum required interface friction angle for all engineered components of the cover system. Balance the forces as shown in Figure 1 and the required factor of safety (FS) then solve for minimum interface shear strength parameters. Account for seismic loads (C_s) as noted in Figure 6.

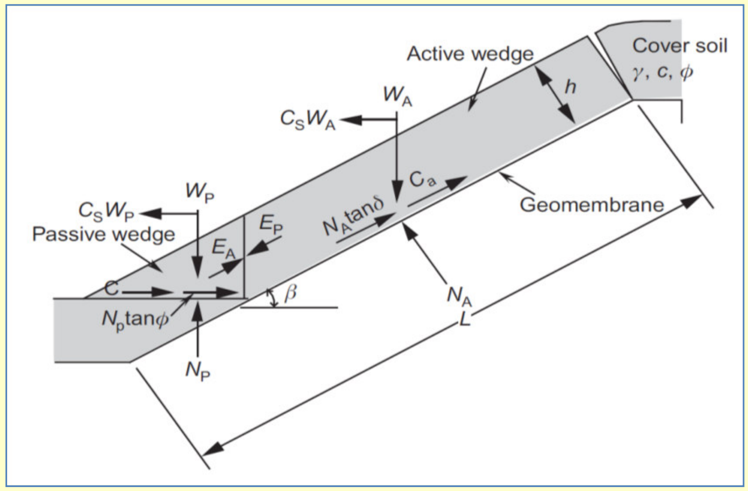

Figure 6. Limit equilibrium forces involved in pseudo-static analysis using average seismic coefficient.

Assumptions:

In addition to the static case assumptions:
1. Seismic force acts on the centroid of the cover soil.
2. Seismic force is horizontal.
3. Deformation analysis not required.

Veg. Cover Soil (VC)/Prot. Cover Soil (CS) and Slope Parameters	VC	CS	Utilized		Source
Thickness (h)	0.5	1.5	2.0	ft	Design - see Sheet 18
Dry Unit Weight (γ_D)	100.0	100.0	100.0	pcf	Assumed values based on lab testing
Mois. Cont. (field cond.) (w_F)	20.0	20.0	20.0	%	Assumed values based on lab testing
Avg Field Unit Wt (γ)	-	-	120.0	pcf	
Min. Friction Angle (ϕ)	25.0	25.0	25.0	deg	Assumed (conservative for clay)
Min. Cohesion (c)	100.0	100.0	100.0	psf	Assumed (conservative for clay)
Slope Angle Beneath the Geom. (β)	-	-	14.0	degrees	Design - see Sheet 10
Ht. of Slp. Meas. Along Geom. (H_L)	-	-	30.00	ft	Design - see Sheet 10
Lng. of Slp. Meas. Along Geom. (L)	-	-	123.75	ft	
Inter. Frict. Angle for DL & Geom. (δ)	-	-	17.8	deg	Min. req. shear strength parameters from static analysis
Adhesion for DL & Geom. (c_a)	-	-	0.0	psf	
Avg. Seismic Coeffcient (C_S or K_S)	-	-	0.090	%g	**from SHAKE analysis**
Required Factor of Safety (FS_R)	-	-	1.00		**Min. req. FS for seismic conditions**

Active Wedge Calculations

Determine the total weight of the active wedge (W_A),the effective force normal to the failure plan of the active wedge (N_A), the adhesive force between the cover soil of the active wedge and the geomembrane (C_a), and the interwedge force acting on the active wedge from the passive wedge (E_A) using the following eqs:

$$W_A = \gamma h^2\left(\frac{L}{h} - \frac{1}{\sin\beta} - \frac{\tan\beta}{2}\right)$$ = **27,659 lbs**

$$N_A = W_A \cos\beta$$ = **26,834 lbs**

$$C_a = c_a\left(L - \frac{h}{\sin\beta}\right)$$ = **- lbs**

$$E_A = \frac{(FS)(C_S W_A + N_A \sin\beta)}{(FS)\cos\beta} + \frac{(N_A \tan\delta + C_a)\cos\beta}{(FS)\cos\beta}$$ = **17,887 lbs**

Passive Wedge Calculations

Determine the total weight of the passive wedge (W_P),the effective force normal to the failure plan of the passive wedge (N_P), the cohesive force along the failure plane (C), and the interwedge force acting on the passive wedge from the active wedge (E_P) using the following eqs:

$$W_P = \frac{\gamma h^2}{\sin 2\beta}$$ = **1,020 lbs**

$$C = \frac{ch}{\sin\beta}$$ = **825 lbs**

$$E_P = \frac{C + W_P \tan\phi - C_S W_P (FS)}{\cos\beta (FS) - \sin\beta \tan\phi}$$ = **1,411 lbs**

$$N_P = W_P + E_P \sin\beta$$ = **1,362 lbs**

Seismic Factor of Safety (Solved for Iteratively)

Determine the calculated Factor of Safety (FS_A) using a quadratic equation relationship where the constants are defined as follows:

$$a = (C_S W_A + N_A \sin\beta)\cos\beta + C_S W_P \beta$$ = **8,801 lbs/ft**

$$b = -\begin{bmatrix} (C_S W_A + N_A \sin\beta)\sin\beta \tan\phi \\ + (N_A \tan\delta + C_a)\cos^2\beta \\ + (C + W_P \tan\phi)\cos\beta \end{bmatrix}$$ = **(10,388) lbs/ft**

$$c = (N_A \tan\delta + C_a)\cos\beta \sin\beta \tan\phi$$ = **945 lbs/ft**

Allow Exceeds Req'd - OK		
FS_R =	**1.00**	
FS_A =	**1.08**	
Min. Peak δ	**17.8**	**deg**
Min. Peak c_a	**0.0**	**psf**

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Determine: Static factor of safety for cover system based on residual strength.

Procedure:

Determine the static stability of the veneer cover system to evaluate the minimum required interface friction angle for all engineered components of the cover system. Balance the forces as shown in Figure 1 and the required factor of safety (FS) then solve for minimum interface shear strength parameters.

Assumptions:

No additional assumptions

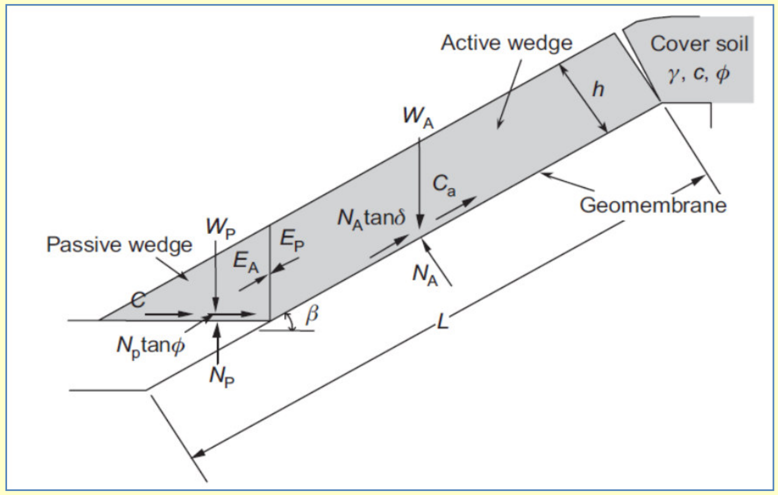


Figure 7. Limit equilibrium forces involved in a finite length slope analysis for a uniformly thick cover soil

Veg. Cover Soil (VC)/Prot. Cover Soil (CS) and Slope Parameters	VC	CS	Utilized		Source
Thickness (h)	0.5	1.5	2.0	ft	Design - see Sheet 18
Dry Unit Weight (γ_D)	100.0	100.0	100.0	pcf	Assumed values based on lab testing
Mois. Cont. (field cond.) (w_F)	20.0	20.0	20.0	%	Assumed values based on lab testing
Avg Field Unit Wt (γ)	-	-	120.0	pcf	
Reference Stress	-	-	**232.8**	psf	
Min. Friction Angle (ϕ)	25.0	25.0	25.0	deg	Assumed (conservative for clay)
Min. Cohesion (c)	100.0	100.0	100.0	psf	Assumed (conservative for clay)
Slope Angle Beneath the Geom. (β)	-	-	14.0	deg	Design - see Sheet 10
Ht. of Slp. Meas. Along Geom. (H_L)	-	-	30.00	ft	Design - see Sheet 10
Lng. of Slp. Meas. Along Geom. (L)	-	-	123.75	ft	
Inter. Frict. Angle for DL & Geom. (δ)	-	-	12.4	deg	**Min. shear strength parameters**
Adhesion for DL & Geom. (c_a)	-	-	0.0	psf	**assuming residual strength**
Required Factor of Safety (FS_R)	-	-	1.10		**Min. req. FS**

Active Wedge Calculations

Determine the total weight of the active wedge (W_A),the effective force normal to the failure plan of the active wedge (N_A), the adhesive force between the cover soil of the active wedge and the geomembrane (C_a), and the interwedge force acting on the active wedge from the passive wedge (E_A) using the following eqs:

$$W_A = \gamma h^2\left(\frac{L}{h} - \frac{1}{\sin\beta} - \frac{\tan\beta}{2}\right) = \textbf{27,659 lbs}$$

$$N_A = W_A \cos\beta = \textbf{26,834 lbs}$$

$$C_a = c_a\left(L - \frac{h}{\sin\beta}\right) = \textbf{- lbs}$$

$$E_A = \frac{(FS)(W_A - N_A\cos\beta) - (N_A\tan\delta + C_a)\sin\beta}{\sin\beta(FS)} = \textbf{1,342 lbs}$$

Passive Wedge Calculations

Determine the total weight of the passive wedge (W_P),the effective force normal to the failure plan of the passive wedge (N_P), the cohesive force along the failure plane (C), and the interwedge force acting on the passive wedge from the active wedge (E_P) using the following eqs:

$$W_P = \frac{\gamma h^2}{\sin 2\beta}$$ = **1,020 lbs**

$$C = \frac{ch}{\sin \beta}$$ = **825 lbs**

$$E_P = \frac{C + W_P \tan \phi}{\cos \beta (FS) - \sin \beta \tan \phi}$$ = **1,363 lbs**

$$N_P = W_P + E_P \sin \beta$$ = **1,351 lbs**

Static Factor of Safety for Residual Strength (Solved for Iteratively)

Determine the calculated Factor of Safety (FS_A) using a quadratic equation relationship where the constants are defined as follows:

$$a = (W_A - N_A \cos \beta)\cos \beta$$ = **1,577 lbs/ft**

$$b = -\begin{bmatrix} (W_A - N_A \cos\beta)\sin\beta\tan\phi \\ +(N_A \tan\delta + C_a)\sin\beta\cos\beta \\ +\sin\beta(C + W_P \tan\phi) \end{bmatrix}$$ = **(1,887) lbs/ft**

$$c = (N_A \tan \delta + C_a)\sin^2 \beta \tan \phi$$ = **162 lbs/ft**

Results Have Converged

FS_R =	**1.10**	
FS_A =	**1.10**	
Min. Resid. δ	**12.4**	**deg**
Min. Resid. c_a	**0.0**	**psf**

ATTACHMENT B

MINIMUM REQUIRED SHEAR STRENGTH FOR FINAL COVER VENEER STABILITY

The following chart depicts the minimum required shear strength values along the interfaces of the engineered components of the final cover system under relatively light uniform loads up to the reference load as illustrated in **Figure B.1**. The envelope depicts the peak shear strength used in the veneer stability analyses represented in the form of the minimum friction angle (ϕ) required to maintain the required factors of safety in the veneer stability analysis. All engineered components of the final cover system located adjacent to an interface along the slope should have a combination of shear strength parameters (i.e. cohesion/adhesion and friction angle, ϕ) where the mininmum strength for a given normal stress exceeds that of the peak shear strength envelope depicted in **Figure B.1**. Because the normal stress in the veneer analysis is due to a uniform load from the overlying material, a reference stress representing the commensurate normal stress applied from those materials is included on the chart for use in determining appropriate confining pressure for laboratory testing.

Using this reference stress, the acceptable combination of cohesion/adhesion and friction angle are plotted on **Figure B.2**. Any combination plotting above and to the right of the peak value curve represents acceptable values for this application as long as the field loads are equal to or less than the reference stress.

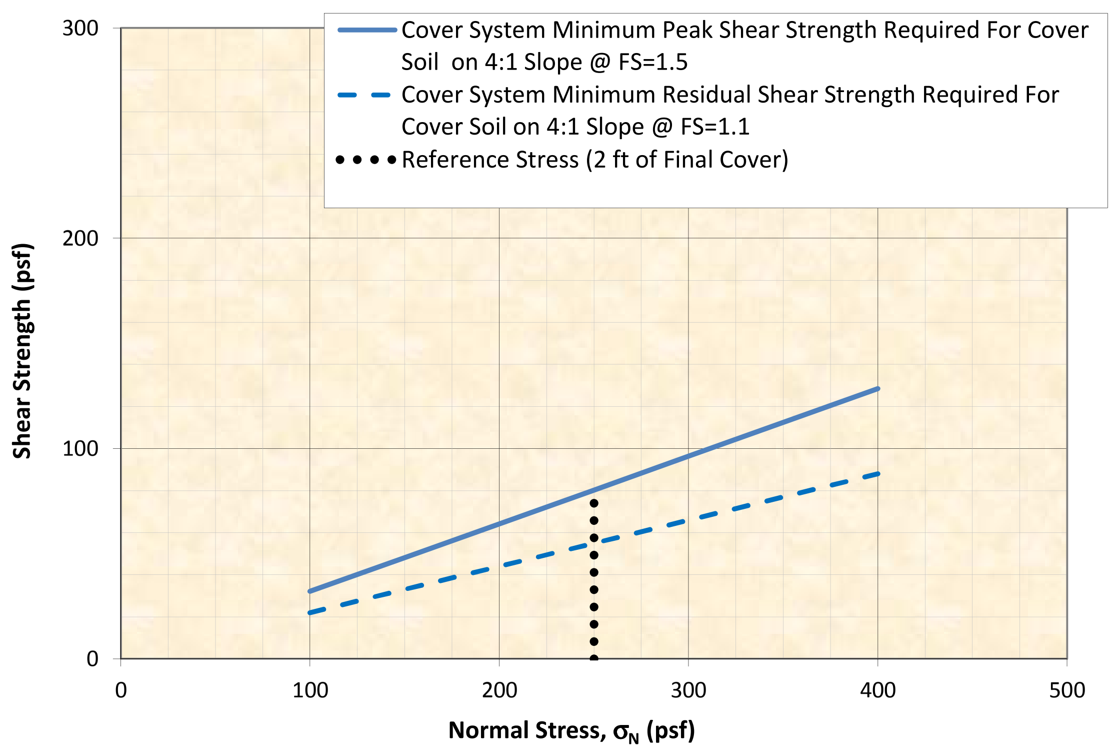


FIGURE B.1

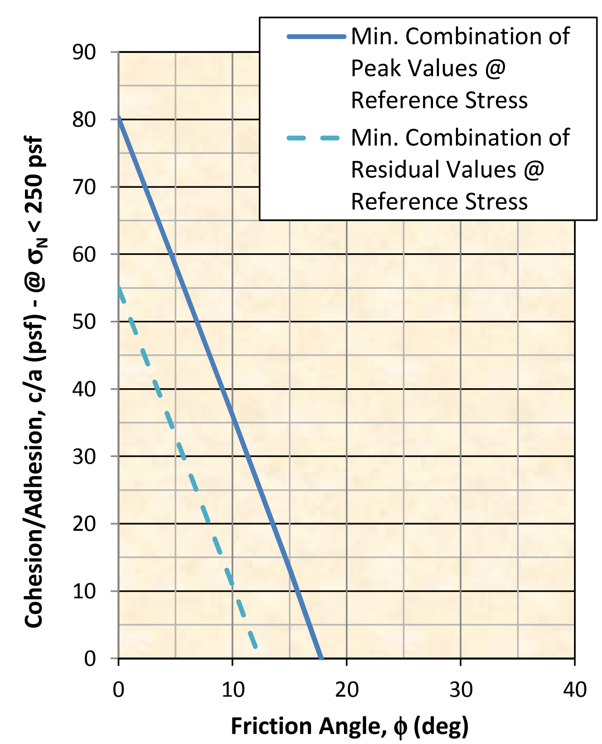


FIGURE B.2

SEISMIC LIQUEFACTION ANALYSIS

BIG SANDY POWER PLANT ASH POND CLOSURE PROJECT

GEOTECHNICAL CALCULATIONS

I. Purpose: **This report presents dynamic liquefaction potential analyses for the fly ash pond at AEP's Big Sandy Plant in Louisa, Kentucky.**

II. Basis of Design

- A subsurface exploration has been performed and included advancement of 8 soil borings (PB-1 through PB-8) within the fly ash pond. Analysis herein is based on the profile of boring PB-7 (see boring log in **Attachment A**), which represents the deepest area of the fly ash pond over which construction of the future landfill may take place.
- The design earthquake event has a 2% probability of recurrence in 50 years.
- Cyclic triaxial and resonant column testing of reconstituted and undisturbed samples of fly ash from Big Sandy were performed in support of this analysis. Supplementary, fly ash cyclic strength data from the Big Sandy Plant, provided by AEP were also used.
- Geophysical testing, including downhole shear wave velocity profiles and Multichannel Analysis of Surface Waves (MASW), was implemented at the Big Sandy site. Downhole testing was implemented over the full depth of boreholes PB-3 through PB-8, and MASW was implemented at transverse sections across the entire width of the fly ash pond at the locations of borings PB-6, 7, and 8. Shear wave velocities utilized in this analysis are based on the results of the geophysical testing. See **Attachment A**.
- Effects of the landfilled fly ash on the future in-situ confining stresses within the ponded fly ash were ignored in this analysis.

III. Methodology

The analysis consisted of the following steps:

1. Development of representative earthquake time histories at bedrock.
2. Implementation of a 1-D equivalent-linear response analysis, to obtain site response (cyclic stress ratios, CSRs) due to vertical propagation of shear waves from the bedrock ground motion. This analysis was performed using computer program SHAKE2000.
3. Determination of Cyclic Resistance Ratios (CRRs) for the ponded ash, based on data available at the time of this analysis, and,
4. Calculation of the factors of safety against liquefaction as a function of depth within the profile of the fly ash pond.

Detailed description of each of these steps is provided in Sections IV through VII below. Conclusions of the analysis are presented in Section VIII.

IV. Development of Representative Time Histories

Bedrock at the Big Sandy site consists of interbedded shale and sandstone, with RQD in the shales generally less than 50% and RQD in the sandstones generally between 50 and 80%. Herein, the site bedrock is considered to be NEHRP Class B material. The response spectrum for a return period of 2,500 years from the USGS National Seismic Hazard Maps was adopted and is given in Table 1 below. This spectrum is for the NEHRP B/C boundary site condition (V_{s30} = 760 m/sec), which corresponds to the anticipated bedrock at the Big Sandy site. V_{s30} is the average shear wave velocity in the upper 30 meters of the rock section at the site. The response spectral values at periods longer than 2 second were obtained assuming a 1/T decay of the acceleration response spectrum.

Table 1. Probabilistic Response Spectrum, 2,500 year return period, B/C Boundary Site

T (S)	SA (G)
0.01	0.0761
0.1	0.1777
0.2	0.1589
0.3	0.1315
0.5	0.0944
1	0.0629
2	0.0372
3	0.0245
4	0.0186
5	0.0149
6	0.0124
7	0.0106
8	0.0093
9	0.0083
10	0.0074

The site is estimated to have a period in the range of 0.8 – 1.2 second, based on the soil profile of boring PB-7 (see supporting calculations in **Attachment B**). Therefore, the deaggregation of the seismic hazard at a period of 1 second for a return period of 2,500 years was obtained from USGS web-based analysis and was utilized, see **Figure 1**. The hazard is clearly dominated by large earthquakes on the New Madrid Seismic Zone, at a closest distance of about 575 km to the site.

There are no recordings of large earthquakes in the central United States. Therefore, ground motion time histories of large earthquakes that have been simulated in previous work using a physics-based strong ground motion simulation procedure were selected as a basis for the analysis. Three separate pairs (each pair includes two orthogonal directions) of time histories were developed in this fashion. These time histories were spectrally matched to the target spectrum listed in Table 1. The process of spectral matching the three pairs of the time histories is shown in Figures 2 through 7. The top panel in each of these figures shows the target response spectrum and the response spectrum of the time history before and after spectral matching. The bottom panel shows the acceleration, velocity and displacement time histories before and after spectral matching, with the unmatched waveform above and the spectrally matched waveform below.

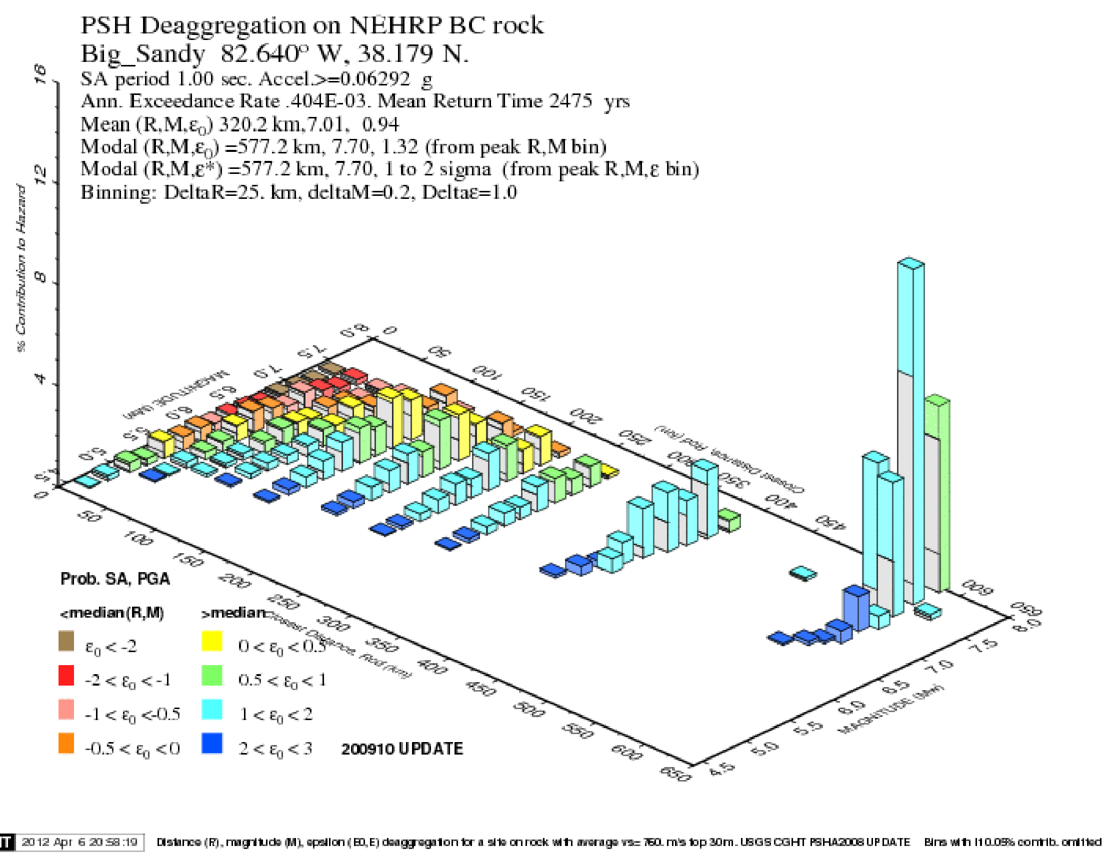
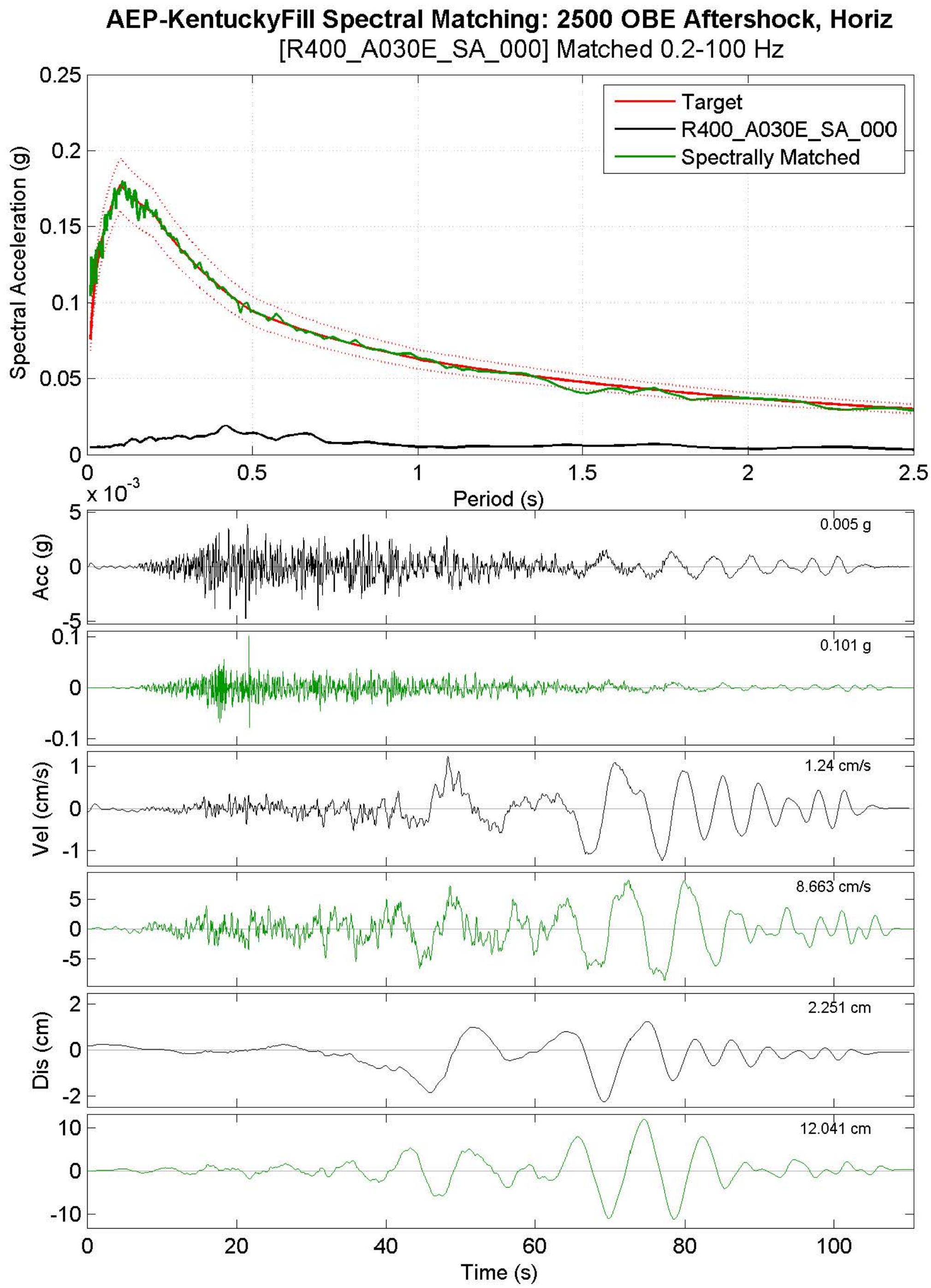


Figure 1. Deaggregation of the 2,500 year seismic hazard at the Big Sandy site at a period of 1 second. Source: USGS.



\\Nas\jeff\Time_Histories_And_Spectral_Matching\AEP-BigSandyFill\RSP Match on 2012/4/24 15:33

Figure 2. North component of time history No.1 spectrally matched to the 2,500 year ARP target:

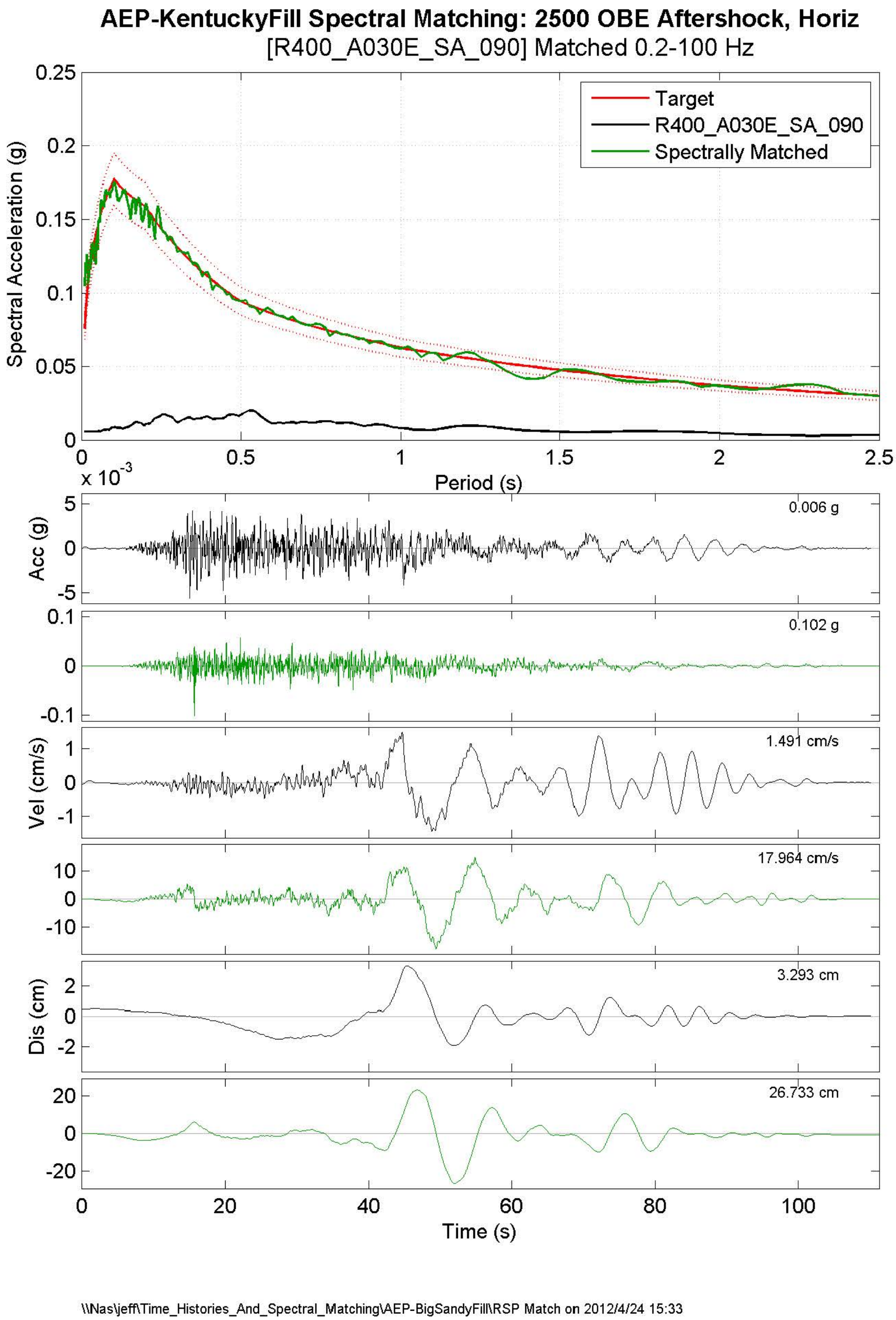


Figure 3. East component of time history No.1 spectrally matched to the 2,500 year ARP target

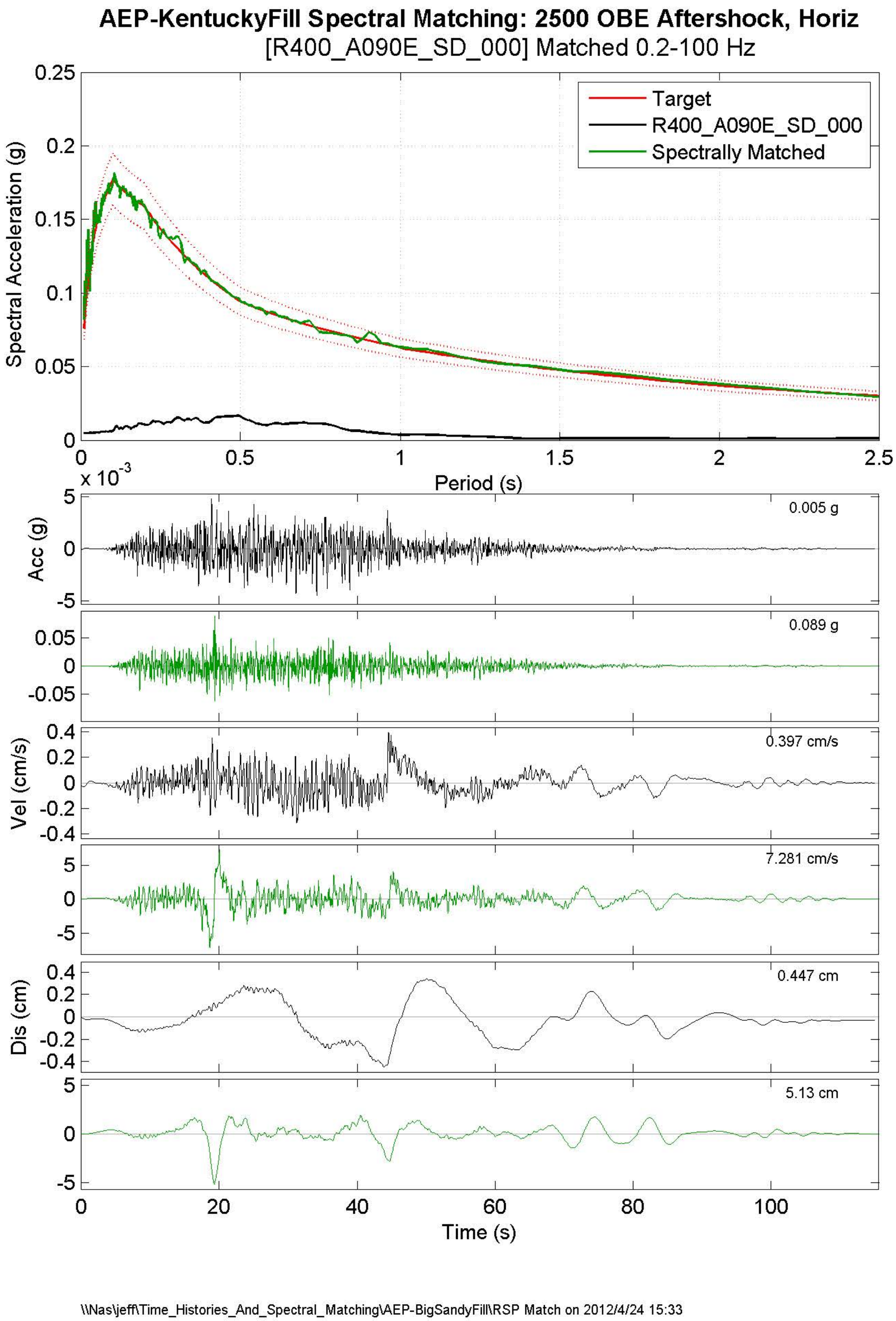


Figure 4. North component of time history No.2 spectrally matched to the 2,500 year ARP target

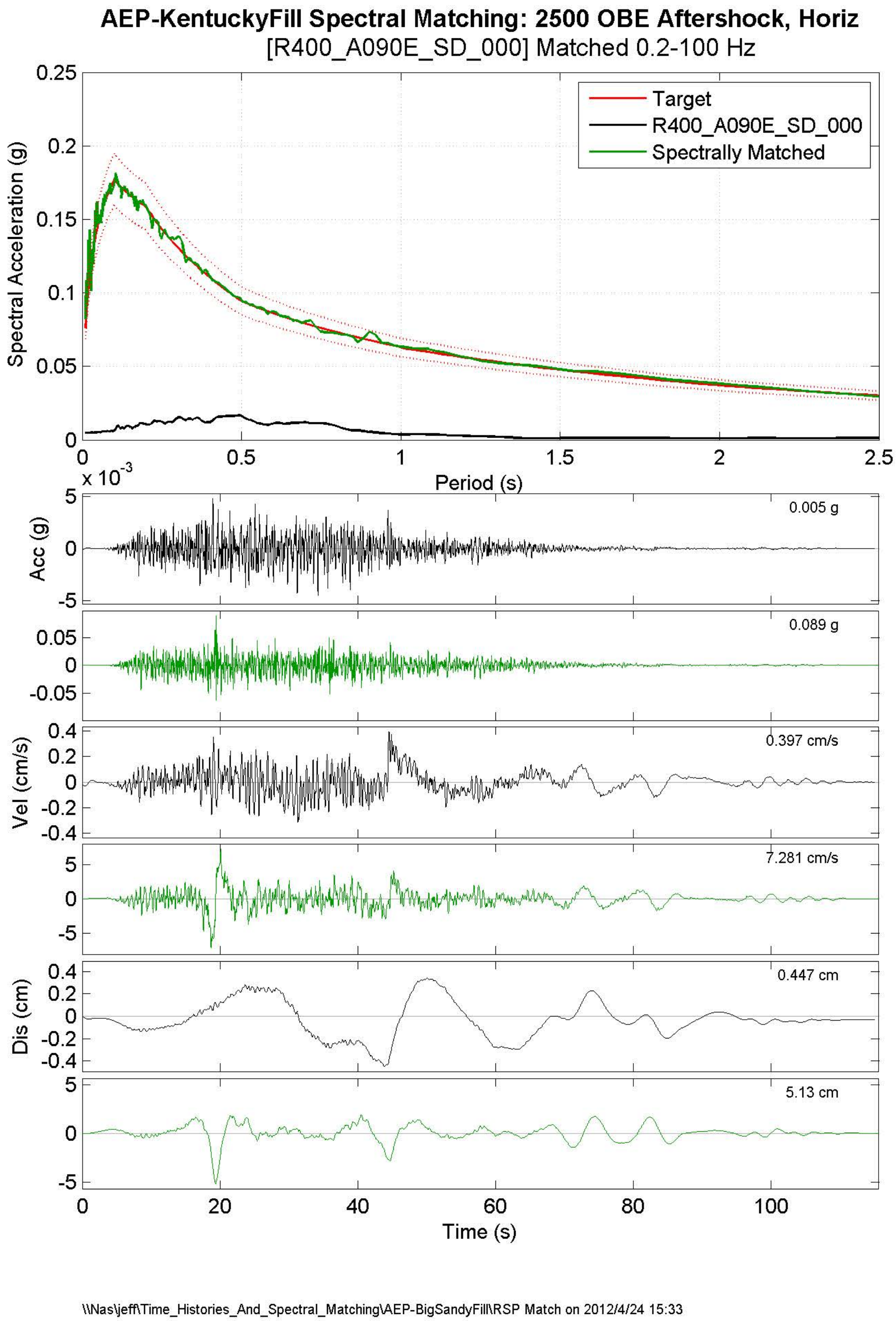


Figure 5. East component of time history No.2 spectrally matched to the 2,500 year ARP target.

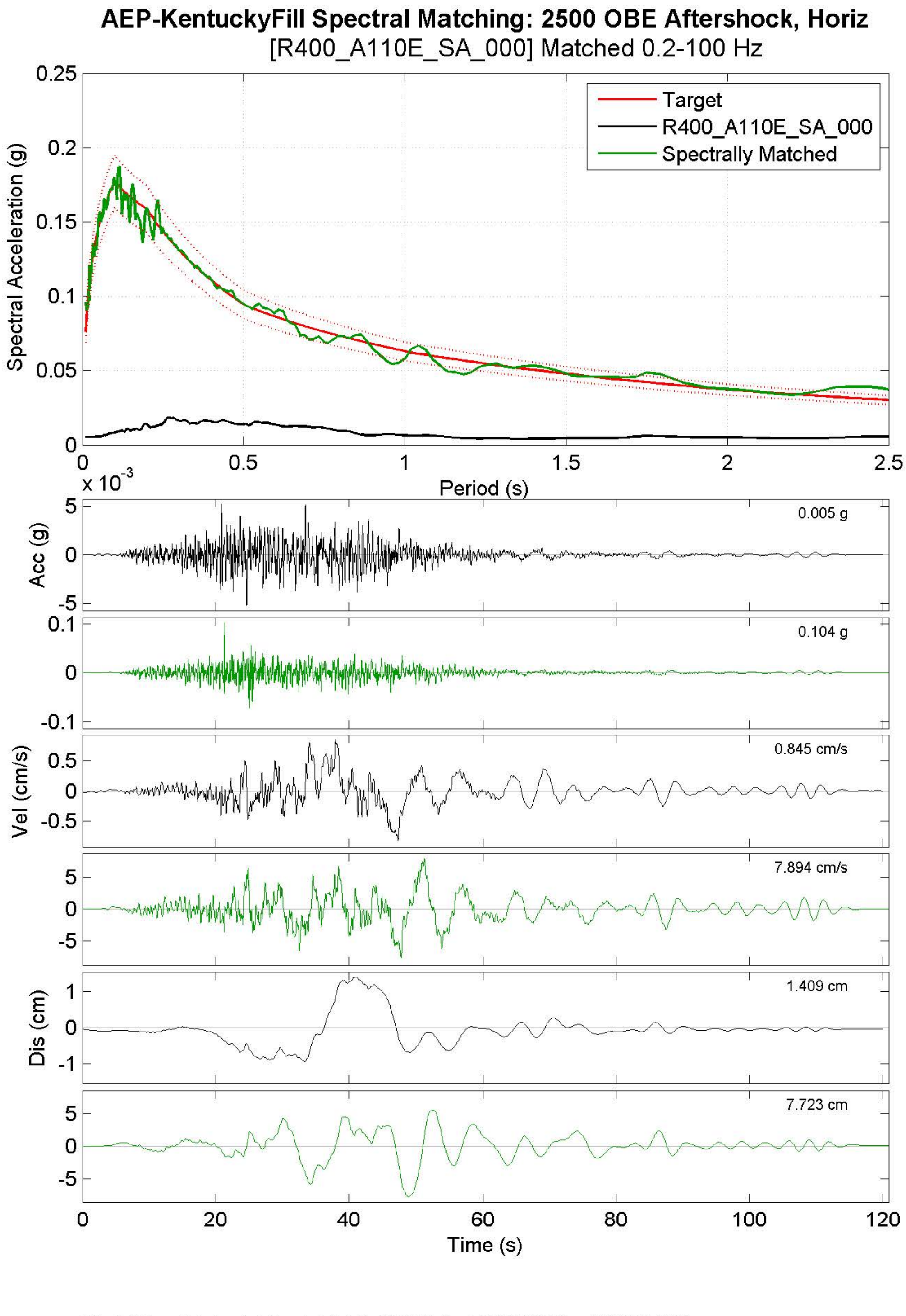


Figure 6. North component of time history No.3 spectrally matched to the 2,500 year ARP target.

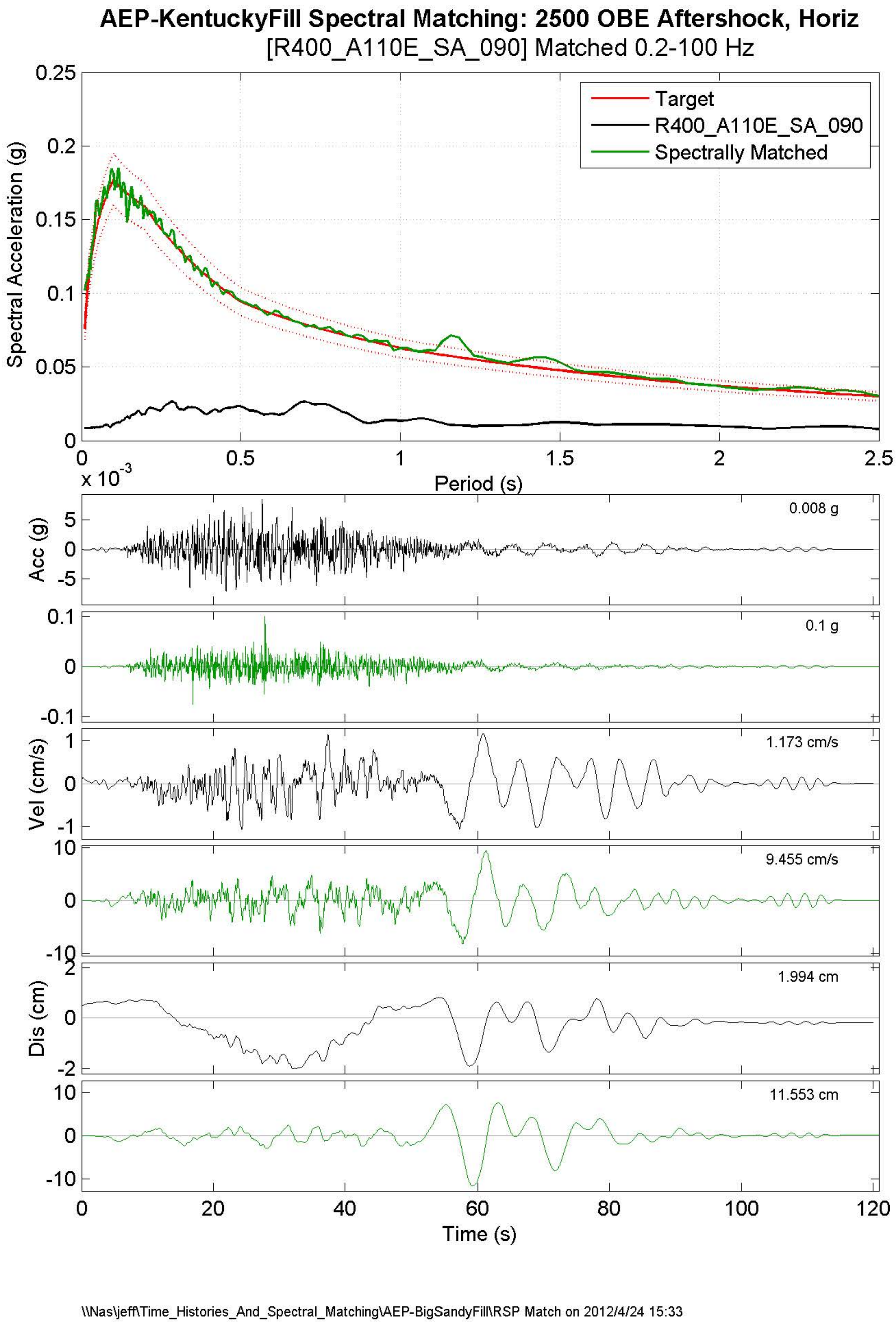


Figure 7. East component of time history No.3 spectrally matched to the 2,500 year ARP target.

V. Site Response Analysis

As stated above, the profile of boring PB-7 was utilized in the site response analysis. This profile denotes the most representative area of the fly ash pond which may be utilized for the proposed closure.

The profile of PB-7 included (from highest to lowest elevation): Approximately 95 ft of very loose saturated fly ash (SPT results generally 0 and up to 4 blows per foot); 15 ft of loose to medium dense alluvium consisting of interbedded sand (SP-SM) and clayey sand (SC); 8 ft of very stiff or very dense alluvium or residuum consisting of interbedded lean clay (CL) and sand (SP-SM), and; Shale bedrock.

Site response analysis was performed using computer program SHAKE2000, which performs equivalent linear analyses of shear wave propagation through layered soils. The profile and material parameters for the soil column input into SHAKE2000 and other details of the model construction are summarized in Table 2 and the bullet points below:

Table 2. Soil Column For SHAKE2000 Analysis

LAYER	THICKNESS (FT)	MODULUS REDUCTION CURVE	DAMPING CURVE	UNIT WEIGHT (PCF)	SHEAR WAVE VELOCITY (FPS)
Fly Ash	100	Big Sandy Plant Ash Pond Resonant Column Testing - Coarse Fraction		100	500
Clayey GRAVEL	5	Gravel – Seed et al (1986)	Gravel – Seed et al (1988)	130	1100
Silty SAND	10	Sand – Seed & Idriss – Avg. (1970)		130	1100
Lean CLAY	5	Vucetic & Dobri		130	1100
Silty SAND	5	Seed & Idriss – Avg. (1970)		130	1100
Bedrock	Infinite	Schnabel (1973) – elastic half space		145	2500

- Shear wave velocities of the various layers were selected based on the results of the geophysical downhole testing at boring PB-7 (see **Attachment A**).
- Total unit weight of fly ash is based on the results of laboratory testing on ash samples from piston tubes obtained from boring PB-7.

- Modulus Reduction and Damping curves were selected based on the results of the resonant column testing performed as part of the subsurface exploration program performed at the site. The results are included in **Attachment C**. Please note that the resonant column testing did not incorporate results beyond about 5%. Because of the displacements calculated as part of the SHAKE2000 model analysis, it was necessary to extrapolate data outside the range tested. This is illustrated in the curves included in **Attachment C**.
- Each major soil unit given in the soil column was subdivided into 5-ft thick layers in the model.
- The reference shear stress (on which SHAKE2000 CSR results are based) was set to $0.65*\tau_{max}$ in the model.

Input Motion:
As described in Section IV above, three separate pairs of bedrock (NEHRP Class B/C) time histories representing the design earthquake were developed. Each pair consists of an acceleration time history in each of the two orthogonal horizontal directions, for a total of 6 time histories. All six time histories were analyzed separately in SHAKE2000. The B/C boundary at which the input ground motions were applied was taken at the top of the bedrock layer in the soil column given in Table 2.

Results of Analysis:

The parameter of interest from the SHAKE2000 analysis for the liquefaction evaluation study is the Cyclic Stress Ratio (CSR). The CSRs as a function of depth within the soil column (where depth 0 is the top of the fly ash deposit) resulting from the analyses are given below in **Figures 8** through **13**. The following observations are made from the results:

- The maximum CSR calculated by SHAKE2000 is approximately 0.15, with the range of maximum CSR among the six analyses being 0.10-0.15. The range is relatively low, indicating that variations in the representative time histories selected for analysis should not substantially affect the results.
- Maximum CSR consistently occurs in the uppermost portion of the soil column, between 0 and 7 ft.

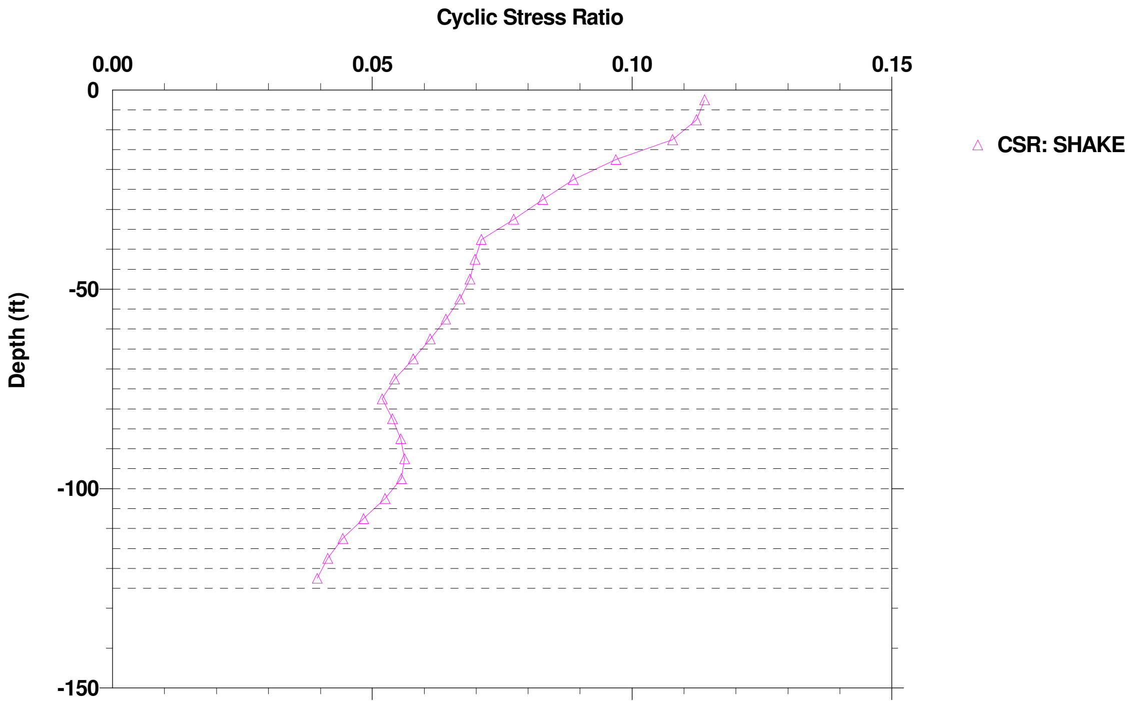


Figure 8: Input Motion 1 – Direction1

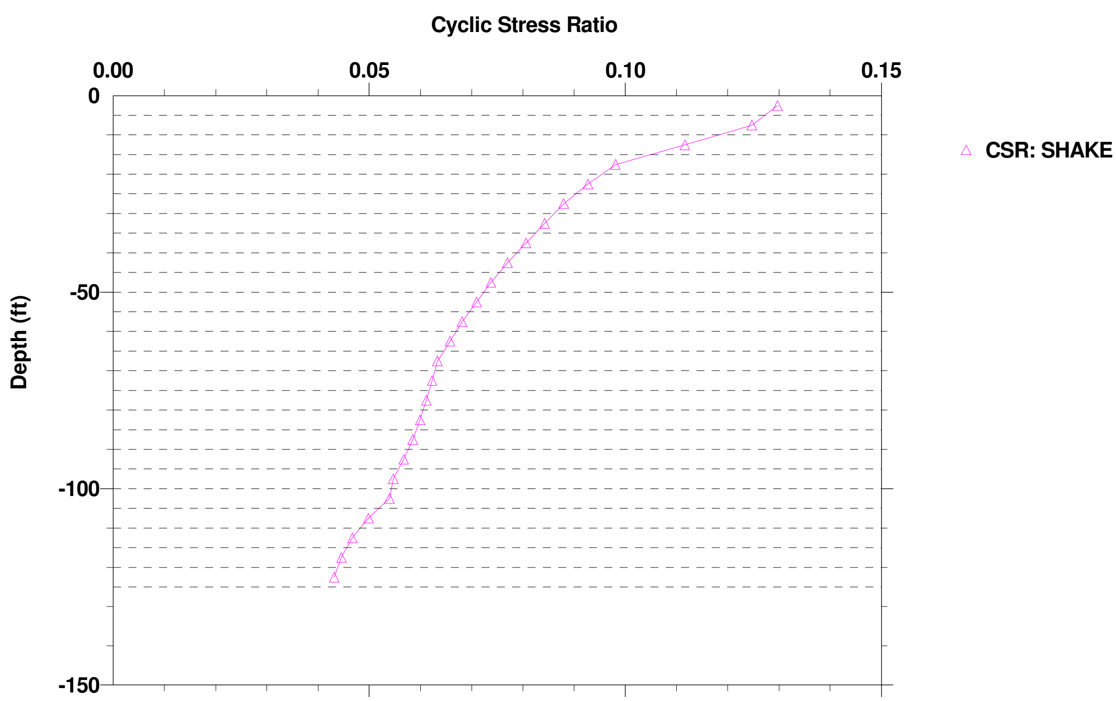


Figure 9: Input Motion 1 – Direction 2

URS Corporation

Job	**AEP Big Sandy Pond Closure**	Project No.	**13815141**	Sheet	**13** of **19**
Description	**Liquefaction Potential Analysis**	Computed by	**NS/JLM**	Date	**5/29/12 (Rev 1/13/12)**
		Checked by	**VKG**	Date	**5/31/12 (Rev1/18/12)**

Figure 10: Input Motion 2 – Direction 1

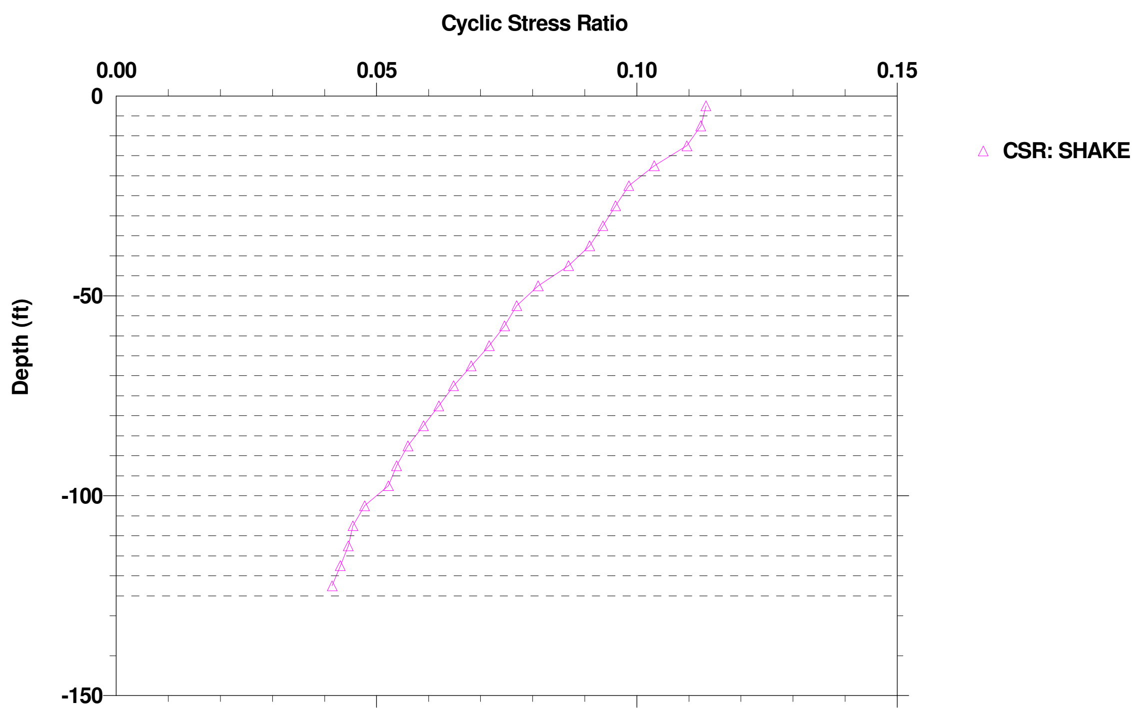


Figure 11: Input Motion 2 – Direction 2

URS Corporation

Job	**AEP Big Sandy Pond Closure**	Project No.	**13815141**	Sheet	**14** of **19**
Description	**Liquefaction Potential Analysis**	Computed by	**NS/JLM**	Date	**5/29/12 (Rev 1/13/12)**
		Checked by	**VKG**	Date	**5/31/12 (Rev1/18/12)**

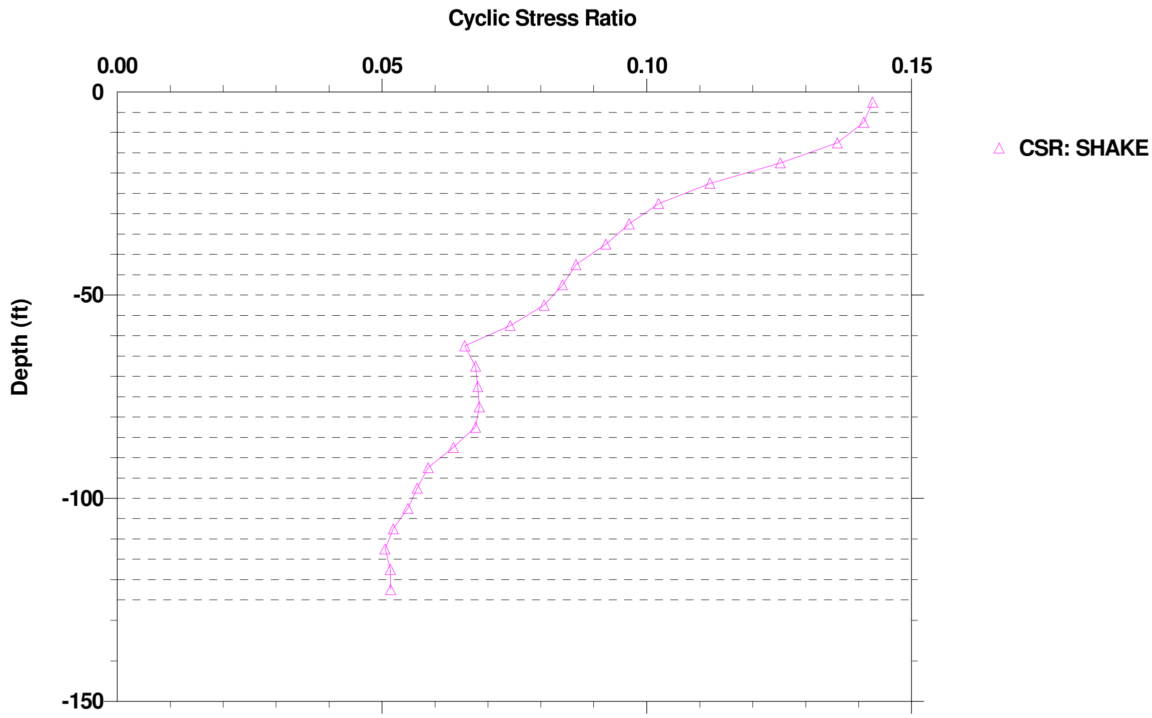


Figure 12: Input Motion 3 – Direction 1

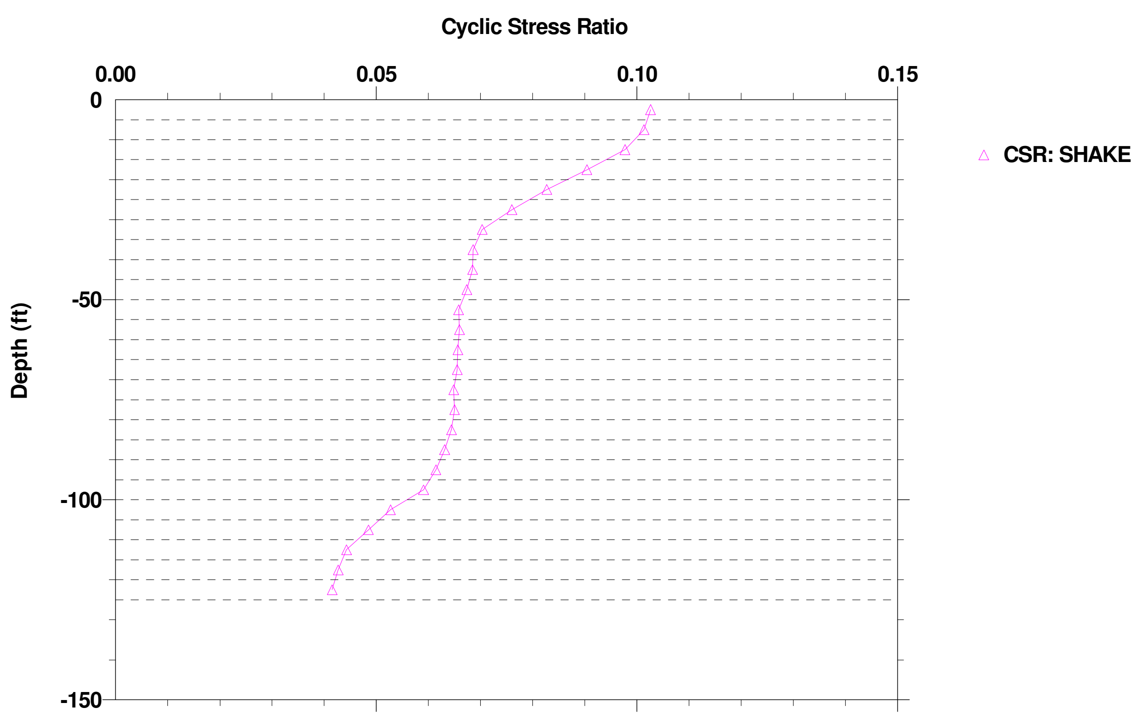


Figure 13: Input Motion 3 – Direction 2

VI. Determination of Cyclic Resistance Ratios

URS determined cyclic resistance ratios on the basis of cyclic triaxial tests performed on fly ash obtained from the site during the subsurface exploration. A total of five (5) specimens were tested. Fly ash materials were obtained from the site using piston tube sampling techniques. Total unit weights of the samples were measured both upon extraction of the tube samples and in the laboratory after extrusion. Extruded fly ash material was then reconstituted in the laboratory using a water pluviation procedure developed in conjunction with AEP. All pluviated samples were subjected to a consistent confining stress of 20 psi, and test stress ratios varied from approximately 0.1 to 0.3. Unit weights of the samples prior to shearing were measured at between 102 and 104 pcf, which is in reasonably close agreement with unit weights measured in the undisturbed piston samples. Four of the five specimens were prepared and tested in this manner. One tube sample was sheared directly upon extrusion from a piston tube (i.e., this was an undisturbed specimen, not reconstituted).

In addition to the URS testing, AEP provided cyclic triaxial data for seven (7) additional reconstituted specimens of Big Sandy Fly Ash that they have tested as part of a separate study being performed in conjunction with the Ohio State University Department of Civil and Environmental Engineering and Geodetic Science. These specimens were prepared in a similar fashion to those tested by URS and over a similar range of stress ratios.

As described above, a total of twelve (12) data points were thus available to establish a distribution of cyclic resistance ratio as a function of the number of cycles to liquefaction, with 11 of the 12 points representing pluviated samples. This curve is provided in **Figure 14** below. The shear stress ratios given on the figure correspond to the values required to induce liquefaction of the test samples, and thus represent the cyclic resistance ratio (CRR) of the fly ash. The combined data points (both URS and AEP results) are in relatively close agreement with each other, with the exception of one AEP data point (tested at CSR = 0.2 and which had only three cycles to liquefaction). This point is considered an outlier and has been excluded from this evaluation. Furthermore, the single specimen that was tested directly from a piston tube exhibited substantially higher CRR than the pluviated specimens. Although this is an encouraging result with respect to liquefaction potential (suggesting that the in-situ ash may have higher resistance), this data point is also conservatively excluded from this evaluation. A log-linear regression line was fit to the ten data points being considered herein, and is depicted on Figure 14. This envelope was then utilized in the liquefaction potential calculations, presented in the sections which follow.

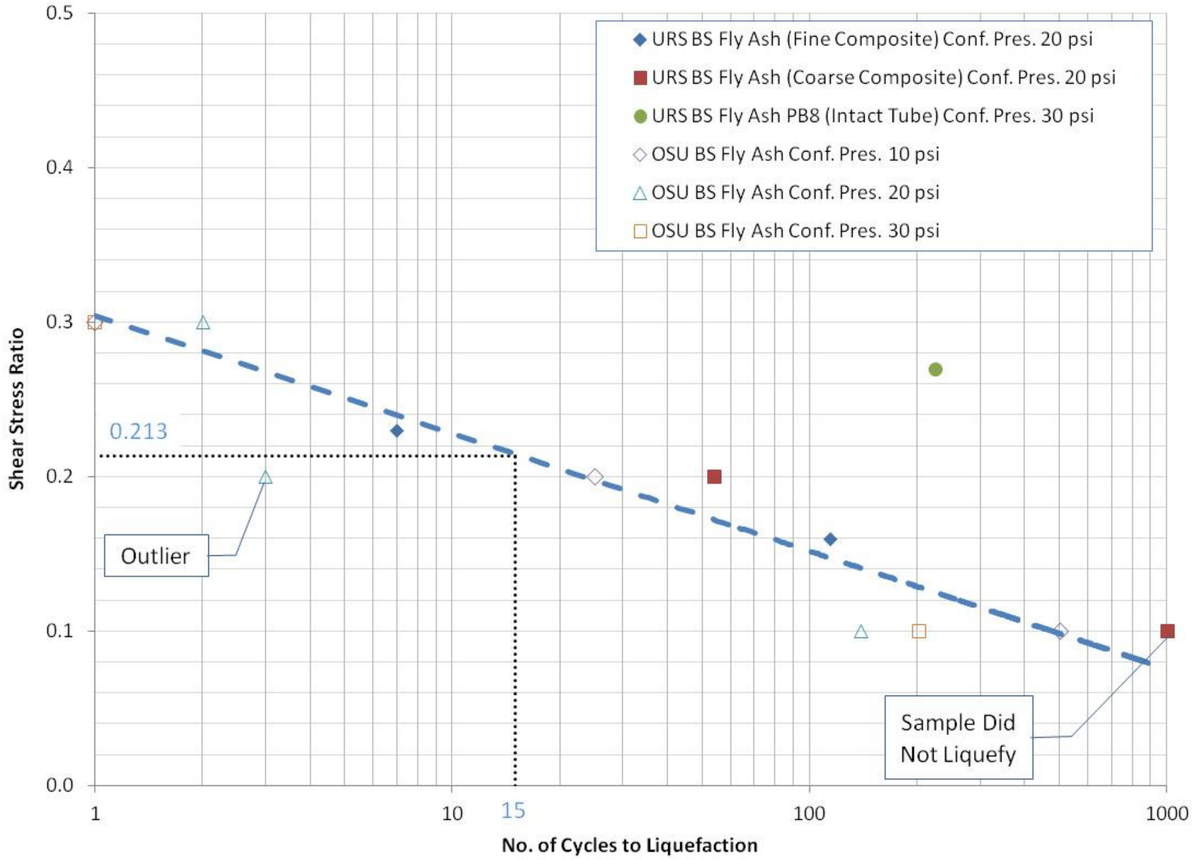


Figure 14: Cyclic Resistance Ratio Envelope from Laboratory Testing Results

From the figure, the CRR depends on the number of cycles of shear stress application. In order to estimate the CRR for liquefaction analysis, the number of cycles anticipated during the design earthquake event must be estimated. In order to do this, the irregular time histories of earthquake-induced shear stresses obtained from SHAKE2000 must be converted to an equivalent number of uniform stress cycles at the reference stress ($0.65\tau_{max}$) on which the SHAKE2000 CSRs are based. For this analysis, a correlation between the number of equivalent cycles at the reference stress and earthquake magnitude, as given in Seed & Idriss (1982) and Idriss (1999) was utilized.

From **Figure 1** (Section IV), the primary contribution to the design ground motions developed for the analyses herein correspond to events with moment magnitude of 7.5. Figure 15 below presents the aforementioned correlation of Seed & Idriss (1982) and Idriss (1999). For events with M = 7.5, the estimated number of equivalent uniform cycles is approximately 15.

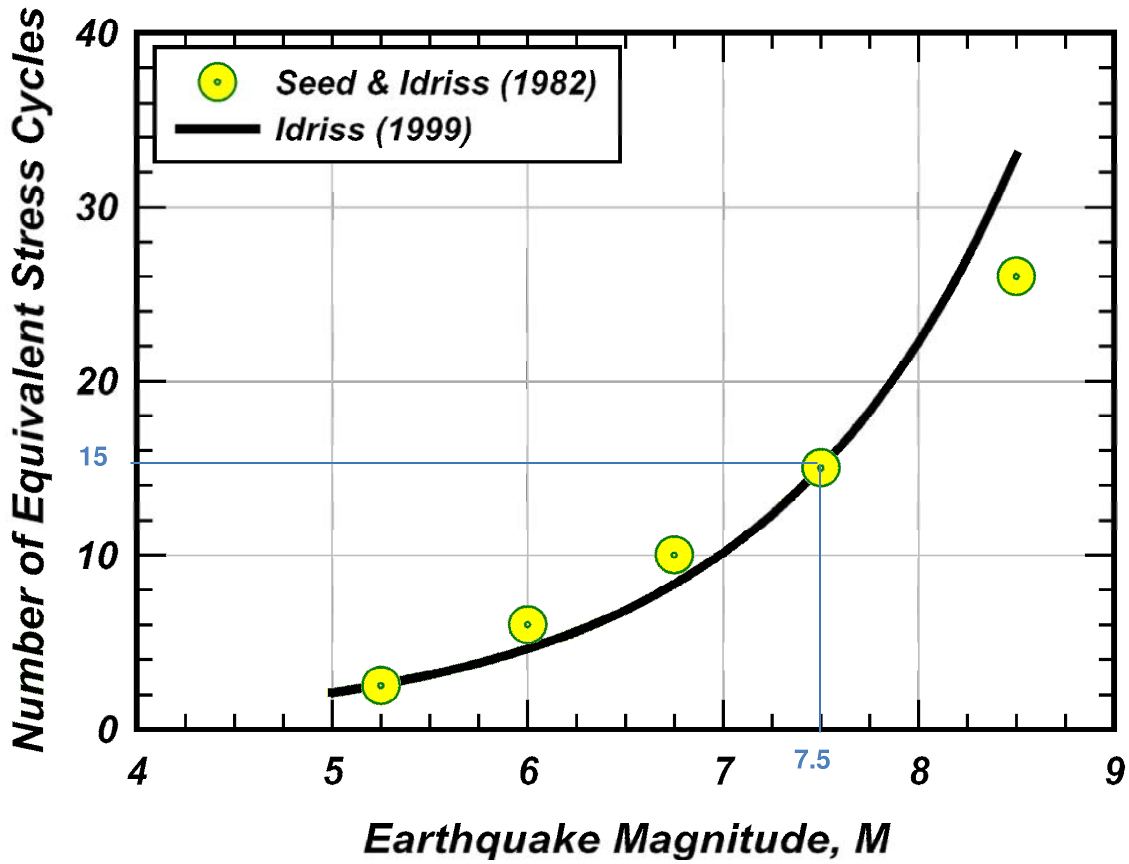


Figure 15. Mean number of equivalent uniform cycles at reference stress of 65% of the peak stress versus earthquake magnitude.

Herein, this value (N = 15) is assumed for all depths within the fly ash deposit.

Returning to the testing results shown on **Figure 14** and using the best fit curve of the site specific data with N=15, the CRR is obtained as approximately **0.213**.

This CRR value corresponds to cyclic triaxial tests with unidirectional shaking. However, to account for multidirectional shaking that may occur in the field, the CRR value is reduced by 10%.

Therefore,

$$(CRR)_{field} = 0.9(CRR)_{lab}$$

$$\mathbf{(CRR)_{field} = 0.192}$$

This value is assumed for all depths within the fly ash deposit herein.

VII. Calculation of Factors of Safety Against Dynamic Liquefaction

The factor of safety against liquefaction is defined as follows:

$$FS_{liq} = \frac{CRR}{CSR}$$

As CSR varies both with ground motion input and with depth, factors of safety as a function of depth in the profile were calculated based on this equation, using the CRR value of 0.192 determined in Section VI above, and the CSRs from SHAKE2000 portrayed in Figures 8 through 13. Resulting factors of safety are portrayed in **Figure 16** below. Supporting calculations are given in **Attachment B**.

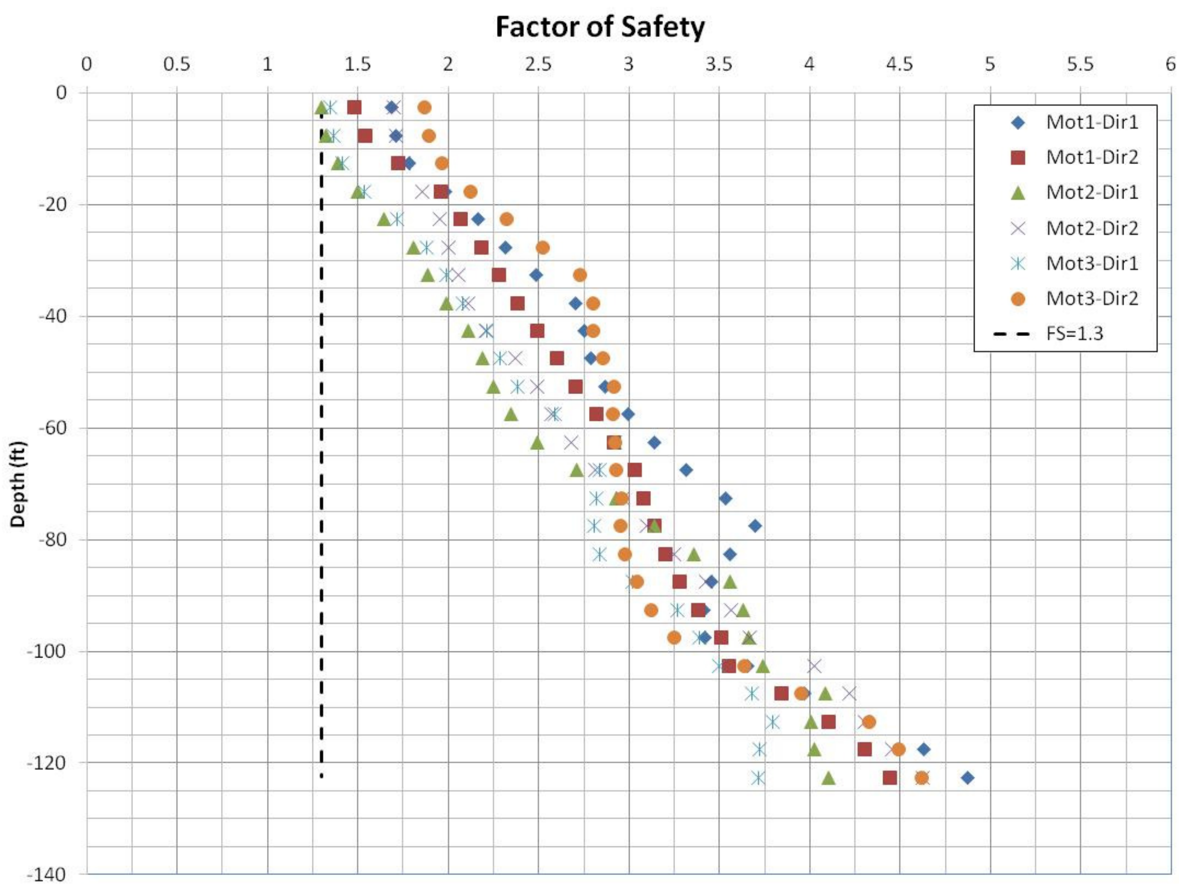


Figure 16. Calculated factors of safety as a function of depth

From **Figure 16**, the minimum calculated factors of safety for the input motions considered are in the range of 1.3 to 1.8, and the minimum values correspond to the near surface (≤7.5 ft) of the profile.

VIII. Conclusions

- Under the seismic action of a 7.5 magnitude earthquake with a return period of ≈ 2,500 years, the present analysis indicates minimum factors of safety against dynamic liquefaction in the range of 1.3 to 1.8 for the input motions within the near surface (≤7.5 ft) of the fly ash. Factors of safety increase substantially below a depth of 7.5 ft below the surface to minimum factors of safety in the range 1.4 to 2.0 across all of the different input motions considered. During the course of the proposed construction, it is anticipated that the near surface fly ash will be densified by construction of the final cover – the material will be stabilized to act as a "bridge" layer to allow practical access for construction equipment. This process is also anticipated to improve the response of these near-surface materials to long term seismic activity.
- According to Ohio Environmental Protection Agency guidance (Ohio EPA, 2004), factors of safety against dynamic liquefaction of greater than or equal to 1.00 are appropriate if the design assumptions are conservative; site-specific, higher quality data are used, and the calculation methods chosen are valid and appropriate for the facility. It is anticipated that the methodologies and results presented herein meet the requirements of this guidance. Furthermore, it is generally accepted that no liquefaction should be anticipated for materials that exhibit factors of safety ≥ 1.4 under seismic activity.

IX. References

Seed, H. B., and Idriss, I. M., 1982. *Ground Motions and Soil Liquefaction During Earthquakes,* Earthquake Engineering Research Institute, Oakland, CA, 134 pp.

Idriss, I. M., 1999. "An update to the Seed-Idriss simplified procedure for evaluating liquefaction potential", in *Proceedings, TRB Workshop on New Approaches to Liquefaction,* Publication No. FHWA-RD-99-165, Federal Highway Administration.

Ohio Environmental Protection Agency, Geotechnical Resource Group, 2004. *Geotechnical and Stability Analyses For Ohio Waste Containment Facilities*.

ATTACHMENT A

SUPPORTING SUBSURFACE INFORMATION

Project: AEP Big Sandy Landfill Investigation
Project Location: Louisa, KY
Project Number: 13815141.10000

Log of Boring PB-7

Date(s) Drilled	4/17/12-4/19/12	Logged By	T. George	Checked By	V. Gautam
Drilling Method	HSA, Mud rotary	Drill Bit Size/Type	4 1/4" ID/8" OD HSA, 4" tricore mud-rotary	Total Depth of Borehole	127.0 feet
Drill Rig Type	CME 55 Tracked ATV	Drilling Contractor	Pennsylvania Drilling	Surface Elevation	695.3 ft above msl
Borehole Backfill	2" SCH 40 PVC riser grouted in place	Sampling Method(s)	Piston/Split-spoon	Hammer Data	140#/30" Drop Auto
Boring Location		Groundwater Level(s)	Encountered 8' ATD		

Elevation, feet	Depth, feet	SAMPLES Type	SAMPLES Number	SAMPLES Sampling Resist. Blows/6" OR CORE% RQD	SAMPLES Recovery, %	SAMPLES Pocket Penetrometer (tsf)	Graphic Log	MATERIAL DESCRIPTION	Water Content %	REMARKS AND OTHER DETAILS
										2.5' stickup
695	0							Bottom ash access road [BOTTOM ASH]		Grout
	2									Augered to 8' without sampling.
	4									2" SCH 40 PVC riser pipe
690	5									
	8		SS-1	2, 1, 1, 1	58			Very loose, wet, light gray with interbedded dark gray fly ash as mostly fine SAND (SP-SM to SM) with interbedded minor sandy SILT (ML) [FLY ASH]		
685	10									
	13		SS-2	1, 0, 0, 0	67			becomes mostly sandy silt (ML)		
680	15									
	18		SS-3	1, 0, 1, 1	58					
	20									

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URS

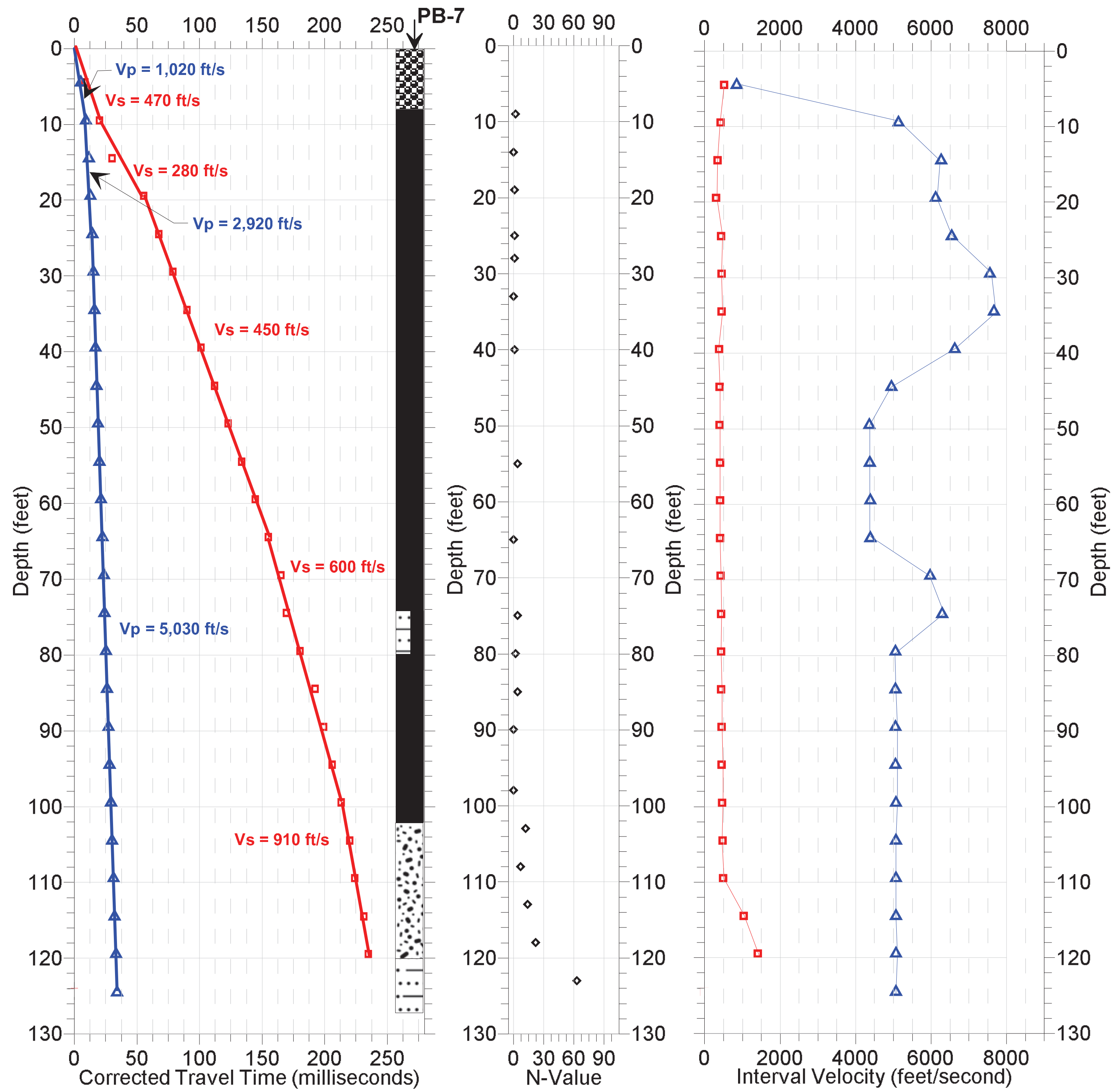
Elevation, feet	Depth, feet	Type	Number	Sampling Resist. Blows/6" OR CORE% RQD	Recovery, %	Pocket Penetrometer (tsf)	Graphic Log	MATERIAL DESCRIPTION	Water Content %		REMARKS AND OTHER DETAILS
675	20										
	21										
	22–24	P	P-1		33						
670	24–26	SS	SS-4	WOH 1 0 1	25						
	27–29	SS	SS-5	1 1 0 1	0						
665	30										
	31										
	32–34	SS	SS-6	WOH 0 0 0	71			becomes with increasing sand			
660	35										
	36										
	37–39	P	P-2		21						
655	39–41	SS	SS-7	WOH 1 0 0							
	42										
	43										

Elevation, feet	Depth, feet	Type	Number	Sampling Resist. Blows/6" OR CORE% RQD	Recovery, %	Pocket Penetrometer (tsf)	Graphic Log	MATERIAL DESCRIPTION	Water Content %		REMARKS AND OTHER DETAILS
	44										
650	45										
	46										
	47		P-3		31			becomes with trace root fibers			*Piston tube bent.*
	48										
	49										
645	50										
	51										
	52							becomes without root fibers			
	53		P-4		75						
	54							becomes mostly sand (SP-SM to SM) with minor sandy silt and occasional seams of bottom ash			
640	55		SS-8	1, 2, 2, 3	67						
	56										
	57										
	58		P-5		56			becomes mostly silt (ML) with interbedded silty sand (SM)			
	59										
635	60										
	61										
	62										
	63		P-6		96						
	64										*Split-spoon @ 64-66' bgs driven 4 ft with 1 blow*
630	65		SS-9	1, 0, 0, 0	0						
	66										

Elevation, feet	Depth, feet	Type	Number	Sampling Resist. Blows/6" OR CORE% RQD	Recovery, %	Pocket Penetrometer (tsf)	Graphic Log	MATERIAL DESCRIPTION	Water Content %		REMARKS AND OTHER DETAILS
	67										
	68										
	69										
	70										
625											
	71										
	72										
	73		P-7		92						
	74										
	75		SS-10	2 1 3 6	92			becomes interbedded SM/SP/ML with light brown lean clay laminae			
620											
	76										
	77										
	78		P-8		75			becomes light brown and gray SILT (ML) with interbedded sand (SP-SM to SM), trace grass			
	79										
	80		SS-11	1 1 1 2	0						
615											
	81										
	82										
	83		P-9		92						
	84							becomes mostly sandy silt (ML)			
	85		SS-12	2 2 2 4	83			becomes mostly fine silty sand (SM)			
610											
	86										
	87										
	88		P-10		75						
	89		SS-13	WOR 0	100			becomes mostly fine silty sand (SM) with minor interbedded sandy silt (ML)			*Sample at 89-91' bgs fell to 96' bgs under weight of rods*
	90										

Elevation, feet	Depth, feet	Sample Type	Sample Number	Sampling Resist. Blows/6" OR CORE% RQD	Recovery, %	Pocket Penetrometer (tsf)	MATERIAL DESCRIPTION	Water Content %	REMARKS AND OTHER DETAILS
605		Split spoon	SS-13	0 0	100				
	91								
	92								
	93								
	94								
600	95								
	96								
	97	Split spoon	SS-14	WOR 0 0 0	0				Split-spoon @ 97-99' bgs fell to 101.5' bgs
	98								
	99								
595	100								
	101								
	102	Split spoon	SS-15	4 6 6 10	25		Medium dense, wet to moist, tan to brown with black staining and oxidation staining, clayey GRAVEL (GC), trace root fibers [ALLUVIUM]		Gravel is sandstone fragments up to 1/2" diameter
	103								
	104								
590	105								
	106						Loose, wet, brown with oxidation staining, medium to fine SAND (SP-SM), trace interbedded lean clay [ALLUVIUM]		Lean clay layers are <1" thick
	107	Split spoon	SS-16	3 4 3 2	71			23.7	%G=0.0 %S=72.5 %F=27.5
	108								
	109								
585	110								
	111						Medium dense, moist, brown with oxidation staining, gray and greenish gray, trace white, clayey SAND (SC) interbedded with silty sand (SM), trace sandstone gravel [ALLUVIUM]		
	112	Split spoon	SS-17	10 9 5	25				
	113								

Elevation, feet	Depth, feet	Type	Number	Sampling Resist. Blows/6" OR CORE% RQD	Recovery, %	Pocket Penetrometer (tsf)	Graphic Log	MATERIAL DESCRIPTION	Water Content %		REMARKS AND OTHER DETAILS
	114		SS-17	4	25						
580	115										
	116							becomes all clayey sand (SC) with sandstone gravel			
	117		SS-18	12 11 11 11	54			Very stiff, moist, gray to dark brown and greenish gray lean CLAY (CL) with sand, trace sandstone gravel [ALLUVIUM]	15.1		%G=11.8 %S=53.3 %F=34.9
	118										
	119										
575	120							Very dense, moist, variably brown with gray mottling, with oxidation staining, medium to fine SAND (SP-SM), with gravel as sandstone fragments [RESIDUUM]			
	121										
	122		SS-19	10 30 33 50/1½"	71				14.1		%G=11.1 %S=67.8 %F=21.1
	123							Gray and dark gray shale, moderately weathered, weak			
	124										Hard drilling 124-127' bgs
570	125										
	126							becomes silty, dark gray, fresh, medium strong			
	127		SS-20	50/½"	100			End of Boring at 127´ bgs			Set PVC casing at 127' bgs. Cement-bentonite grout placed using tremie pipe.
	128										
	129										
565	130										
	131										
	132										
	133										
	134										
560	135										
	136										



Legend:

- △ - Compressional Wave Travel Times
- □ - Shear Wave Travel Times
- Avg Vp - Average Compressional Wave Velocity
- Avg Vs - Average Shear Wave Velocity

Bottom Ash

Alluvium

Flyash

Residual Soils

Legend:

- ▲ - Compressional Wave Interval Velocity
- ■ - Shear Wave Interval Velocity

URS
Geophysical Services
12420 Milestone Center Dr., Suite 150
Germantown, Maryland 20876
(301) 820-3125

Geophysical Investigation Results
Vertical Seismic Profiling Results
Boring PB-7

AEP Big Sandy Plant

Louisa, Kentucky

DESIGNED BY		DRAWN BY		CHECKED BY		JOB NUMBER	FIGURE
TJK	5/8/2012	TJK	5/8/2012	SH	5/10/12	13815141	6

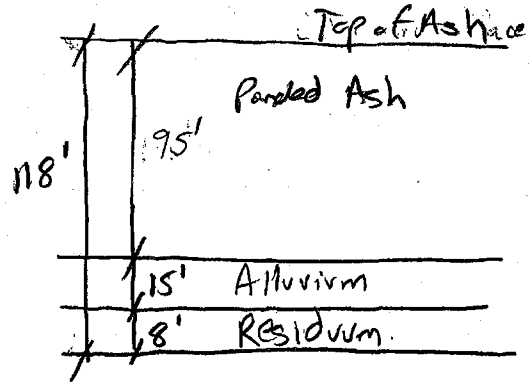
ATTACHMENT B

SUPPORTING CALCULATIONS

URS

Job AEP Big Sandy Pond Closure | Project No. 13815151 | Page ____ of ____
Sheet ____ of ____
Description Estimation of Site Period for Selection of Design Earthquake | Computed by VKG | Date 5/9/12
Checked by ____ | Date ____
Reference

Boring BB-7 Profile is approximately:



note that the bottom ash access road is removed from this profile

from the results of geophysical downhole testing at BB-7, the range of measured shear wave velocities (excluding outliers) is approximately:

- Fly Ash: 340 - 475 fps.
- Alluvium: 475 - 489 fps.
- Residuum: 1040 - 1410 fps.

The period of the soil column is approximated using the equation:

$$T = \frac{4H}{V_s}$$ where $H = 118'$

V_s = Taken as a weighted average (based on layer thickness) of V_s over the depth of the profile.

Period is calculated for both lower and upper bound of measured shear wave velocities given above.

Lower Bound:

$$V_s|_{avg} = \frac{340(95) + 475(15) + 1040(8)}{118} = 404 \text{ fps.}$$

$$T_{\substack{lower\\bound}} = \frac{4 \cdot 118'}{404 \text{ fps}} = \underline{\underline{1.17 \text{ s}}}$$

Upper bound:

$$V_s|_{avg} = \frac{475(95) + (489)(15) + (8)(1410)}{118} = 540 \text{ fps}$$

$$T_{\substack{upper\\bound}} = \frac{4 \cdot 118'}{540 \text{ fps}} = \underline{\underline{0.87 \text{ s}}}$$

Job ~~AEP - Big Sandy~~ Project No. 13815151 Sheet ___ of ___

Description Calculation of Factor of Safety Computed by N.S. Date 6/1/2012

Checked by VKG Date 6/1/2012

Reference

PROBLEM: CALCULATE FACTOR OF SAFETY AGAINST LIQUEFACTION.

CALCULATION:

$$FS_{LIQ} = \frac{CRR \leftarrow \text{Cyclic Resistance Ratio}}{CSR \leftarrow \text{Cyclic Stress Ratio}}$$

Estimated CRR from Big Sandy lab test data is 0.1917

CSR profile (CSR Vs Depth) is obtained from SHAKE output.

Sample Calculation ⇒

Lets pick a CSR value for Motion 1 - direction 1 at depth 2.5 ft below ground surface,

$$CSR_{@2.5'} = 0.1140$$

$$FS_{@2.5'} = \frac{0.1917}{0.1140} = 1.68$$

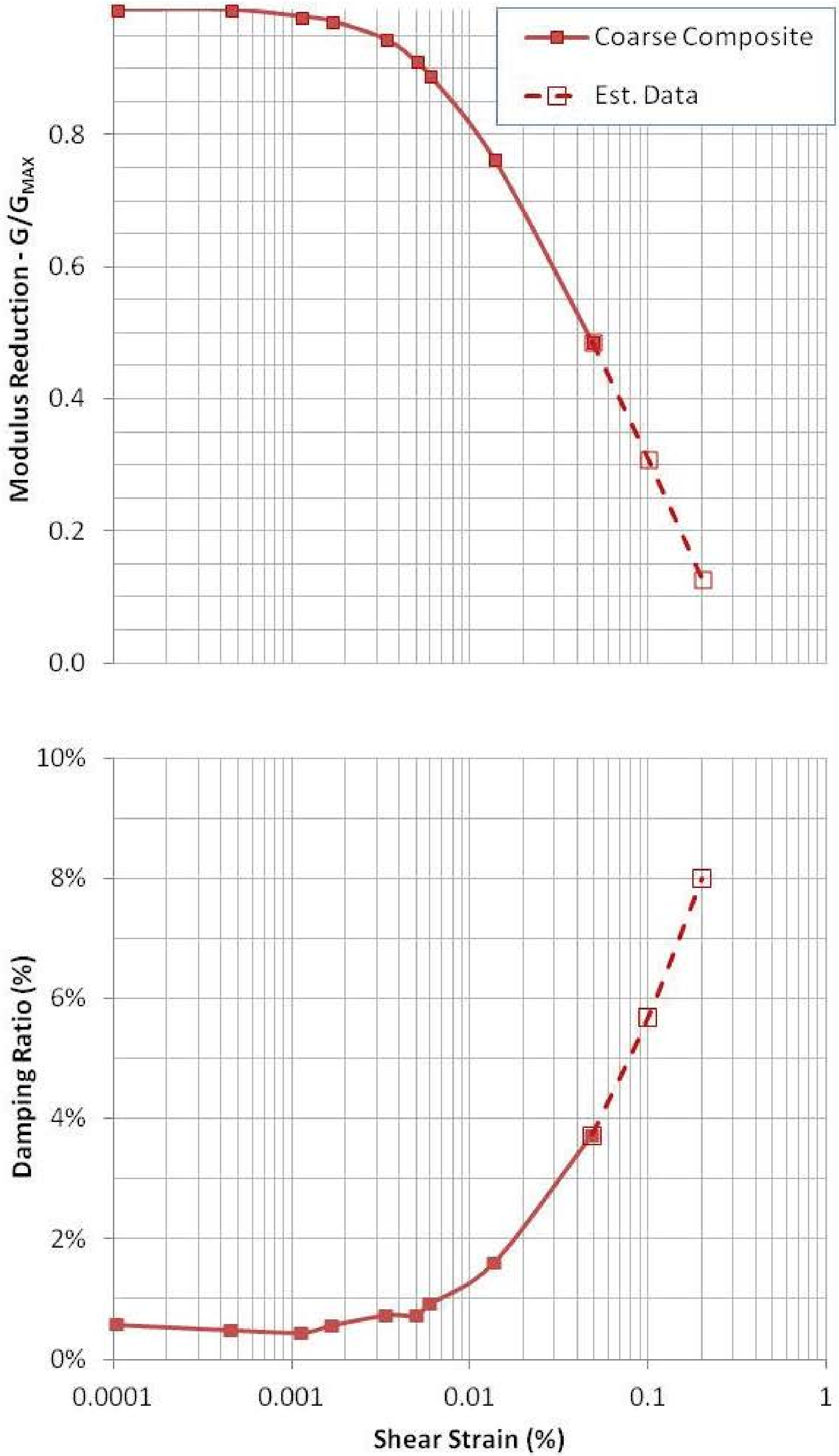
attached tables Present the complete Calculations in excel sheets.

Liquefaction Potential Analysis - Complete Factor of Safety Calculations
AEP Big Sandy Plant

		CSR						CRR	Factor of Safety					
Layer #	Depth (ft)	Mot1-Dir1	Mot1-Dir2	Mot2-Dir1	Mot2-Dir2	Mot3-Dir1	Mot3-Dir2		Mot1-Dir1	Mot1-Dir2	Mot2-Dir1	Mot2-Dir2	Mot3-Dir1	Mot3-Dir2
1	-2.5	0.1140	0.1297	0.1478	0.1133	0.1427	0.1026	0.1917	1.6822	1.4778	1.2967	1.6925	1.3438	1.8681
2	-7.5	0.1124	0.1247	0.1453	0.1123	0.1410	0.1014	0.1917	1.7057	1.5370	1.3193	1.7078	1.3596	1.8910
3	-12.5	0.1078	0.1116	0.1382	0.1096	0.1360	0.0977	0.1917	1.7785	1.7172	1.3870	1.7491	1.4097	1.9623
4	-17.5	0.0969	0.0981	0.1283	0.1034	0.1252	0.0904	0.1917	1.9786	1.9543	1.4945	1.8544	1.5310	2.1204
5	-22.5	0.0887	0.0927	0.1167	0.0984	0.1119	0.0827	0.1917	2.1623	2.0671	1.6426	1.9474	1.7126	2.3175
6	-27.5	0.0828	0.0880	0.1062	0.0959	0.1022	0.0760	0.1917	2.3151	2.1796	1.8050	1.9988	1.8749	2.5223
7	-32.5	0.0772	0.0843	0.1018	0.0934	0.0966	0.0703	0.1917	2.4843	2.2749	1.8836	2.0514	1.9835	2.7263
8	-37.5	0.0710	0.0806	0.0966	0.0909	0.0923	0.0686	0.1917	2.7017	2.3783	1.9843	2.1084	2.0780	2.7956
9	-42.5	0.0698	0.0770	0.0908	0.0869	0.0866	0.0685	0.1917	2.7475	2.4899	2.1102	2.2070	2.2130	2.7989
10	-47.5	0.0688	0.0737	0.0876	0.0810	0.0840	0.0672	0.1917	2.7864	2.5994	2.1888	2.3658	2.2809	2.8510
11	-52.5	0.0668	0.0709	0.0854	0.0769	0.0806	0.0658	0.1917	2.8679	2.7024	2.2455	2.4921	2.3784	2.9125
12	-57.5	0.0641	0.0681	0.0819	0.0746	0.0742	0.0660	0.1917	2.9896	2.8141	2.3414	2.5701	2.5836	2.9066
13	-62.5	0.0611	0.0658	0.0771	0.0716	0.0656	0.0656	0.1917	3.1363	2.9149	2.4873	2.6763	2.9209	2.9205
14	-67.5	0.0579	0.0633	0.0708	0.0682	0.0677	0.0655	0.1917	3.3129	3.0274	2.7073	2.8122	2.8336	2.9249
15	-72.5	0.0543	0.0623	0.0655	0.0648	0.0681	0.0649	0.1917	3.5330	3.0780	2.9285	2.9593	2.8148	2.9554
16	-77.5	0.0519	0.0612	0.0611	0.0619	0.0684	0.0650	0.1917	3.6941	3.1348	3.1395	3.0947	2.8047	2.9489
17	-82.5	0.0539	0.0599	0.0571	0.0590	0.0677	0.0644	0.1917	3.5581	3.1984	3.3584	3.2497	2.8334	2.9745
18	-87.5	0.0555	0.0585	0.0539	0.0561	0.0635	0.0631	0.1917	3.4555	3.2773	3.5538	3.4197	3.0193	3.0380
19	-92.5	0.0562	0.0567	0.0528	0.0538	0.0587	0.0614	0.1917	3.4102	3.3793	3.6282	3.5617	3.2661	3.1197
20	-97.5	0.0561	0.0547	0.0524	0.0523	0.0566	0.0591	0.1917	3.4161	3.5047	3.6613	3.6663	3.3842	3.2447
21	-102.5	0.0525	0.0540	0.0513	0.0477	0.0549	0.0527	0.1917	3.6539	3.5528	3.7357	4.0195	3.4925	3.6344
22	-107.5	0.0483	0.0499	0.0469	0.0455	0.0521	0.0485	0.1917	3.9670	3.8433	4.0841	4.2166	3.6778	3.9514
23	-112.5	0.0443	0.0467	0.0479	0.0445	0.0505	0.0443	0.1917	4.3272	4.1037	4.0028	4.3034	3.7924	4.3270
24	-117.5	0.0414	0.0446	0.0477	0.0430	0.0516	0.0427	0.1917	4.6275	4.3016	4.0226	4.4551	3.7184	4.4872
25	-122.5	0.0394	0.0432	0.0468	0.0414	0.0516	0.0415	0.1917	4.8687	4.4416	4.1003	4.6251	3.7135	4.6166

ATTACHMENT C

SHEAR MODULUS AND DAMPING CURVES



Coarse Composite
Est. Data
Modulus Reduction - G/G_MAX
0.0
0.2
0.4
0.6
0.8
Damping Ratio (%)
0%
2%
4%
6%
8%
10%
0.0001
0.001
0.01
0.1
1
Shear Strain (%)

COVER SYSTEM GEOCOMPOSITE TRANSMISSIVITY

BIG SANDY POWER PLANT ASH POND CLOSURE PROJECT

GEOTECHNICAL CALCULATIONS

I. PURPOSE

The purpose of this analysis is to calculate the required transmissivity of the final cover system geocomposite. The following sections summarize the methodology, assumptions, and results of the final cover system geocomposite design for the proposed Big Sandy Fly Ash Pond Closure located at the American Electric Power's (AEP) Big Sandy Plant. For further detail on the specific calculations performed, refer to the corresponding input/output data provided in the attachments.

II. METHODOLOGY

Slope geometry and cover system materials were established from the proposed permit design drawings and are summarized below.

The cap system will be installed at the final surface of the crest and sideslopes overlying the waste material. The proposed maximum sideslope angle for the proposed cap system is 4 Horizontal to 1 Vertical (4H:1V). The longest 4H:1V slope length is approximately 120 feet. Water collected in the geocomposite will be daylighting to surface water features.

Layers and layer thicknesses for the final cover system cap are anticipated as follows:

Table 1. Layer Summary for the Final Cover System

THICKNESS	LAYER
Final Cover	
6 in	Vegetative Cover Soil
18 in	Protective Cover Soil
n/a	Geocomposite Drainage Layer
n/a	PVC Geomembrane

n/a – thickness of layer is small (negligible)

The geocomposite must be designed to transmit the expected flow of water into the geocomposite through the overlying cover soil. It is assumed that the maximum flow into the geocomposite will occur when the overlying soil is saturated. The cover soil was modeled with a conservative long-term permeability (k_c) of 5x10^{-5} cm/sec. The following equation can be used to model the relationship between the average head level in the geocomposite (h_{avg}), the slope length (L) and angle (β), the permeability of the cover soil (k_c) and the required permeability of the geocomposite (k_d).:

$$h_{avg} = \frac{k_c L(\cos\beta)}{k_d(\sin\beta)} \qquad (1)$$

The minimum required transmissivity (T_{design}) of the geocomposite drainage layer is determined by limiting the average head (h_{avg}) on the drainage layer to the thickness of the drainage layer (t_d). For the purpose of calculations, a geocomposite thickness of 0.5 cm (0.2 in or 200 mils) was utilized. Limiting the average head to the approximate thickness of the drainage layer ensures drainage occurs within the drainage layer.

The minimum required transmissivity (T_{design}) of the geocomposite is calculated using the following equation (from "Designing with GRI Standard GC8," Narejo and Richardson, 2003):

$$T_{design} = k_d t_d \qquad (2)$$

Where,

t_d = Thickness of the Drainage Layer

A factor of safety is then applied to T_{design} to obtain the allowable transmissivity (T_{allow}) (from "Designing with GRI Standard GC8," Narejo and Richardson, 2003), as shown in the equation below:

$$FS = \frac{T_{allow}}{T_{design}} \qquad (3)$$

Reduction factors are then applied to the allowable transmissivity (T_{allow}), which represents long-term in-situ conditions. The decrease in flow capacity from the minimum required transmissivity (T_{spec}) to the long-term in-situ conditions is described by reduction factors (RF) as given in "GSI White Paper #4: Reduction Factors Used in Geosynthetic Design" (Koerner and Koerner, 2005). The equation below was used to determine T_{spec}:

$$T_{allow} = \frac{T_{spec}}{RF_{IN} RF_{CR} RF_{CC} RF_{BC}} \qquad (4a)$$

Substituting equation (3) and solving for T_{spec}:

$$T_{spec} = T_{design} FS \times RF_{IN} RF_{CR} RF_{CC} RF_{BC} \qquad (4b)$$

Typical values for reduction factors for landfill covers from Koerner and Koerner (2005) and Narejo and Richardson (2003) are included in **Attachment A**. Values chosen for reduction factors were taken from the range of values presented and are summarized below:

Table 2. Reduction Factor Summary

Reduction Factor	Value	Comments
Intrusion, RF_{IN}	1.4	Testing to be conducted with steel plates; 1.4 recommended by GSI White Paper #4 – included in Attachment A
Creep, RF_{CR}	1.1	Low loading conditions
Chemical Clogging, RF_{CC}	1.2	Potential for some precipitate from onsite cover soil
Biological Clogging, RF_{BC}	2.3	Middle of range from Narejo (2004) – included in Attachment A
Drainage Factor of Safety, *FS*	2	Conventionally factor of safety for drainage applications

III. RESULTS OF ANALYSIS

As discussed in Section II, the geocomposite drainage layer must be selected with adequate transmissivity to limit the depth of flow to the thickness of the geocomposite. Conservative assumptions regarding factors of safety, reduction factors, and the assumed saturated hydraulic conductivity of the overlying soils are considered when calculating the specified minimum transmissivity of the final cover system geocomposite.

The design drainage length is approximately 120 feet (length of longest slope), corresponding to the longest 4H:1V slope on the project. **The minimum transmissivity required to maintain drainage inside the geocomposite on the 4H:1V slopes is 6.22×10^{-4} m^2/sec. This is the minimum required value for testing and manufacturer's specifications, (T_{spec}).** Refer to **Attachment B** for supporting calculations.

IV. REFERENCES

Narejo, D. and Richardson, G. (2003), "Designing with GRI Standard GC8." *GFR*, vol. 21, no. 6, pp. 20–23.

Koerner, R. M. and Koerner, G. R. (2005), "GSI White Paper #4 Reduction Factors (RFs) Used in Geosynthetic Design." Geosynthetic Institute

ATTACHMENT B

Application Area	Range of Reduction Factor Values			
	RF_{IN}	RF_{CR}*	RF_{CC}	RF_{BC}
Sport fields	1.0 to 1.2	1.0 to 1.5	1.0 to 1.2	1.1 to 1.3
Capillary breaks	1.1 to 1.3	1.0 to 1.2	1.1 to 1.5	1.1 to 1.3
Roof and plaza decks	1.2 to 1.4	1.0 to 1.2	1.0 to 1.2	1.1 to 1.3
Retaining walls, seeping rock, and soil slopes	1.3 to 1.5	1.2 to 1.4	1.1 to 1.5	1.0 to 1.5
Drainage blankets	1.3 to 1.5	1.2 to 1.4	1.0 to 1.2	1.0 to 1.2
<u>Infiltrating water drainage for landfill covers</u>	<u>1.3 to 1.5</u>	<u>1.1 to 1.4</u>	<u>1.0 to 1.2</u>	<u>1.5 to 2.0</u>
Secondary leachate collection (landfills)	1.5 to 2.0	1.4 to 2.0	1.5 to 2.0	1.5 to 2.0
Primary leachate collection (landfills)	1.5 to 2.0	1.4 to 2.0	1.5 to 2.0	1.5 to 2.0
Wick Drains (PVDs)	1.5 to 2.5	1.0 to 2.5	1.0 to 1.2	1.0 to 1.2
Highway edge drains	1.2 to 1.8	1.5 to 3.0	1.1 to 5.0	1.0 to 1.2

From Koerner R., and Koerner G. (2005) "GSI White Paper #4: Reduction Factors Used in Geosynthetic Design." Geosynthetic Institute.

LONG-TERM PERFORMANCE CONSIDERATIONS FOR GEONET DRAINAGE GEOCOMPOSITES

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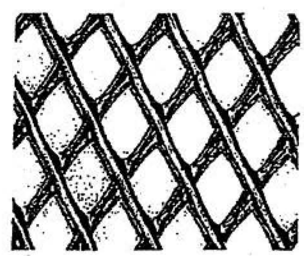
ABSTRACT
Drainage geocomposites have gained increasing acceptance within the engineering community as the material of choice for the lateral conveyance of liquids and gases. The hydraulic performance of these materials is typically expressed as transmissivity or flow rate at site-specific gradient, normal stress and boundary conditions. However, since these materials are visco-elastic in nature, compressive creep can significantly affect their long-term hydraulic performance. In addition to creep, there is the potential for the chemical and biological clogging of the filter geotextile and the geonet drainage core. Over the last several years, significant progress has been made in characterizing the engineering properties of geonet drainage geocomposites and developing models to predict their long-term behaviour on the basis of short-term laboratory tests. Additional work is needed in the area of chemical and biological clogging to further supplement the current information. In addition, the impact of leachate recirculation and higher temperatures in bioreactor landfills on the long-term performance of geocomposites merits further study.

RÉSUMÉ
Drainage geocomposites have gained increasing acceptance within the engineering community as the material of choice for the lateral conveyance of liquids and gases. The hydraulic performance of these materials is typically expressed as transmissivity or flow rate at site-specific gradient, normal stress and boundary conditions. However, since these materials are visco-elastic in nature, compressive creep can significantly affect their long-term hydraulic performance. In addition to creep, there is the potential for the chemical and biological clogging of the filter geotextile and the geonet drainage core. Over the last several years, significant progress has been made in characterizing the engineering properties of geonet drainage geocomposites and developing models to predict their long-term behaviour on the basis of short-term laboratory tests. Additional work is needed in the area of chemical and biological clogging to further supplement the current information. In addition, the impact of leachate recirculation and higher temperatures in bioreactor landfills on the long-term performance of geocomposites merits further study.

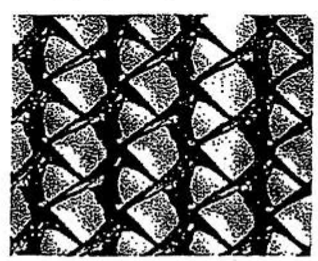
1. INTRODUCTION

A geonet drainage geocomposite consists of a geonet core and a geotextile, where the geotextile is heat-laminated to one or both sides of the geonet. The geonet is made of extruded High Density Polyethylene (HDPE) in a manner that forms a relatively open structure ideal for the in-plane transmission of liquids and/or gases. The geotextile serves as a filter and separator, while the geonet core is intended to provide the lateral flow capacity. Geotextiles currently used for this purpose are almost exclusively of the nonwoven needlepunched type made from polypropylene (PP) or polyester (PE) resins. Geonet drainage geocomposites are differentiated primarily by the structure of the geonet as illustrated in Figures 1 (a) and (b).

Drainage geocomposites are used predominantly in environmental applications such as landfills and lagoons. However, there is growing interest in the use of these materials in such civil engineering applications as roadways, buildings, canals, etc. Landfills – the dominant market segment for these materials – are characterized by relatively large areas with slopes ranging from as low as 2% to as high as 33%. Specifically, there are four applications in landfills where drainage geocomposites are utilized: i) landfill cover drainage layer, ii) landfill gas vent layer, iii) landfill leachate collection and removal



(a) biplanar geonet



(a) triplanar geonet

Figure 1 Plan view of biplanar and triplanar geonets.

layer, and iv) landfill leakage detection layer. The design of each of these layer may involve the following performance properties of the drainage geocomposite: i) flow rate or transmissivity (heretofore referred to as transmissivity), ii) interface shear strength, and iii) filtration properties (including "filtration opening size" and permeability). This paper deals with only one of the above three performance characteristics, namely transmissivity.

The transmissivity of drainage geocomposites is a function of available pore-space as illustrated in Figure 2. Any mechanism that tends to reduce this pore space would decrease geocomposite transmissivity. Currently known factors include the following: i) geonet creep, ii) geotextile intrusion into the core structure, iii) chemical clogging within the core, and iv) biological clogging within the core. The reader should note that the concern with biological and chemical clogging of the drainage geocomposite core is differentiated here from a similar concern for the drainage geocomposite filter geotextile. Although mechanisms involved may be similar, the testing and design must be performed separately for the filter and drainage media.

Figure 2 Cross-section of a biplanar drainage geocomposite.

2. TRANSMISSIVITY AND REDUCTION FACTORS

Transmissivity is defined as the flow rate of water transmitted through a unit width of the product under a specific hydraulic gradient as measured in a laboratory test. The transmissivity test is performed using the type of equipment shown schematically in Figure 3. For the test to provide a transmissivity value that can be used in design, the specimen top and bottom boundaries as well as the gradient should be the same as in the field. The test is typically continued for a reasonably long enough time to include the effect of initial compression, and intrusion of geotextile into the geonet structure. The current state-of-the-practice in the US is represented by GRI GC8 which requires the test to be continued for 100 hours. The resulting value is then modified to include the effect of creep, chemical clogging and biological clogging as in Equation 1 (from GRI GC8, 2001):

$$\theta_{allow} = \frac{\theta_{100}}{RF_{cr} x RF_{cc} x RF_{bc}} \quad [1]$$

where θ_{allow} = allowable transmissivity for the specific product being considered (m^2/sec), θ_{100} = 100-hour performance transmissivity from actual test, RF_{cr} = reduction factor for creep of the geonet core, RF_{cc} = reduction factor for chemical clogging, RF_{bc} = reduction factor for biological clogging.

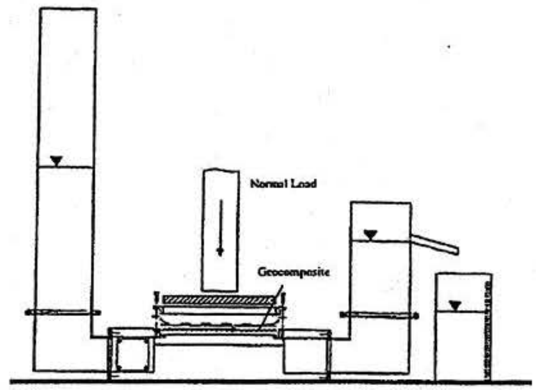


Figure 3 Schematics of the transmissivity test (Richardson et. al., 2000).

It must be noted here that certain versions of Equation 1 use such additional reduction factors as geotextile intrusion into the geonet structure and for "particulate clogging" of the geonet core. It is the author's opinion that a reduction factor for intrusion may not be necessary as the performance transmissivity test already includes this effect. The concern regarding particulate clogging of drainage core can, and should, be addressed by proper geotextile filter design so that fines do not pass the geotextile in the first place. This should then be supplemented with proper construction quality assurance (CQA) procedures that minimize infiltration of dust into the drainage core during the installation process.

The allowable value of transmissivity from Equation 1 must then be compared with "required transmissivity" to calculate a factor of safety as provided in the equation below:

$$FS = \frac{\theta_{allow}}{\theta_{req}} \quad [2]$$

where FS = factor of safety for drainage, and θ_{req} = required transmissivity (m^2/sec) for a specific project.

The three reduction factors in the denominator of Equation 1 along with the performance transmissivity value (θ_{100}) determine whether a particular product is acceptable for a given project. It is recognized that this decision can be only as good as the quality of the data used to arrive at the reduction factors. The state-of-the-practice, limitations of current approach and the need for future research on reduction factors is discussed in the following sections.

2.1 Reduction Factor for Creep, RF_{cr}

Reduction factor for creep is intended to account for the time-dependent compression of the geonet core component of the geocomposite. It should be based on actual testing of the geonet core component of the geocomposite. Geonets can be tested for creep according to one of the two methods currently being used in the industry: a) conventional method, and b) accelerated method. The main difference between the two procedures is the test temperature. In conventional creep method, tests are performed at ambient temperature of around 20 degrees Celsius or any other site-specific temperature. In the accelerated procedure, the testing is performed at several elevated temperatures and the resulting data is then extrapolated to the ambient temperature through time-temperature superposition. Further details of creep testing and the associated calculations can be found in Narejo & Allen (2004). The advantage of the accelerated testing over conventional methods is that the required information can be obtained within hours versus the 14 months required by the conventional tests. Moreover, accelerated testing means that different product formulations and variations can be evaluated economically and within a reasonable time, and more data can be generated for statistical analysis.

Irrespective of whether accelerated or conventional creep testing is performed, the resulting information is of the form presented in Figure 4. For the product and test conditions represented by Figure 4, creep rate is constant at any given normal stress. However, the creep rate increases with an increase in normal stress. Since the creep rate is linear on a semi-log scale, the curves can be extended to obtain thickness at the design life of a project, say 50 years. This value of thickness can then be used to calculate the creep reduction factor, RF_{cr} (Narejo & Allen, 2004) for site-specific stress. Depending on the quality of the product, this creep reduction factor is typically around 1.1 to 1.2 for low stress (<50 kPa) but can be close to 2 for pressures higher than 700 kPa (Narejo & Allen, 2004).

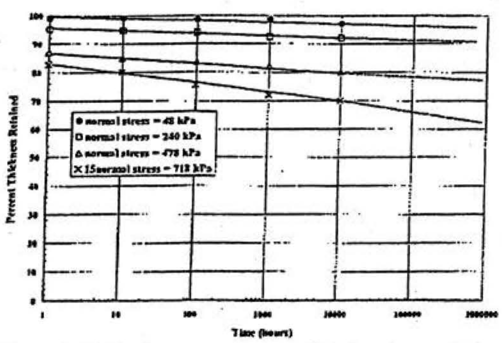


Figure 4 Typical creep response of biplanar geonets.

The creep of geonets is influenced significantly by the physical properties and structure of the geonet, including rib structure, mass, thickness, etc. As such, creep data for geonets is typically specific to products and its generalization is currently not possible.

2.2 Reduction Factor for Chemical Clogging, RF_{cc}

Chemical clogging of drainage materials in landfills results from chemical processes such as the precipitation of calcium carbonate, manganese carbonate and other insoluble substances (e.g., sulfides, chlorides and silicates). A reduction factor for chemical clogging, RF_{cr}, is intended to account for the influence of chemical clogging on the transmissivity of drainage geocomposites. Current industry practice, at least within the US, is to use reduction factors for chemical clogging proposed in the textbook *Designing with Geosynthetics* (Koerner, 1998) and GRI procedure GC8. The values are reproduced in Table 1.

Table 1 Chemical clogging reduction factors (*from* Koerner 1998 and GRI GC8).

Application	Reduction Factor for Chemical Clogging (RF_{cc})
Landfill covers	1.0 to 1.2
Primary leachate collection	1.5 to 2.0
Secondary leachate collection	1.1 to 1.5

2.3 Reduction Factor for Biological Clogging, RF_{bc}

Biological clogging refers to the growth of micro-organisms on and within the drainage media. Biological growth depends on the presence of a suitable bio-chemical environment and nutrients which sustain growth. The biomass growth within the drainage media would reduce the opening size and, hence transmissivity. A reduction factor for biological clogging, RF_{bc}, is used to account for the influence of the biological clogging on geocomposite transmissivity. Currently, the only sources of reference on biological clogging of drainage geocomposites are the geosynthetics textbook – *Designing with Geosynthetics* – and GRI GC8. Suggested reduction factors for biological clogging from these two sources are cited in Table 2.

Table 2 Biological clogging reduction factors (*from* Koerner, 1998, and GRI GC8).

Application	Reduction Factor for Biological Clogging (RF_{bc})
Landfill covers	1.2 to 3.5
Primary leachate collection	1.1 to 1.3
Secondary leachate collection	1.1 to 1.3

3. CRITIQUE OF REDUCTION FACTORS AND RECOMMENDATIONS FOR FURTHER RESEARCH

Reduction factor for creep can be tested in the laboratory with a reasonable degree of confidence as the site conditions can be conveniently modelled. The main variable in creep testing is normal stress, which is determined from the layout and the final contours of the site. As such geosynthetic manufacturers have been testing geonets for creep and a reasonable amount of data already exists. Unfortunately, creep results for geonets are product-specific and each commercially available geonet must be evaluated separately. The SIM Method offers a technique which can help generate a significant amount of data at a reasonable cost. However, manufacturers must demonstrate the validity of this method by performing comparable tests with the conventional technique.

Chemical and biological clogging is very difficult to model in the laboratory. The main reason for this is that the bio-chemical environment for each site may be different. Hence it is difficult to develop a test program the results of which can then be applied uniformly to the design process. It is for this reason that most of the published literature on this topic is of qualitative nature as far as its utilization during the design process is concerned. There is a need for more extensive testing that examines the basic process of clogging in what may be idealized or extreme conditions. This information may then be used to make an "educated guess" about a particular site based on anticipated waste stream and hydrologic conditions.

4. ELEVATED TEMPERATURES AND LEACHATE RECIRCULATION

Bioreactor landfills involve leachate recirculation to accelerate decomposition of the waste mass. Leachate recirculation poses two important challenges to the use of drainage geocomposites: i) elevated temperatures, and ii) higher flow requirements. Elevated temperatures would tend to increase reduction factors for creep, thus lowering the allowable transmissivity. However, the required transmissivity itself may need to be increased beyond that for conventional projects to account for a higher flow of liquid through the drainage layer. Not much is known at this time about the response of drainage geocomposites to leachate circulation. Much research needs to be done in this area to develop recommendations for the design purpose.

5. SUMMARY AND RECOMMENDATIONS

The long-term hydraulic performance of drainage geonets and geocomposites depends on many material as well as site characteristics. A performance transmissivity test provides a 100-hour transmissivity or flow-rate value which can then be further modified to account for site-specific and time-dependent factors. In this regard, there are three specific reductions factors of creep, chemical clogging and biological clogging. Geosynthetic manufacturers have been performing creep tests on their products to develop information on creep reduction factors. However, very limited information is available on biological clogging and chemical clogging of drainage materials. Manufacturers and academics should collaborate to develop further information in this regard. It must be recognized that a model that represents "general" application conditions is very difficult to develop. On the other hand, the tendency to use extremely aggressive conditions in the models provides little practical information for designer. Instead, it may be useful to perform idealized set of testing which can then be analyzed to develop general recommendations for the purpose of design.

6. REFERENCES

GRI GC8. 2001. Geosynthetic Research Institute. Standard guide for determination of the allowable flow rate of a drainage geocomposite.

Koerner, R., 1998. Designing with Geosynthetics. Fourth Edition. Prentice Hall, New Jersey, USA.

Narejo, D., and Allen, S., 2004. Using the Stepped Isothermal Method for geonet creep evaluation. Third European Geosynthetic Conference, 01-03 March 2004, Munich, Germany, pp. 539-544.

Richardson, G., Giroud, J., and Zhao, A. 2000. Tenax Corporation. Design of lateral drainage systems.

ATTACHMENT A

FINAL COVER SYSTEM
GEOCOMPOSITE DESIGN CALCULATIONS

Symbol	Description	Values	Units	US units		Comments
β	angle of slope	14.04	degrees	25.0	%	*input percent slope in US units cell*
L	slope length	3,658	cm	120	feet	*longest length betwn drainage outlets to be conservative*
k_d	permeability of drainage layer	1.44E+00	cm/sec			*Value required to make $h_{avg} = t_d$*
t_d	thickness of drainage layer	0.51	cm	0.2	inches	*minimum 200 mil geocomposite*
k_c	permeability of protective material	5.00E-05	cm/sec			*Conservative long term cover soil permeability*
t_c	thickness of protective material	60.96	cm	2	feet	
RF_{IN}	Reduction Factor for intrusion	1.40				*Middle of range from GSI White Paper #4*
RF_{CR}	Reduction Factor for creep	1.10				*Low loading*
RF_{CC}	Reduction Factor for chemical clogging	1.20				*Precipitate not anticipated*
RF_{BC}	Reduction Factor for biological clogging	2.30				*Middle of range from Narejo (2004)*
FS	Factor of Safety	2.00				
T_{design}	Design/Minimum Transmissivity =kd*Td	7.32E-05	m^2/sec			
T_{spec}	Spec./Lab Transmissivity =ΣRF*Tdesign*FS	6.22E-04	m^2/sec			

MAIN DAM STABILITY ANALYSIS

BIG SANDY POWER PLANT ASH POND CLOSURE PROJECT

GEOTECHNICAL CALCULATIONS

Calculation Notes URS

Subject: Big Sandy Ash Pond – Stability Analysis		Project Name: Big Sandy Ash Pond Closure
By: Craig Helm	Date: 07 November 2012	Project No: 13815153
Checked By: : Vik Gautam	Date: 26 November 2012	Task No. .50000 File No.:

OBJECTIVE:

The objective is to perform stability analysis on the Main Dam at the Big Sandy Ash Pond. The analysis will be performed at the maximum cross-section, after the upper crest is lowered to EL 656 feet, simulating the long term pond closure configuration of the embankment. Steady-state and seismic loading conditions were evaluated.

BACKGROUND INFORMATION:

Previous analysis of the dam was completed by American Electric Power Service Corp (AEPSC) and is documented in the "Stage 3 Raising, Engineering Report" (AEPSC,1993 Report), dated March 1993 by AEPSC. In 1993 the crest elevation was at EL 675 feet, however the Owner was planning to raise the dam to a crest elevation of 711 feet to increase the ash storage volume. This stability analysis was performed with a crest height of EL 711. Material parameters, stability results and design cross-section (Design Drawings, 1993) were provided in the AEPSC, 1993 Report.

APPROACH AND ASSUMPTIONS:

Cross-Section Geometry

A 2010 topography survey provided by AEP shows the crest of the dam is at El 690. When compared to the design cross-section at elevation 690, the width of the dam from the 2010 survey is largerThe exterior geometries are based on the 2010 topography survey and the design cross-section developed by URS, 2012 (crest El 656). The internal geometries of the cross-section were also developed from the Design Drawings, 1993.

Materials used in the analysis include: Rock-fill, Embankment, Drain, Foundation Soils and Bedrock.

Model geometry of the maximum cross-section are shown in Appendix B

Phreatic Surface

The phreatic surface was estimated based on the elevation of the existing pool, the design cross section and information on the phreatic surface from the AEPSC, 1993 Report. The phreatic surface upstream of the crest was conservatively estimated to be at El 655.5 feet, 0.5 foot below the crest elevation. Over time the phreatic surface should lower as the reservoir drains.
The phreatic surface remained at EL 655.5 through the centerline of the crest, before dropping into the vertical drain on the downstream edge of the crest, remaining in the vertical chimney drain and horizontal blanket drain within the downstream shell of the dam.

Material Properties

Material properties used in the stability analysis (except rockfill) replicate parameters developed in the AEPSC, 1993 Report. The material strength used for the rock-fill in this 1993 report/analysis was c'=0 PSF & phi'=24 degrees, and considered to be extremely low for such a material. Experience gained from other projects on similar material, and engineering judgment, suggest strengths between c'=0 PSF & phi'= 33 to 36 degrees are achieved. A friction angle of phi'=32 degrees was conservatively used for the rock fill in this analysis.

Drained strength parameters were used for materials during the steady state analysis.

Both drained and undrained strength parameters were used during the seismic analysis and 2-stage stability computations were performed. For materials that are considered to behave undrained during seismic events (foundation and embankment clay materials), "two stage strength envelopes" are used. These consist of the drained and undrained strength envelopes for each material. The undrained strength envelope for the foundation soils was selected from two consolidated-undrained (CU or, R) triaxial tests performed on this material (Tables No. 5.1 of the AEPSC, 1993 Report). The undrained strength envelope for the embankment clays was selected from two consolidated-un-drained (CU or, R) triaxial tests performed on this material (Tables No. 5.2 of the AEPSC, 1993 Report).The lower strength from each of the two tests was used.

The un-drained strength used in the second stage stability computation is calculated from the "two stage strength envelope" using linear interpretation between the drained and un-drained strength envelopes.

Material properties used in these analyses are presented in Table 1.

Table 1: Material Properties for the Main Dam

	Unit Weight	Steady State		Seismic (pseudo-static)			
Soil Name	γ_t (pcf)	ϕ' (deg)	c' (psf)	ϕ' (deg)	c' (psf)	ϕ (deg)	c (psf)
Rockfill	110	0	32	0	32	-	-
Embankment	130	0	25	0	25	0	25
Drain	70	0	38	0	38	-	-
Foundation Soils	135	0	25	0	25	0	25
Bedrock	150	8000	0	8000	0	-	-

Seismic Coefficients

The peak ground accelerations at the site is 0.07 g on bedrock. This seismic coefficient was applied during the pseudo-static analysis.

ANALYSIS:

The computer software UTEXAS 4, which utilizes Spencer's method of slices in conjunction with automated search routines, was used to locate and analyze critical shear surfaces. Circular shear surfaces within the crest and downstream slope were evaluated for this analysis. Multiple initial shear surfaces, including deep and shallow surfaces, were developed to determine the critical shear surface.

The stability of the proposed future crest elevations were evaluated using the cross-section geometry, material properties, and phreatic surface conditions as discussed above.

CONCLUSIONS/RESULTS:

The required and computed minimum factor of safety's are shown in table 2 for the seismic and steady state analyses, for the proposed dam geometry (Big Sandy Ash Pond Closure, Crest El=656). Calculated factors of safety for both loading conditions exceed the recommended values.

Table 2: Stability Results

Minimum Factor of Safety	Steady State	Seismic
Recommended (KYDEP)	1.5	1.0
Calculated (UTEXAS4)	1.74	1.32

The input text files, output text files, and graphical output files for the critical shear surface are presented in Attachments A. Three graphical print outs are included

All files associated with the stability analysis can be found at:

Calculation Notes URS

Subject: Big Sandy Ash Pond – Stability Analysis | Project Name: Big Sandy Ash Pond Closure

By: Craig Helm | Date: 07 November 2012 | Project No: 13815153

Checked By: : Vik Gautam | Date: 26 November 2012 | Task No. .50000 | File No.:

G:\PROJECTS\AEP BIG SANDY\10.0_CALCULATIONS_ANALYSIS_DATA\10.03_GEOTECHNICAL \STABILITY\UTEXAS4

ATTACHMENTS:

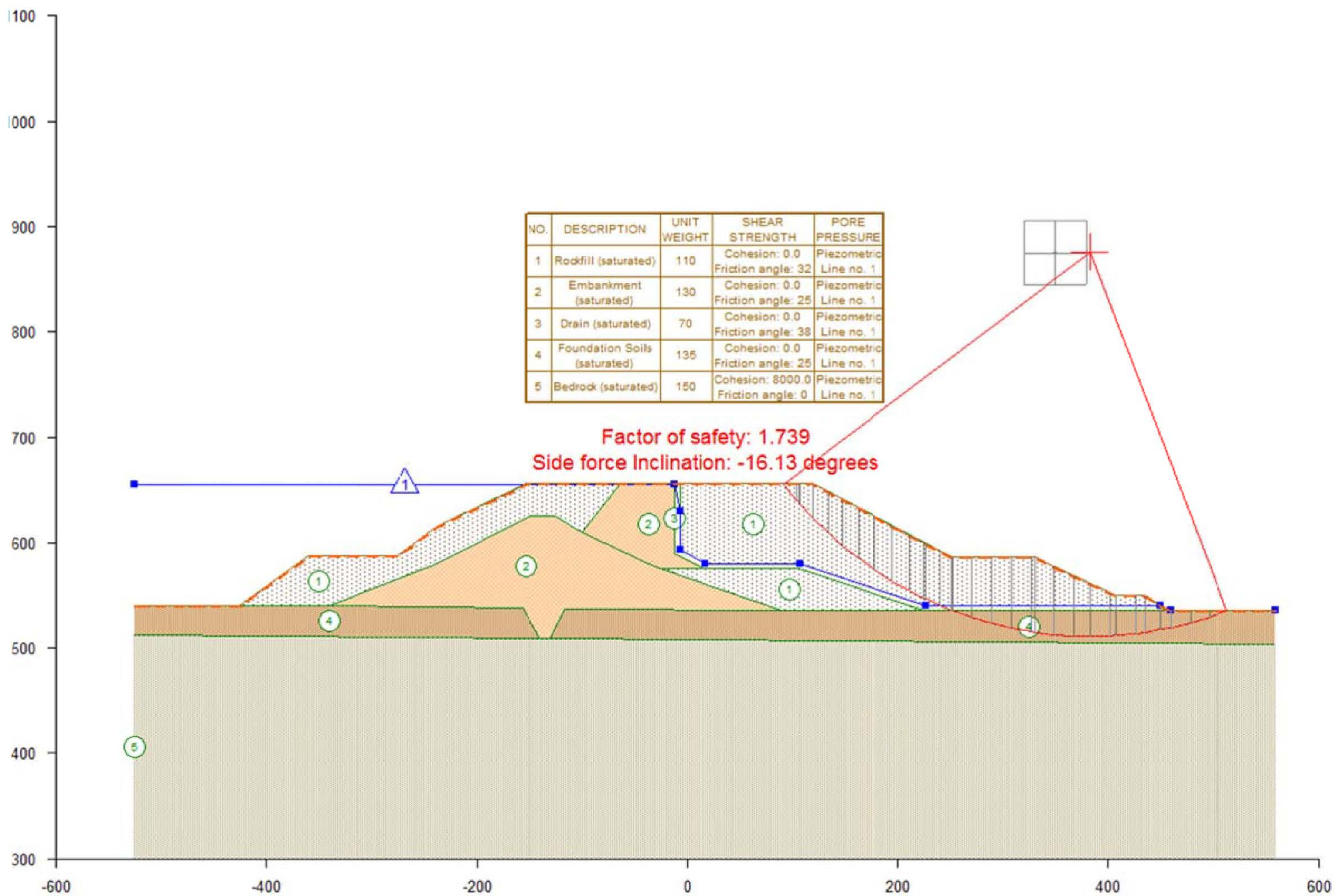
- Attachment A – Utexas4, input, output and graph files
- Attachment B – Geometry (maximum cross-section) and Material Properties

REFERENCES:

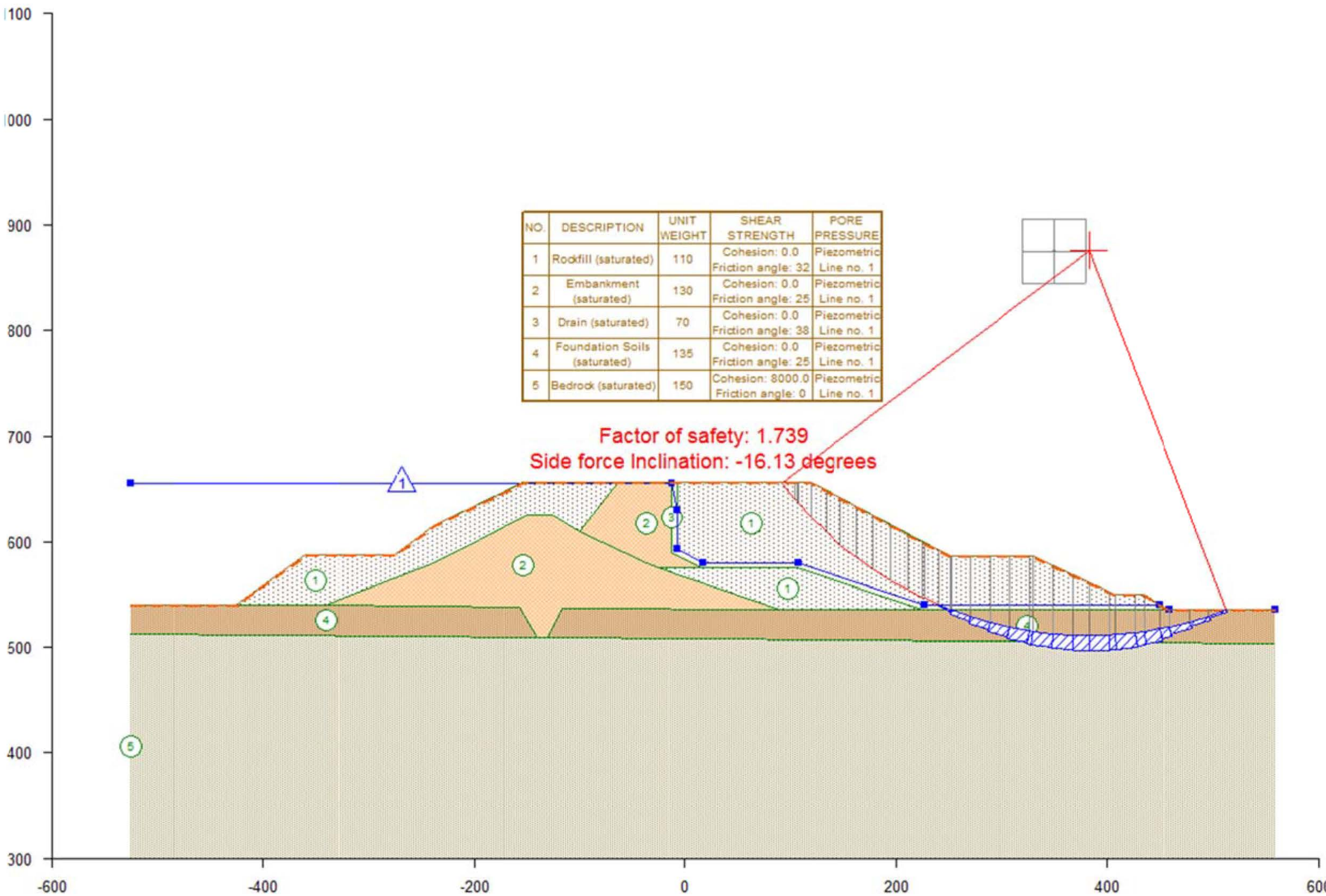
AEPSC, 1993 Report, *Kentucky Power Company Big Sandy Plant Fly Ash Retention Dam Stage 3 Raising, Engineering Report,* prepared by American Electric Power Service Corporation (AEPSC), dated March 1993.

ATTACHMENT A

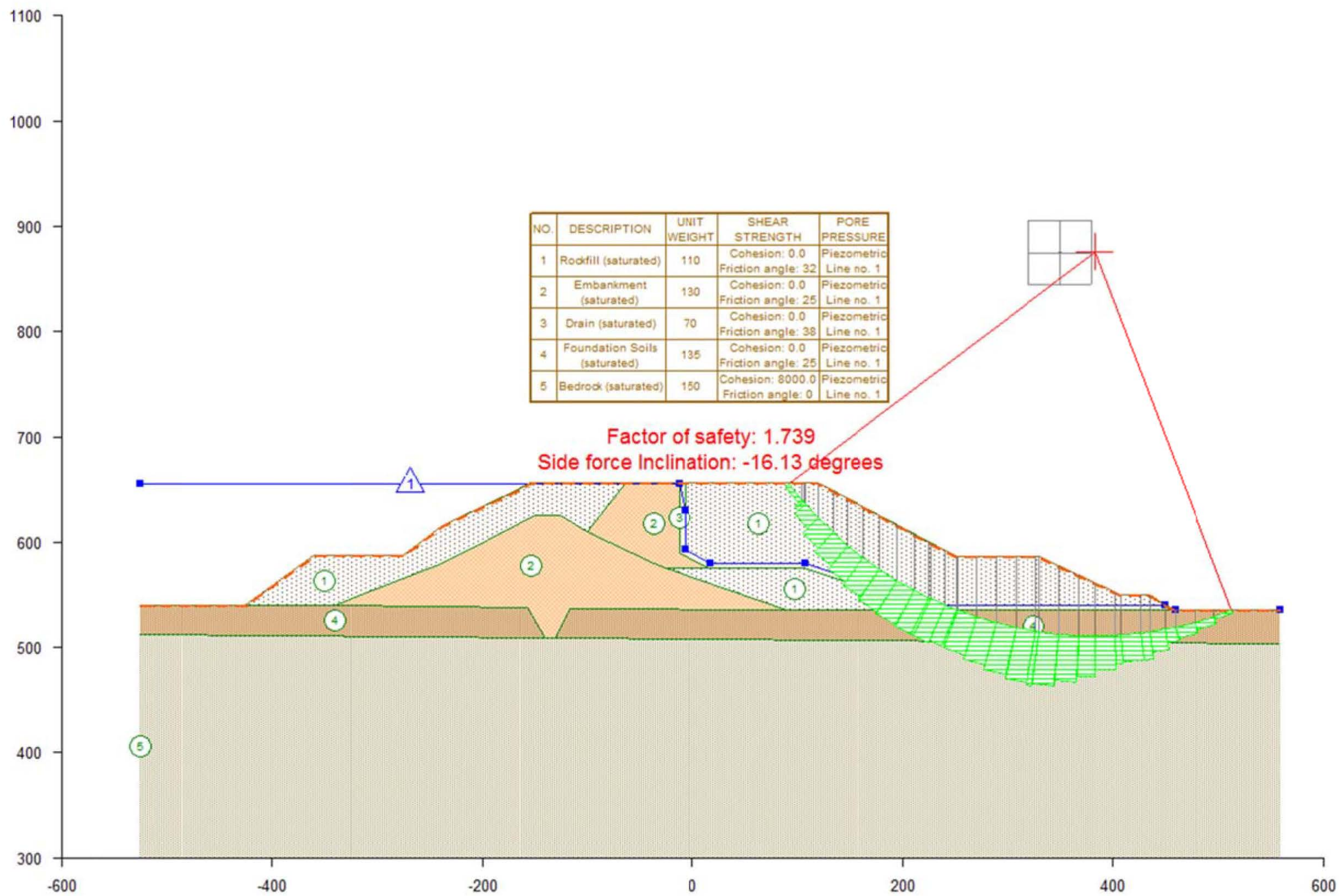
UTEXAS4 – Input, Output and Graphical Files



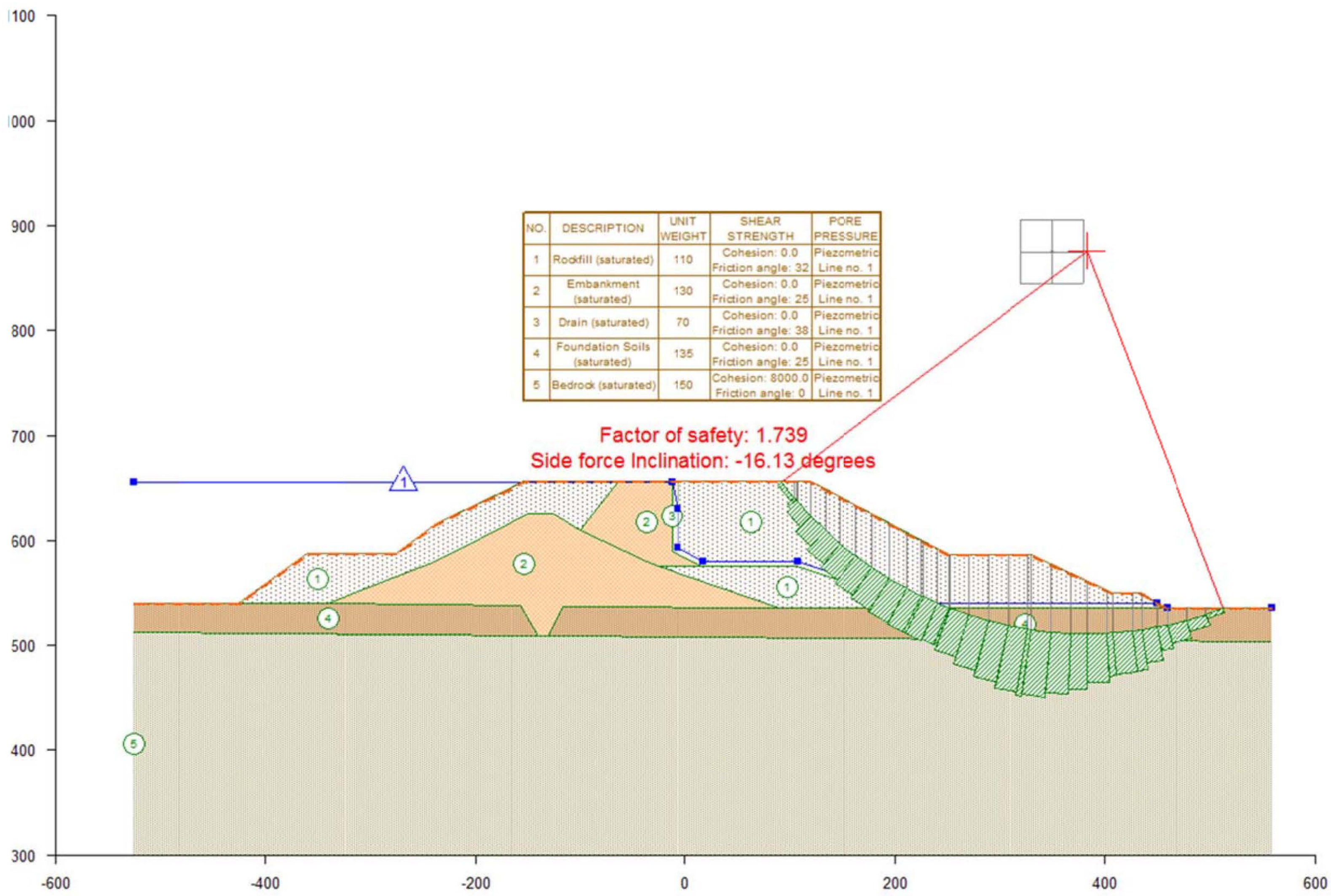
Model Output: 1 Big Sandy Dam – Steady State Analysis – Critical shear surface



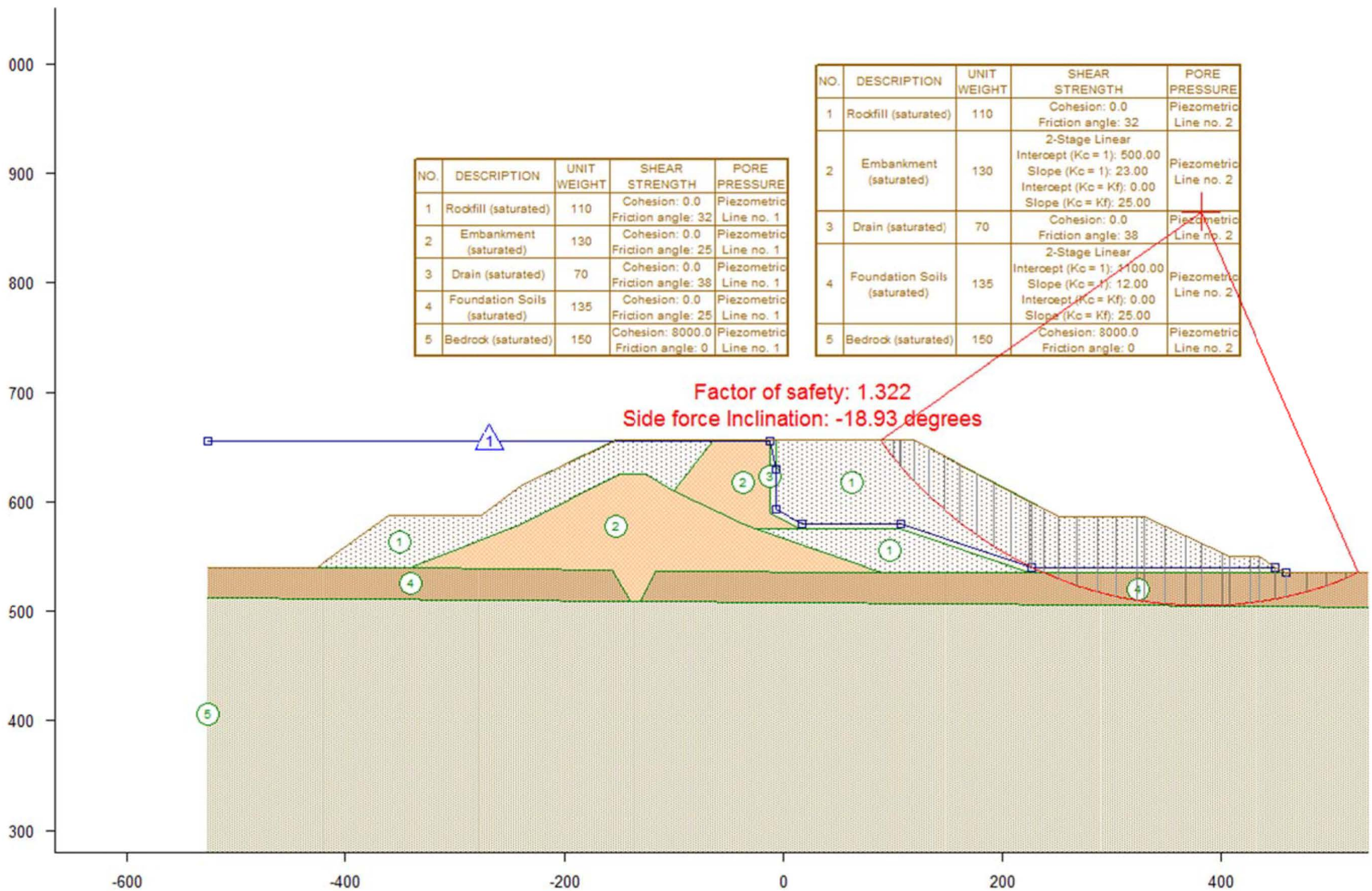
Model Output: 2 Big Sandy Dam – Steady State Analysis – Pore Pressure on shear surface



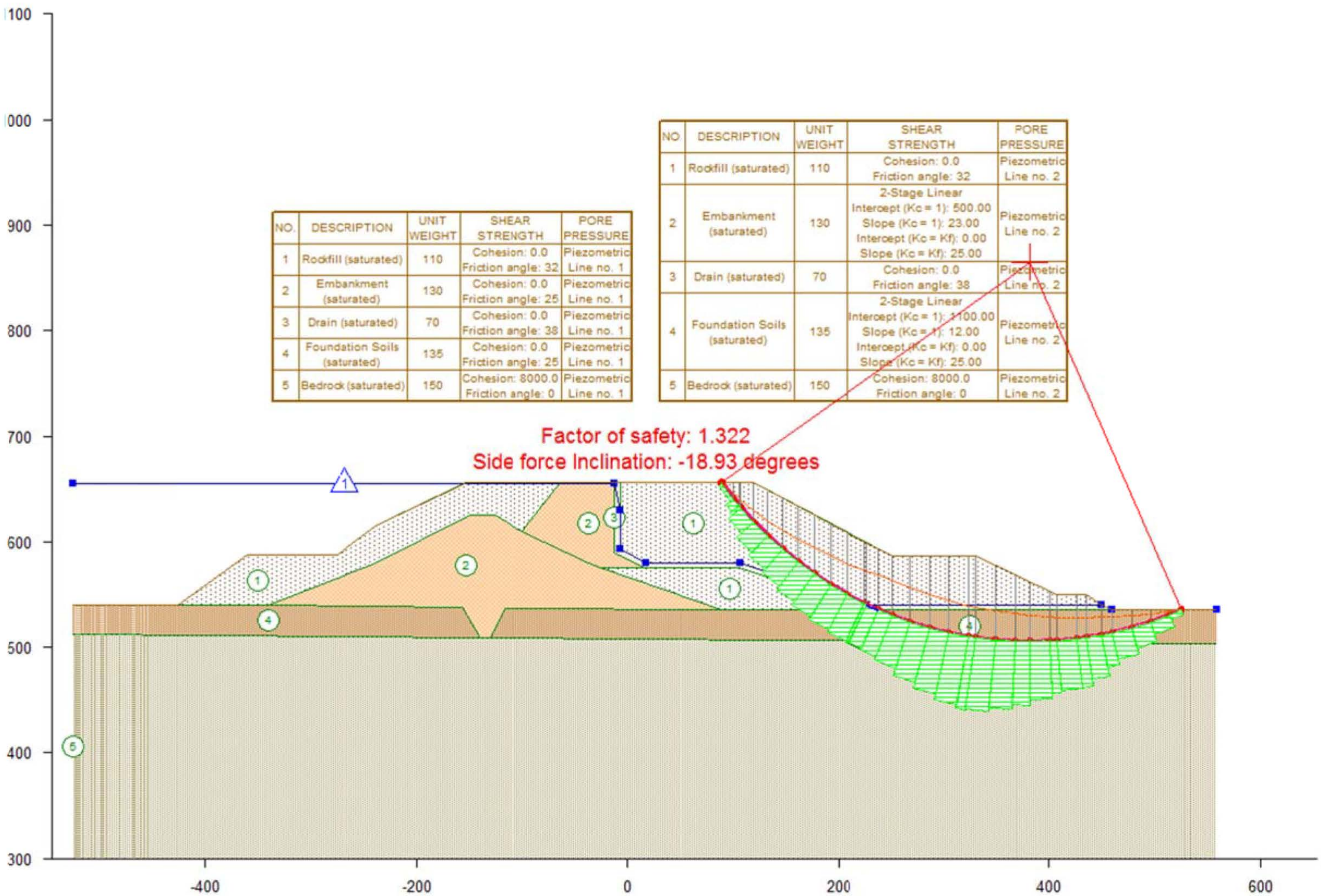
Model Output: 3 Big Sandy Dam – Steady State Analysis – Effective Stress on shear surface



Model Output: 4 Big Sandy Dam – Steady State Analysis – Total Stress on shear surface



Model Output: 5 Big Sandy Dam - Seismic Analysis – Critical shear surface



Model Output 6: Big Sandy Dam - Seismic Analysis – Effective Stress on shear surface

```
TABLE NO. 1
COMPUTER PROGRAM DESIGNATION: UTEXAS4
Originally Coded By Stephen G. Wright
Version No. 4.1.0.8 - Last Revision Date: 11/9/2009
(C) Copyright 1985-2008 S. G. Wright - All rights reserved
*******************************************************************
* RESULTS OF COMPUTATIONS PERFORMED USING THIS SOFTWARE           *
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* BEEN VERIFIED BY INDEPENDENT ANALYSES, EXPERIMENTAL DATA        *
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UTEXAS4 S/N:10001  - Version: 4.1.0.8 - Latest Revision: 11/9/2009
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Time and date of run: Tue Dec 04 11:11:53 2012
Name of input data file: G:\Projects\AEP Big
Sandy\10.0_Calculations_Analysis_Data\10.03_Geotechnical\Stability\UTEXAS4\Steady
State\BigSandy_EL656_SS_circular.txt

BIG SANDY DAM
Max Cross-section - Steady State Seepage
Circular slip surface;
Filename: BigSandy_EL656_SS_circular.dat
PROJECT NO. 13815153

TABLE NO. 3
*************************
* NEW PROFILE LINE DATA *
*************************

----------------------------------------------------------
----- Profile Line No. 1 - Material Type (Number): 1 -----
----------------------------------------------------------
Description: Rockfill - A

Point        X           Y

   1     -425.00      540.00
   2     -360.00      587.00
   3     -275.00      587.00
   4     -238.00      615.00
   5     -153.00      656.00
   6      -63.00      656.00

----------------------------------------------------------
----- Profile Line No. 2 - Material Type (Number): 2 -----
----------------------------------------------------------
Description: Embankment - A

Point        X           Y

   1     -100.00      610.00
   2      -63.00      656.00
   3      -12.00      656.00
   4      -12.00      590.00
   5       15.00      575.00
```

----- Profile Line No. 3 - Material Type (Number): 3 -----

Description: Drain

Point	X	Y
1	-12.00	656.00
2	-7.00	656.00
3	-7.00	593.00
4	18.00	580.00
5	108.00	580.00
6	227.00	540.00
7	450.00	540.00
8	460.00	535.00

----- Profile Line No. 4 - Material Type (Number): 1 -----

Description: Rockfill - B

Point	X	Y
1	-7.00	656.00
2	120.00	656.00
3	251.00	586.00
4	330.00	586.00
5	408.00	550.00
6	435.00	550.00
7	450.00	540.00

----- Profile Line No. 5 - Material Type (Number): 1 -----

Description: Rockfill - C

Point	X	Y
1	-25.00	575.00
2	15.00	575.00
3	106.00	575.00
4	225.00	535.00

----- Profile Line No. 6 - Material Type (Number): 2 -----

Description: Embankment - B

Point	X	Y
1	-340.00	540.00
2	-238.00	580.00
3	-150.00	625.00
4	-125.00	625.00
5	-100.00	610.00
6	-37.00	580.00
7	-25.00	575.00
8	90.00	535.00

----- Profile Line No. 7 - Material Type (Number): 4 -----

Description: Foundation Soils - A

```
Point        X           Y

   1     -525.00      540.00
   2     -340.00      540.00
   3     -156.00      538.00
   4     -140.00      509.00

-----------------------------------------------------------
----- Profile Line No. 8 - Material Type (Number): 4 -----
-----------------------------------------------------------
Description: Foundation Soils - B

Point        X           Y

   1     -130.00      509.00
   2     -116.00      537.00
   3       90.00      535.00
   4      225.00      535.00
   5      560.00      535.00

-----------------------------------------------------------
----- Profile Line No. 9 - Material Type (Number): 5 -----
-----------------------------------------------------------
Description: Bedrock - C

Point        X           Y

   1     -525.00      512.00
   2     -140.00      509.00
   3     -130.00      509.00
   4      560.00      503.00
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BIG SANDY DAM
Max Cross-section - Steady State Seepage
Circular slip surface;
Filename: BigSandy_EL656_SS_circular.dat
PROJECT NO. 13815153

TABLE NO. 4
**************************************************************************
* NEW MATERIAL PROPERTY DATA - CONVENTIONAL/FIRST-STAGE COMPUTATIONS *
**************************************************************************

----------------------------------------------------------------------
------------------- DATA FOR MATERIAL NUMBER 1 -------------------
----------------------------------------------------------------------
Description: Rockfill (saturated)

Constant unit weight of soil (material): 110.0

CONVENTIONAL (ISOTROPIC) SHEAR STRENGTHS
Cohesion - - - - - - - - 0.0
Friction angle - - - - - 32.00 (degrees)

Pore water pressures are defined by a piezometric line.
```

Piezometric line number: 1
Negative pore water pressures are NOT allowed - set to zero.

```
-------------------------------------------------------------------
------------------- DATA FOR MATERIAL NUMBER 2 -------------------
-------------------------------------------------------------------
```

Description: Embankment (saturated)

Constant unit weight of soil (material): 130.0

CONVENTIONAL (ISOTROPIC) SHEAR STRENGTHS
Cohesion - - - - - - - - - 0.0
Friction angle - - - - - 25.00 (degrees)

Pore water pressures are defined by a piezometric line.
Piezometric line number: 1
Negative pore water pressures are NOT allowed - set to zero.

```
-------------------------------------------------------------------
------------------- DATA FOR MATERIAL NUMBER 3 -------------------
-------------------------------------------------------------------
```

Description: Drain (saturated)

Constant unit weight of soil (material): 70.0

CONVENTIONAL (ISOTROPIC) SHEAR STRENGTHS
Cohesion - - - - - - - - - 0.0
Friction angle - - - - - 38.00 (degrees)

Pore water pressures are defined by a piezometric line.
Piezometric line number: 1
Negative pore water pressures are NOT allowed - set to zero.

```
-------------------------------------------------------------------
------------------- DATA FOR MATERIAL NUMBER 4 -------------------
-------------------------------------------------------------------
```

Description: Foundation Soils (saturated)

Constant unit weight of soil (material): 135.0

CONVENTIONAL (ISOTROPIC) SHEAR STRENGTHS
Cohesion - - - - - - - - - 0.0
Friction angle - - - - - 25.00 (degrees)

Pore water pressures are defined by a piezometric line.
Piezometric line number: 1
Negative pore water pressures are NOT allowed - set to zero.

```
-------------------------------------------------------------------
------------------- DATA FOR MATERIAL NUMBER 5 -------------------
-------------------------------------------------------------------
```

Description: Bedrock (saturated)

Constant unit weight of soil (material): 150.0

CONVENTIONAL (ISOTROPIC) SHEAR STRENGTHS
Cohesion - - - - - - - - - 8000.0
Friction angle - - - - - 0.00 (degrees)

Pore water pressures are defined by a piezometric line.
Piezometric line number: 1
Negative pore water pressures are NOT allowed - set to zero.
♀

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BIG SANDY DAM
Max Cross-section - Steady State Seepage
Circular slip surface;
Filename: BigSandy_EL656_SS_circular.dat
PROJECT NO. 13815153

TABLE NO. 6
****
* NEW PIEZOMETRIC LINE DATA - CONVENTIONAL/FIRST-STAGE COMPUTATIONS *
****

--------------- Piezometric Line Number 1 ---------------

Description: Piezometric Line - Maximum Section
Unit weight of fluid (water): 62.4

Point	X	Y
1	-525.00	655.50
2	-12.00	655.50
3	-7.00	630.00
4	-7.00	593.00
5	18.00	580.00
6	108.00	580.00
7	227.00	540.00
8	450.00	540.00
9	460.00	535.00
10	560.00	535.00

♀
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Time and date of run: Tue Dec 04 11:11:53 2012
Name of input data file: G:\Projects\AEP Big Sandy\10.0_Calculations_Analysis_Data\10.03_Geotechnical\Stability\UTEXAS4\Steady State\BigSandy_EL656_SS_circular.txt

BIG SANDY DAM
Max Cross-section - Steady State Seepage
Circular slip surface;
Filename: BigSandy_EL656_SS_circular.dat
PROJECT NO. 13815153

TABLE NO. 11
****
* NEW DISTRIBUTED LOAD DATA - CONVENTIONAL/FIRST-STAGE COMPUTATIONS *
****

Point	X	Y	Normal Pressure	Shear Stress

Distributed loads will be generated from piezometric line number 1
See Output Table number 27

♀
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BIG SANDY DAM
Max Cross-section - Steady State Seepage
Circular slip surface;
Filename: BigSandy_EL656_SS_circular.dat
PROJECT NO. 13815153

TABLE NO. 16

```
*********************************
* NEW ANALYSIS/COMPUTATION DATA *
*********************************

Starting Center Coordinate for Search at -
                                             X: 350.00
                                             Y: 875.00

Required accuracy for critical center
(= minimum spacing between grid points): 1.000

No center allowed to pass below: 700.00

Critical shear surface not allowed to pass below Y: 490.00
For the initial mode of search circles are tangent to horizontal line at -
                                             Y: 510.00
                                             Radius: 365.00


Depth of crack: 1.000
Iteration limit: 100
Will save the following number of shear surfaces with the lowest factors of
safety:10

--------------------------------------------------------------------------------
The following represent default values or values that were prevously defined:
Subtended angle for slice subdivision: 3.00(degrees)
There is no water in a crack.
Conventional (single-stage) computations will be performed.
Seismic coefficient: 0.000
Unit weight of water (or other fluid) in crack: 62.4
Automatic search output will be in long form.
Search will be continued after the initial mode to find a most critical circle.
Maximum number of trial grids for a given search mode: 50
No restrictions exist on the lateral extent of the search.
Neither slope face was explicitly designated for analysis.
Standard sign convention used for direction of shear stress on shear surface.
Procedure of Analysis: Spencer

Force imbalance: 1.000000e-005 (fraction of total weight)
Moment imbalance: 1.000000e-005 (fraction of moment due to total weight)
Minimum weight required for computations to be performed: 100
Initial trial factor of safety: 3.000
Initial trial side force inclination: 17.189 (degrees)
Minimum (most negative) side force inclination allowed in Spencer's procedure:
-10.00
```

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BIG SANDY DAM
Max Cross-section - Steady State Seepage
Circular slip surface;
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TABLE NO. 26
****
* NEW, COMPUTED SLOPE GEOMETRY DATA *
****

These slope geometry were generated from the Profile Lines.

Point	X	Y
1	-525.00	540.00
2	-425.00	540.00
3	-360.00	587.00
4	-340.00	587.00
5	-275.00	587.00
6	-238.00	615.00
7	-156.00	654.55
8	-153.00	656.00
9	-150.00	656.00
10	-140.00	656.00
11	-130.00	656.00
12	-125.00	656.00
13	-116.00	656.00
14	-100.00	656.00
15	-63.00	656.00
16	-37.00	656.00
17	-25.00	656.00
18	-12.00	656.00
19	-7.00	656.00
20	15.00	656.00
21	18.00	656.00
22	90.00	656.00
23	106.00	656.00
24	108.00	656.00
25	120.00	656.00
26	225.00	599.89
27	227.00	598.82
28	251.00	586.00
29	330.00	586.00
30	408.00	550.00
31	435.00	550.00
32	450.00	540.00
33	460.00	535.00
34	560.00	535.00

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BIG SANDY DAM
Max Cross-section - Steady State Seepage
Circular slip surface;

Filename: BigSandy_EL656_SS_circular.dat
PROJECT NO. 13815153

TABLE NO. 27
****
* NEW DISTRIBUTED LOAD DATA - CONVENTIONAL/FIRST-STAGE COMPUTATIONS *
****

Point	X	Y	Normal Pressure	Shear Stress
1	-525.00	540.00	7207.2	0.0
2	-425.00	540.00	7207.2	0.0
3	-360.00	587.00	4274.4	0.0
4	-340.00	587.00	4274.4	0.0
5	-275.00	587.00	4274.4	0.0
6	-238.00	615.00	2527.2	0.0
7	-156.00	654.55	59.1	0.0
8	-154.04	655.50	0.0	0.0

The above data were generated automatically from piezometric line number 1.

Search will be conducted for RIGHT face of slope
♀
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BIG SANDY DAM
Max Cross-section - Steady State Seepage
Circular slip surface;
Filename: BigSandy_EL656_SS_circular.dat
PROJECT NO. 13815153

TABLE NO. 30
****
* OUTPUT FOR TYPE 1 AUTOMATIC SEARCH WITH CIRCLES *
****

------ Output for Circles Tangent to a Given Horizontal Line ------
------ Tangent line elevation, Y: 510.00

Center Coordinates X	Center Coordinates Y	Radius	1-Stage Factor of Safety	Side Force Inclination (degrees)	Iterations	Messages
320.00	845.00	335.00	1.849	-14.285	5	
350.00	845.00	335.00	1.804	-15.936	5	
380.00	845.00	335.00	1.748	-16.164	5	
320.00	875.00	365.00	1.850	-13.931	5	
350.00	875.00	365.00	1.802	-15.432	5	
380.00	875.00	365.00	1.740	-15.964	5	
320.00	905.00	395.00	1.854	-13.610	5	
350.00	905.00	395.00	1.813	-14.941	5	
380.00	905.00	395.00	1.750	-15.609	5	
- - - - - - New 9-Point Grid (only new points calculated) - - - - - -						
410.00	845.00	335.00	1.860	-13.807	5	
410.00	875.00	365.00	1.802	-14.775	5	
410.00	905.00	395.00	1.769	-15.121	5	

```
- - - - - - New 9-Point Grid (only new points calculated) - - - - - -
    375.00    870.00    360.00   1.742    -16.017         5
    380.00    870.00    360.00   1.740    -16.015         5
    385.00    870.00    360.00   1.740    -15.964         5
    375.00    875.00    365.00   1.743    -15.959         5
    385.00    875.00    365.00   1.739    -15.930         5
    375.00    880.00    370.00   1.744    -15.899         5
    380.00    880.00    370.00   1.741    -15.909         5
    385.00    880.00    370.00   1.739    -15.886         5
- - - - - - New 9-Point Grid (only new points calculated) - - - - - -
    390.00    875.00    365.00   1.742    -15.843         5
    390.00    880.00    370.00   1.741    -15.816         5
    380.00    885.00    375.00   1.742    -15.852         5
    385.00    885.00    375.00   1.740    -15.838         5
    390.00    885.00    375.00   1.741    -15.785         5
- - - - - - New 9-Point Grid (only new points calculated) - - - - - -
    382.00    877.00    367.00   1.739    -15.934         5
    385.00    877.00    367.00   1.739    -15.913         5
    388.00    877.00    367.00   1.740    -15.871         5
    382.00    880.00    370.00   1.740    -15.903         5
    388.00    880.00    370.00   1.740    -15.852         5
    382.00    883.00    373.00   1.740    -15.871         5
    385.00    883.00    373.00   1.740    -15.858         5
    388.00    883.00    373.00   1.740    -15.828         5
- - - - - - New 9-Point Grid (only new points calculated) - - - - - -
    382.00    874.00    364.00   1.739    -15.963         5
    385.00    874.00    364.00   1.740    -15.937         5
    388.00    874.00    364.00   1.741    -15.888         5
- - - - - - New 9-Point Grid (only new points calculated) - - - - - -
    379.00    871.00    361.00   1.740    -16.009         5
    382.00    871.00    361.00   1.739    -15.991         5
    385.00    871.00    361.00   1.740    -15.957         5
    379.00    874.00    364.00   1.740    -15.977         5
    379.00    877.00    367.00   1.741    -15.944         5
- - - - - - New 9-Point Grid (only new points calculated) - - - - - -
    381.00    873.00    363.00   1.739    -15.980         5
    382.00    873.00    363.00   1.739    -15.973         5
    383.00    873.00    363.00   1.739    -15.966         5
    381.00    874.00    364.00   1.739    -15.970         5
    383.00    874.00    364.00   1.739    -15.957         5
    381.00    875.00    365.00   1.740    -15.960         5
    382.00    875.00    365.00   1.739    -15.954         5
    383.00    875.00    365.00   1.739    -15.948         5
- - - - - - New 9-Point Grid (only new points calculated) - - - - - -
    384.00    874.00    364.00   1.739    -15.948         5
    384.00    875.00    365.00   1.739    -15.939         5
    382.00    876.00    366.00   1.739    -15.944         5
    383.00    876.00    366.00   1.739    -15.938         5
    384.00    876.00    366.00   1.739    -15.931         5
----- Critical Circle After the Current Mode of Search -----
X: 383.00     Y: 875.00     Radius: 365.000
Factor of safety: 1.739     Side force inclination: -15.948
♀
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Sandy\10.0_Calculations_Analysis_Data\10.03_Geotechnical\Stability\UTEXAS4\Steady
State\BigSandy_EL656_SS_circular.txt

BIG SANDY DAM
Max Cross-section - Steady State Seepage
Circular slip surface;
```

Filename: BigSandy_EL656_SS_circular.dat
PROJECT NO. 13815153

TABLE NO. 31
```
**************************************************
* OUTPUT FOR TYPE 1 AUTOMATIC SEARCH WITH CIRCLES *
**************************************************

------ Output for Circles with a Given, Constant Radius ------
------ Radius: 365.00

                                  1-Stage
Center Coordinates                Factor   Side Force
                                    of     Inclination
     X          Y        Radius   Safety    (degrees)   Iterations          Messages

    353.00    845.00    365.00 Center rejected as follows:      UTEXAS WARNING
NUMBER 8080
                                                                Circle passes below
the limiting depth of: 490.000

    383.00    845.00    365.00 Center rejected as follows:      UTEXAS NOTICE NUMBER
8060
                                                                Circle does not
intersect the slope.

    413.00    845.00    365.00 Center rejected as follows:      UTEXAS NOTICE NUMBER
8060
                                                                Circle does not
intersect the slope.

    353.00    875.00    365.00   1.802    -15.571         5
    413.00    875.00    365.00   1.819    -14.453         5
    353.00    905.00    365.00   2.159    -17.350         4
    383.00    905.00    365.00   2.254    -18.001         4
    413.00    905.00    365.00   2.620    -14.874         4
- - - - - - New 9-Point Grid (only new points calculated) - - - - - -
    378.00    870.00    365.00   1.760    -15.165         5
    383.00    870.00    365.00   1.744    -15.078         5
    388.00    870.00    365.00   1.747    -14.999         5
    378.00    875.00    365.00   1.741    -15.967         5
    388.00    875.00    365.00   1.741    -15.883         5
    378.00    880.00    365.00   1.746    -16.854         5
    383.00    880.00    365.00   1.745    -16.880         5
    388.00    880.00    365.00   1.748    -16.849         5
- - - - - - New 9-Point Grid (only new points calculated) - - - - - -
    380.00    872.00    365.00   1.742    -15.441         5
    383.00    872.00    365.00   1.742    -15.419         5
    386.00    872.00    365.00   1.742    -15.381         5
    380.00    875.00    365.00   1.740    -15.964         5
    386.00    875.00    365.00   1.740    -15.916         5
    380.00    878.00    365.00   1.741    -16.506         5
    383.00    878.00    365.00   1.741    -16.505         5
    386.00    878.00    365.00   1.741    -16.479         5
- - - - - - New 9-Point Grid (only new points calculated) - - - - - -
    382.00    874.00    365.00   1.740    -15.777         5
    383.00    874.00    365.00   1.740    -15.769         5
    384.00    874.00    365.00   1.740    -15.760         5
    382.00    875.00    365.00   1.739    -15.954         5
    384.00    875.00    365.00   1.739    -15.939         5
    382.00    876.00    365.00   1.739    -16.137         5
    383.00    876.00    365.00   1.739    -16.131         5
    384.00    876.00    365.00   1.739    -16.122         5
```

```
- - - - - - New 9-Point Grid (only new points calculated) - - - - - -
    382.00    877.00    365.00   1.740    -16.321         5
    383.00    877.00    365.00   1.740    -16.317         5
    384.00    877.00    365.00   1.740    -16.310         5
----- Critical Circle After the Current Mode of Search -----
X: 383.00     Y: 876.00     Radius: 365.000
Factor of safety: 1.739     Side force inclination: -16.131
♀
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State\BigSandy_EL656_SS_circular.txt

BIG SANDY DAM
Max Cross-section - Steady State Seepage
Circular slip surface;
Filename: BigSandy_EL656_SS_circular.dat
PROJECT NO. 13815153

TABLE NO. 33
*********************************************
* 1-STAGE FINAL CRITICAL CIRCLE INFORMATION *
*********************************************
X Coordinate of Center . . . . . . . . . . . . .  383.00
Y Coordinate of Center . . . . . . . . . . . . .  876.00
Radius . . . . . . . . . . . . . . . . . . . . .  365.00
Factor of Safety . . . . . . . . . . . . . . . . 1.739
Side Force Inclination (degrees) . . . . . . . . -16.13
Number of Circles Tried  . . . . . . . . . . . .  89
Number of Circles F Calculated for . . . . . . .  86
Time Required for Search (seconds) . . . . . . .  0.1

TABLE NO. 34
***************************************************************
* Summary of the 10 Circles with the Lowest Factors of Safety *
***************************************************************
```

Center Coordinates X	Y	Radius	Elevation of Bottom of Circle	Factor of Safety	Side Force Inclin.	X-Left	X-Right
383.00	876.00	365.00	511.00	1.739	-16.13	92.51	513.17
384.00	876.00	365.00	511.00	1.739	-16.12	93.51	514.17
382.00	876.00	365.00	511.00	1.739	-16.14	91.51	512.17
383.00	875.00	365.00	510.00	1.739	-15.95	91.75	515.76
383.00	876.00	366.00	510.00	1.739	-15.94	91.26	515.95
384.00	876.00	366.00	510.00	1.739	-15.93	92.26	516.95
383.00	874.00	364.00	510.00	1.739	-15.96	92.25	515.57
384.00	875.00	365.00	510.00	1.739	-15.94	92.75	516.76
383.00	873.00	363.00	510.00	1.739	-15.97	92.75	515.38
382.00	874.00	364.00	510.00	1.739	-15.96	91.25	514.57

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BIG SANDY DAM
Max Cross-section - Steady State Seepage
Circular slip surface;
Filename: BigSandy_EL656_SS_circular.dat
PROJECT NO. 13815153

TABLE NO. 43

```
**********************************************************
* Coordinate, Weight, Strength and Pore Water Pressure     *
* Information for Individual Slices for Conventional       *
* Computations or First Stage of Multi-Stage Computations. *
* (Information is for the critical shear surface in the    *
* case of an automatic search.)                            *
**********************************************************
```

Slice No.	X	Y	Slice Weight	Matl. No.	Cohesion	Friction Angle	Pore Pressure
	92.51	655.00					
1	98.49	647.55	11121	1	0.0	32.00	0.0
	104.48	640.10					
2	105.24	639.21	2817	1	0.0	32.00	0.0
	106.00	638.31					
3	107.00	637.16	4146	1	0.0	32.00	0.0
	108.00	636.00					
4	114.00	629.45	35041	1	0.0	32.00	0.0
	120.00	622.91					
5	126.80	616.20	54130	1	0.0	32.00	0.0
	133.61	609.49					
6	140.75	603.15	65650	1	0.0	32.00	0.0
	147.90	596.80					
7	155.36	590.84	75998	1	0.0	32.00	0.0
	162.83	584.88					
8	170.60	579.32	84848	1	0.0	32.00	0.0
	178.37	573.76					
9	186.42	568.61	91908	1	0.0	32.00	0.0
	194.47	563.46					
10	202.77	558.74	96917	1	0.0	32.00	0.0
	211.08	554.02					
11	218.04	550.50	81318	1	0.0	32.00	0.0
	225.00	546.97					
12	226.00	546.49	11630	1	0.0	32.00	0.0
	227.00	546.02					
13	233.71	543.01	77104	1	0.0	32.00	0.0
	240.42	540.00					
14	245.71	537.85	58414	3	0.0	38.00	134.0
	251.00	535.70					
15	251.92	535.35	9860	3	0.0	38.00	290.0
	252.83	535.00					
16	261.84	531.83	105235	4	0.0	25.00	510.0
	270.86	528.65					
17	280.02	525.96	121555	4	0.0	25.00	876.2
	289.19	523.26					
18	298.48	521.05	135581	4	0.0	25.00	1182.6
	307.78	518.84					
19	317.18	517.11	147075	4	0.0	25.00	1428.2

```
        326.57    515.39
  20    328.29    515.13         27731    4          0.0    25.00      1552.0
        330.00    514.87
  21    339.49    513.73        147987    4          0.0    25.00      1639.3
        348.97    512.59
  22    358.51    511.95        134888    4          0.0    25.00      1750.4
        368.04    511.31
  23    375.52    511.15         94529    4          0.0    25.00      1800.0
        383.00    511.00
  24    392.55    511.25        103929    4          0.0    25.00      1794.0
        402.10    511.50
  25    405.05    511.68         28001    4          0.0    25.00      1767.2
        408.00    511.86
  26    417.51    512.76         84698    4          0.0    25.00      1699.7
        427.02    513.66
  27    431.01    514.19         33971    4          0.0    25.00      1610.3
        435.00    514.72
  28    442.50    515.96         52051    4          0.0    25.00      1499.9
        450.00    517.20
  29    455.00    518.21         24419    4          0.0    25.00      1203.8
        460.00    519.21
  30    469.28    521.47         33904    4          0.0    25.00       844.0
        478.57    523.73
  31    487.72    526.48         21066    4          0.0    25.00       531.9
        496.87    529.22
  32    505.02    532.11          6361    4          0.0    25.00       180.4
        513.17    535.00
```

No water in crack.
♀
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BIG SANDY DAM
Max Cross-section - Steady State Seepage
Circular slip surface;
Filename: BigSandy_EL656_SS_circular.dat
PROJECT NO. 13815153

TABLE NO. 44

```
**********************************************************
* Seismic Forces and Forces Due to Distributed Loads for *
* Individual Slices for Conventional Computations or the *
* First Stage of Multi-Stage Computations.               *
* (Information is for the critical shear surface in the  *
* case of an automatic search.)                          *
**********************************************************
```

There are no seismic forces or forces due to distributed loads
for the current shear surface
♀
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BIG SANDY DAM

Max Cross-section - Steady State Seepage
Circular slip surface;
Filename: BigSandy_EL656_SS_circular.dat
PROJECT NO. 13815153

TABLE NO. 47
```
***************************************************************
*  Information for the Iterative Solution for the Factor of  *
*  Safety and Side Force Inclination by Spencer's Procedure  *
***************************************************************
Allowable force imbalance for convergence:  21
Allowable moment imbalance for convergence: 5978

        Trial     Trial
       Factor   Side Force     Force       Moment                  Delta
Iter-    of    Inclination   Imbalance   Imbalance    Delta-F      Theta
ation  Safety   (degrees)      (lbs.)    (ft.-lbs.)              (degrees)

   1   3.00000   -17.1887  -2.270e+005   1.354e+008
First-order corrections to F and Theta ............   -2.1727     1.9287
Reduced values - Deltas were too large ............   -0.5000     0.4439

   2   2.50000   -16.7449  -1.643e+005   9.799e+007
First-order corrections to F and Theta ............   -1.0915     0.9429
Reduced values - Deltas were too large ............   -0.5000     0.4319

   3   2.00000   -16.3130  -7.035e+004   4.197e+007
First-order corrections to F and Theta ............   -0.2997     0.2175
Second-order corrections to F and Theta ...........   -0.2649     0.1864

   4   1.73513   -16.1265   1.230e+003  -7.328e+005
First-order corrections to F and Theta ............    0.0040    -0.0040
Second-order corrections to F and Theta ...........    0.0040    -0.0040

   5   1.73911   -16.1305  -6.306e-003   3.747e+000
First-order corrections to F and Theta ............   -0.0000     0.0000
```
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Time and date of run: Tue Dec 04 11:11:53 2012
Name of input data file: G:\Projects\AEP Big Sandy\10.0_Calculations_Analysis_Data\10.03_Geotechnical\Stability\UTEXAS4\Steady State\BigSandy_EL656_SS_circular.txt

BIG SANDY DAM
Max Cross-section - Steady State Seepage
Circular slip surface;
Filename: BigSandy_EL656_SS_circular.dat
PROJECT NO. 13815153

TABLE NO. 55
```
**********************************************************************
* Check of Computations by Spencer's Procedure (Results are for the *
* critical shear surface in the case of an automatic search.)       *
**********************************************************************
```

Summation of Horizontal Forces: 1.52454e-010

Summation of Vertical Forces: 1.39153e-010

Summation of Moments: 2.98023e-008

Mohr Coulomb Shear Force/Shear Strength Check Summation: 7.09406e-011

♀
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BIG SANDY DAM
Max Cross-section - Steady State Seepage
Circular slip surface;
Filename: BigSandy_EL656_SS_circular.dat
PROJECT NO. 13815153

TABLE NO. 58
****
* Final Results for Stresses Along the Shear Surface *
* (Results are for the critical shear surface in the case of a search.) *
****

SPENCER'S PROCEDURE USED TO COMPUTE THE FACTOR OF SAFETY
Factor of Safety: 1.739   Side Force Inclination: -16.13

-------- VALUES AT CENTER OF BASE OF SLICE --------

Slice No.	X-Center	Y-Center	Total Normal Stress	Effective Normal Stress	Shear Stress
1	98.49	647.55	545.6	545.6	196.0
2	105.24	639.21	1115.0	1115.0	400.6
3	107.00	637.16	1259.7	1259.7	452.6
4	114.00	629.45	1820.9	1820.9	654.3
5	126.80	616.20	2590.7	2590.7	930.9
6	140.75	603.15	3121.4	3121.4	1121.5
7	155.36	590.84	3599.4	3599.4	1293.3
8	170.60	579.32	4014.2	4014.2	1442.3
9	186.42	568.61	4355.3	4355.3	1564.9
10	202.77	558.74	4612.9	4612.9	1657.4
11	218.04	550.50	4770.6	4770.6	1714.1
12	226.00	546.49	4825.6	4825.6	1733.9
13	233.71	543.01	4840.9	4840.9	1739.4
14	245.71	537.85	4725.5	4591.5	2062.7
15	251.92	535.35	4676.5	4386.5	1970.6
16	261.84	531.83	5226.7	4716.8	1264.7
17	280.02	525.96	6104.3	5228.1	1401.8
18	298.48	521.05	6896.8	5714.2	1532.2
19	317.18	517.11	7596.9	6168.7	1654.0
20	328.29	515.13	7979.9	6427.9	1723.5
21	339.49	513.73	7808.4	6169.1	1654.1
22	358.51	511.95	7261.4	5511.0	1477.7
23	375.52	511.15	6626.8	4826.7	1294.2
24	392.55	511.25	5825.6	4031.6	1081.0
25	405.05	511.68	5156.1	3388.9	908.7
26	417.51	512.76	4919.1	3219.4	863.2
27	431.01	514.19	4804.6	3194.3	856.5
28	442.50	515.96	3958.9	2459.0	659.3
29	455.00	518.21	2818.2	1614.4	432.9
30	469.28	521.47	2167.6	1323.6	354.9
31	487.72	526.48	1412.2	880.2	236.0
32	505.02	532.11	495.6	315.2	84.5

♀
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Name of input data file: G:\Projects\AEP Big Sandy\10.0_Calculations_Analysis_Data\10.03_Geotechnical\Stability\UTEXAS4\Steady State\BigSandy_EL656_SS_circular.txt

BIG SANDY DAM
Max Cross-section - Steady State Seepage
Circular slip surface;
Filename: BigSandy_EL656_SS_circular.dat
PROJECT NO. 13815153

TABLE NO. 59
****
* Final Results for Side Forces and Stresses Between Slices *
* (Results are for the critical shear surface in the case of a search.) *
****

--------------- VALUES AT RIGHT SIDE OF SLICE ---------------

Slice No.	X-Right	Side Force	Y-Coord. of Side Force Location	Fraction of Height	Sigma at Top	Sigma at Bottom
1	104.48	6021	645.82	0.360	57.6	669.9
2	106.00	7461	644.14	0.330	-8.6	819.0
3	108.00	9549	642.10	0.305	-77.8	995.2
4	120.00	26194	631.70	0.266	-309.0	1829.8
5	133.61	49195	621.44	0.304	-208.9	2617.6
6	147.90	73736	611.90	0.341	73.0	3125.5
7	162.83	98300	602.86	0.373	463.3	3452.3
8	178.37	121454	594.31	0.403	948.7	3621.9
9	194.47	141906	586.28	0.433	1540.2	3628.6
10	211.08	158563	578.84	0.465	2265.8	3449.0
11	225.00	168756	573.22	0.496	2990.3	3136.0
12	227.00	169933	572.46	0.501	3105.1	3077.4
13	240.42	175954	567.64	0.535	3960.6	2584.1
14	251.00	174367	564.83	0.579	4912.6	1748.1
15	252.83	174041	564.36	0.576	4766.9	1789.5
16	270.86	184834	557.40	0.501	3119.6	3073.0
17	289.19	192353	550.97	0.442	1914.7	3975.8
18	307.78	194482	545.30	0.394	1012.3	4550.9
19	326.57	189382	540.55	0.356	355.1	4797.7
20	330.00	187550	539.80	0.350	260.8	4804.9
21	348.97	173404	536.22	0.365	496.1	4656.8
22	368.04	153773	533.45	0.388	840.3	4330.4
23	383.00	135733	531.80	0.411	1209.9	3950.1
24	402.10	111203	530.19	0.454	1868.5	3314.5
25	408.00	103709	529.77	0.469	2133.9	3090.0
26	427.02	77359	529.12	0.425	1128.6	2961.9
27	435.00	64954	529.44	0.417	890.3	2647.3
28	450.00	44442	530.32	0.576	2721.2	1024.1
29	460.00	34033	530.69	0.727	4894.5	-752.4
30	478.57	16977	531.89	0.724	3392.5	-497.6
31	496.87	4417	534.47	0.908	2528.1	-1060.5
32	513.17	-0	535.00	0.000	0.0	0.0

Read end-of-file on input while looking for another command word.
End of input data assumed - normal termination.

TABLE NO. 1
COMPUTER PROGRAM DESIGNATION: UTEXAS4
Originally Coded By Stephen G. Wright
Version No. 4.1.0.8 - Last Revision Date: 11/9/2009


```
*****************************************************************
* RESULTS OF COMPUTATIONS PERFORMED USING THIS SOFTWARE         *
* SHOULD NOT BE USED FOR DESIGN PURPOSES UNLESS THEY HAVE       *
* BEEN VERIFIED BY INDEPENDENT ANALYSES, EXPERIMENTAL DATA      *
* OR FIELD EXPERIENCE. THE USER SHOULD UNDERSTAND THE ALGORITHMS *
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*****************************************************************
```

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Name of input data file: G:\Projects\AEP Big Sandy\10.0_Calculations_Analysis_Data\10.03_Geotechnical\Stability\UTEXAS4\Earthquake\BigSandy_EL656_EQ_circular.txt

```
PROJECT:        BIG SANDY DAM
PROJECT NO:     13815153
ANALYSIS:       Max Cross-section - Seismic (pseudo-static) Analysis
                PGA=0.07g
BY:             C.Helm
Filename:       BigSandy_EL656_EQ_circular.dat
```

TABLE NO. 3

```
*************************
* NEW PROFILE LINE DATA *
*************************
```

```
----------------------------------------------------------
----- Profile Line No. 1 - Material Type (Number): 1 -----
----------------------------------------------------------
```

Description: Rockfill - A

Point	X	Y
1	-425.00	540.00
2	-360.00	587.00
3	-275.00	587.00
4	-238.00	615.00
5	-153.00	656.00
6	-63.00	656.00

```
----------------------------------------------------------
----- Profile Line No. 2 - Material Type (Number): 2 -----
----------------------------------------------------------
```

Description: Embankment - A

Point	X	Y
1	-100.00	610.00
2	-63.00	656.00
3	-12.00	656.00
4	-12.00	590.00
5	15.00	575.00

----- Profile Line No. 3 - Material Type (Number): 3 -----

Description: Drain

Point	X	Y
1	-12.00	656.00
2	-7.00	656.00
3	-7.00	593.00
4	18.00	580.00
5	108.00	580.00
6	227.00	540.00
7	450.00	540.00
8	460.00	535.00

----- Profile Line No. 4 - Material Type (Number): 1 -----

Description: Rockfill - B

Point	X	Y
1	-7.00	656.00
2	120.00	656.00
3	251.00	586.00
4	330.00	586.00
5	408.00	550.00
6	435.00	550.00
7	450.00	540.00

----- Profile Line No. 5 - Material Type (Number): 1 -----

Description: Rockfill - C

Point	X	Y
1	-25.00	575.00
2	15.00	575.00
3	106.00	575.00
4	225.00	535.00

----- Profile Line No. 6 - Material Type (Number): 2 -----

Description: Embankment - B

Point	X	Y
1	-340.00	540.00
2	-238.00	580.00
3	-150.00	625.00
4	-125.00	625.00
5	-100.00	610.00
6	-37.00	580.00
7	-25.00	575.00
8	90.00	535.00

----- Profile Line No. 7 - Material Type (Number): 4 -----

```
Description: Foundation Soils - A

Point         X           Y

   1     -525.00      540.00
   2     -340.00      540.00
   3     -156.00      538.00
   4     -140.00      509.00

-----------------------------------------------------------
----- Profile Line No. 8 - Material Type (Number): 4 -----
-----------------------------------------------------------
Description: Foundation Soils - B

Point         X           Y

   1     -130.00      509.00
   2     -116.00      537.00
   3       90.00      535.00
   4      225.00      535.00
   5      560.00      535.00

-----------------------------------------------------------
----- Profile Line No. 9 - Material Type (Number): 5 -----
-----------------------------------------------------------
Description: Bedrock - C

Point         X           Y

   1     -525.00      512.00
   2     -140.00      509.00
   3     -130.00      509.00
   4      560.00      503.00
♀
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Time and date of run: Tue Dec 04 16:48:11 2012
Name of input data file: G:\Projects\AEP Big
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e\BigSandy_EL656_EQ_circular.txt

PROJECT:        BIG SANDY DAM
PROJECT NO:     13815153
ANALYSIS:       Max Cross-section - Seismic (pseudo-static) Analysis
                PGA=0.07g
BY:             C.Helm
Filename:       BigSandy_EL656_EQ_circular.dat

TABLE NO. 4
**********************************************************************
* NEW MATERIAL PROPERTY DATA - CONVENTIONAL/FIRST-STAGE COMPUTATIONS *
**********************************************************************

------------------------------------------------------------------
------------------- DATA FOR MATERIAL NUMBER 1 -------------------
------------------------------------------------------------------
Description: Rockfill (saturated)

Constant unit weight of soil (material): 110.0

CONVENTIONAL (ISOTROPIC) SHEAR STRENGTHS
Cohesion - - - - - - - - 0.0
Friction angle - - - - - 32.00 (degrees)
```

Pore water pressures are defined by a piezometric line.
Piezometric line number: 1
Negative pore water pressures are NOT allowed - set to zero.

------------------ DATA FOR MATERIAL NUMBER 2 ------------------
----
Description: Embankment (saturated)

Constant unit weight of soil (material): 130.0

CONVENTIONAL (ISOTROPIC) SHEAR STRENGTHS
Cohesion - - - - - - - - 0.0
Friction angle - - - - - 25.00 (degrees)

Pore water pressures are defined by a piezometric line.
Piezometric line number: 1
Negative pore water pressures are NOT allowed - set to zero.

------------------ DATA FOR MATERIAL NUMBER 3 ------------------
----
Description: Drain (saturated)

Constant unit weight of soil (material): 70.0

CONVENTIONAL (ISOTROPIC) SHEAR STRENGTHS
Cohesion - - - - - - - - 0.0
Friction angle - - - - - 38.00 (degrees)

Pore water pressures are defined by a piezometric line.
Piezometric line number: 1
Negative pore water pressures are NOT allowed - set to zero.

------------------ DATA FOR MATERIAL NUMBER 4 ------------------
----
Description: Foundation Soils (saturated)

Constant unit weight of soil (material): 135.0

CONVENTIONAL (ISOTROPIC) SHEAR STRENGTHS
Cohesion - - - - - - - - 0.0
Friction angle - - - - - 25.00 (degrees)

Pore water pressures are defined by a piezometric line.
Piezometric line number: 1
Negative pore water pressures are NOT allowed - set to zero.

------------------ DATA FOR MATERIAL NUMBER 5 ------------------
----
Description: Bedrock (saturated)

Constant unit weight of soil (material): 150.0

CONVENTIONAL (ISOTROPIC) SHEAR STRENGTHS
Cohesion - - - - - - - - 8000.0
Friction angle - - - - - 0.00 (degrees)

Pore water pressures are defined by a piezometric line.
Piezometric line number: 1

Negative pore water pressures are NOT allowed - set to zero.

♀

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PROJECT: BIG SANDY DAM
PROJECT NO: 13815153
ANALYSIS: Max Cross-section - Seismic (pseudo-static) Analysis
PGA=0.07g
BY: C.Helm
Filename: BigSandy_EL656_EQ_circular.dat

TABLE NO. 6

```
*******************************************************************
* NEW PIEZOMETRIC LINE DATA - CONVENTIONAL/FIRST-STAGE COMPUTATIONS *
*******************************************************************
```

```
---------------------------------------------------------
--------------- Piezometric Line Number 1 ---------------
---------------------------------------------------------
```

Description: Piezometric Line - Maximum Section
Unit weight of fluid (water): 62.4

Point	X	Y
1	-525.00	655.50
2	-12.00	655.50
3	-7.00	630.00
4	-7.00	593.00
5	18.00	580.00
6	108.00	580.00
7	227.00	540.00
8	450.00	540.00
9	460.00	535.00
10	560.00	535.00

♀

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Name of input data file: G:\Projects\AEP Big Sandy\10.0_Calculations_Analysis_Data\10.03_Geotechnical\Stability\UTEXAS4\Earthquake\BigSandy_EL656_EQ_circular.txt

PROJECT: BIG SANDY DAM
PROJECT NO: 13815153
ANALYSIS: Max Cross-section - Seismic (pseudo-static) Analysis
PGA=0.07g
BY: C.Helm
Filename: BigSandy_EL656_EQ_circular.dat

TABLE NO. 11

```
*******************************************************************
* NEW DISTRIBUTED LOAD DATA - CONVENTIONAL/FIRST-STAGE COMPUTATIONS *
*******************************************************************
```

Point	X	Y	Normal Pressure	Shear Stress

Distributed loads will be generated from piezometric line number 1
See Output Table number 27

```
♀
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e\BigSandy_EL656_EQ_circular.txt

PROJECT:        BIG SANDY DAM
PROJECT NO:     13815153
ANALYSIS:       Max Cross-section - Seismic (pseudo-static) Analysis
                PGA=0.07g
BY:             C.Helm
Filename:       BigSandy_EL656_EQ_circular.dat

TABLE NO. 5
2222222222222222222222222222222222222222222222222222222222
2 NEW MATERIAL PROPERTY DATA - SECOND_STAGE COMPUTATIONS 2
2222222222222222222222222222222222222222222222222222222222

------------------------------------------------------------------
------------------- DATA FOR MATERIAL NUMBER 1 -------------------
------------------------------------------------------------------
Description: Rockfill (saturated)

Constant unit weight of soil (material): 110.0

CONVENTIONAL (ISOTROPIC) SHEAR STRENGTHS
Cohesion - - - - - - - - - 0.0
Friction angle - - - - - 32.00 (degrees)

Pore water pressures are defined by a piezometric line.
Piezometric line number: 2
Negative pore water pressures are NOT allowed - set to zero.

------------------------------------------------------------------
------------------- DATA FOR MATERIAL NUMBER 2 -------------------
------------------------------------------------------------------
Description: Embankment (saturated)

Constant unit weight of soil (material): 130.0

---- 2-STAGE STRENGTHS FOR SECOND STAGE OF COMPUTATIONS

Kc = 1 ENVELOPE:
     Intercept of envelope ("d") - - - - - - - - - 500.0
     Slope of envelope ("psi") - - - - - - - - - - 23.00 (degrees)
Kc = Kf ENVELOPE:
     Intercept of envelope ("d") - - - - - - - - - 0.0
     Slope of envelope ("psi") - - - - - - - - - - 25.00 (degrees)

Pore water pressures are defined by a piezometric line.
Piezometric line number: 2
Negative pore water pressures are NOT allowed - set to zero.

------------------------------------------------------------------
------------------- DATA FOR MATERIAL NUMBER 3 -------------------
------------------------------------------------------------------
Description: Drain (saturated)

Constant unit weight of soil (material): 70.0
```

```
CONVENTIONAL (ISOTROPIC) SHEAR STRENGTHS
Cohesion - - - - - - - - 0.0
Friction angle - - - - - 38.00 (degrees)

Pore water pressures are defined by a piezometric line.
Piezometric line number: 2
Negative pore water pressures are NOT allowed - set to zero.

-----------------------------------------------------------------
------------------- DATA FOR MATERIAL NUMBER 4 -------------------
-----------------------------------------------------------------
Description: Foundation Soils (saturated)

Constant unit weight of soil (material): 135.0

---- 2-STAGE STRENGTHS FOR SECOND STAGE OF COMPUTATIONS

Kc = 1 ENVELOPE:
     Intercept of envelope ("d") - - - - - - - - - 1100.0
     Slope of envelope ("psi") - - - - - - - - - - 12.00 (degrees)
Kc = Kf ENVELOPE:
     Intercept of envelope ("d") - - - - - - - - - 0.0
     Slope of envelope ("psi") - - - - - - - - - - 25.00 (degrees)

Pore water pressures are defined by a piezometric line.
Piezometric line number: 2
Negative pore water pressures are NOT allowed - set to zero.

-----------------------------------------------------------------
------------------- DATA FOR MATERIAL NUMBER 5 -------------------
-----------------------------------------------------------------
Description: Bedrock (saturated)

Constant unit weight of soil (material): 150.0

CONVENTIONAL (ISOTROPIC) SHEAR STRENGTHS
Cohesion - - - - - - - - 8000.0
Friction angle - - - - - 0.00 (degrees)

Pore water pressures are defined by a piezometric line.
Piezometric line number: 2
Negative pore water pressures are NOT allowed - set to zero.
♀
UTEXAS4 S/N:10001  - Version: 4.1.0.8 - Latest Revision: 11/9/2009
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Name of input data file: G:\Projects\AEP Big
Sandy\10.0_Calculations_Analysis_Data\10.03_Geotechnical\Stability\UTEXAS4\Earthquak
e\BigSandy_EL656_EQ_circular.txt

PROJECT:        BIG SANDY DAM
PROJECT NO:     13815153
ANALYSIS:       Max Cross-section - Seismic (pseudo-static) Analysis
                PGA=0.07g
BY:             C.Helm
Filename:       BigSandy_EL656_EQ_circular.dat

TABLE NO. 7
2222222222222222222222222222222222222222222222222222222222222222222222
2 NEW PIEZOMETRIC LINE DATA - CONVENTIONAL/FIRST-STAGE COMPUTATIONS 2
2222222222222222222222222222222222222222222222222222222222222222222222

---------------------------------------------------------
```

```
--------------- Piezometric Line Number 2 ---------------
---------------------------------------------------------
Description: Piezometric Line - Maximum Section
Unit weight of fluid (water): 62.4

Point        X           Y

  1     -525.00      655.50
  2      -12.00      655.50
  3       -7.00      630.00
  4       -7.00      593.00
  5       18.00      580.00
  6      108.00      580.00
  7      227.00      540.00
  8      450.00      540.00
  9      460.00      535.00
 10      560.00      535.00
♀
UTEXAS4 S/N:10001  - Version: 4.1.0.8 - Latest Revision: 11/9/2009
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Name of input data file: G:\Projects\AEP Big
Sandy\10.0_Calculations_Analysis_Data\10.03_Geotechnical\Stability\UTEXAS4\Earthquak
e\BigSandy_EL656_EQ_circular.txt

PROJECT:        BIG SANDY DAM
PROJECT NO:     13815153
ANALYSIS:       Max Cross-section - Seismic (pseudo-static) Analysis
                PGA=0.07g
BY:             C.Helm
Filename:       BigSandy_EL656_EQ_circular.dat

TABLE NO. 12
222222222222222222222222222222222222222222222222222222222222222
2 NEW DISTRIBUTED LOAD DATA - SECOND/THIRD-STAGE COMPUTATIONS 2
222222222222222222222222222222222222222222222222222222222222222
                                     Normal        Shear
Point       X           Y        Pressure       Stress

Distributed loads will be generated from piezometric line number 2
See Output Table number 28

♀
UTEXAS4 S/N:10001  - Version: 4.1.0.8 - Latest Revision: 11/9/2009
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Name of input data file: G:\Projects\AEP Big
Sandy\10.0_Calculations_Analysis_Data\10.03_Geotechnical\Stability\UTEXAS4\Earthquak
e\BigSandy_EL656_EQ_circular.txt

PROJECT:        BIG SANDY DAM
PROJECT NO:     13815153
ANALYSIS:       Max Cross-section - Seismic (pseudo-static) Analysis
                PGA=0.07g
BY:             C.Helm
Filename:       BigSandy_EL656_EQ_circular.dat

TABLE NO. 16
*********************************
* NEW ANALYSIS/COMPUTATION DATA *
*********************************

Starting Center Coordinate for Search at -
```

```
                                          X: 350.00
                                          Y: 875.00

Required accuracy for critical center
(= minimum spacing between grid points): 1.000

No center allowed to pass below: 700.00

Critical shear surface not allowed to pass below Y: 490.00
For the initial mode of search circles are tangent to horizontal line at -
                                          Y: 510.00
                                          Radius: 365.00

Iteration limit: 100
Will save the following number of shear surfaces with the lowest factors of
safety:10
Two-stage computations will be performed.
Seismic coefficient: 0.070
Seismic force acts at center of gravity.

-----------------------------------------------------------------------------------
The following represent default values or values that were prevously defined:
Subtended angle for slice subdivision: 3.00(degrees)
There is no crack.
There is no water in a crack.
Unit weight of water (or other fluid) in crack: 62.4
Automatic search output will be in long form.
Search will be continued after the initial mode to find a most critical circle.
Maximum number of trial grids for a given search mode: 50
No restrictions exist on the lateral extent of the search.
Neither slope face was explicitly designated for analysis.
Standard sign convention used for direction of shear stress on shear surface.
Procedure of Analysis: Spencer

Force imbalance: 1.000000e-005 (fraction of total weight)
Moment imbalance: 1.000000e-005 (fraction of moment due to total weight)
Minimum weight required for computations to be performed: 100
Initial trial factor of safety: 3.000
Initial trial side force inclination: 17.189 (degrees)
Minimum (most negative) side force inclination allowed in Spencer's procedure:
-10.00
♀
UTEXAS4 S/N:10001  - Version: 4.1.0.8 - Latest Revision: 11/9/2009
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Time and date of run: Tue Dec 04 16:48:11 2012
Name of input data file: G:\Projects\AEP Big
Sandy\10.0_Calculations_Analysis_Data\10.03_Geotechnical\Stability\UTEXAS4\Earthquak
e\BigSandy_EL656_EQ_circular.txt

PROJECT:        BIG SANDY DAM
PROJECT NO:     13815153
ANALYSIS:       Max Cross-section - Seismic (pseudo-static) Analysis
                PGA=0.07g
BY:             C.Helm
Filename:       BigSandy_EL656_EQ_circular.dat

TABLE NO. 26
*************************************
* NEW, COMPUTED SLOPE GEOMETRY DATA *
*************************************

These slope geometry were generated from the Profile Lines.
```

Point	X	Y
1	-525.00	540.00
2	-425.00	540.00
3	-360.00	587.00
4	-340.00	587.00
5	-275.00	587.00
6	-238.00	615.00
7	-156.00	654.55
8	-153.00	656.00
9	-150.00	656.00
10	-140.00	656.00
11	-130.00	656.00
12	-125.00	656.00
13	-116.00	656.00
14	-100.00	656.00
15	-63.00	656.00
16	-37.00	656.00
17	-25.00	656.00
18	-12.00	656.00
19	-7.00	656.00
20	15.00	656.00
21	18.00	656.00
22	90.00	656.00
23	106.00	656.00
24	108.00	656.00
25	120.00	656.00
26	225.00	599.89
27	227.00	598.82
28	251.00	586.00
29	330.00	586.00
30	408.00	550.00
31	435.00	550.00
32	450.00	540.00
33	460.00	535.00
34	560.00	535.00

♀
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Time and date of run: Tue Dec 04 16:48:11 2012
Name of input data file: G:\Projects\AEP Big Sandy\10.0_Calculations_Analysis_Data\10.03_Geotechnical\Stability\UTEXAS4\Earthquake\BigSandy_EL656_EQ_circular.txt

PROJECT: BIG SANDY DAM
PROJECT NO: 13815153
ANALYSIS: Max Cross-section - Seismic (pseudo-static) Analysis PGA=0.07g
BY: C.Helm
Filename: BigSandy_EL656_EQ_circular.dat

TABLE NO. 27
****
* NEW DISTRIBUTED LOAD DATA - CONVENTIONAL/FIRST-STAGE COMPUTATIONS *
****

Point	X	Y	Normal Pressure	Shear Stress
1	-525.00	540.00	7207.2	0.0
2	-425.00	540.00	7207.2	0.0
3	-360.00	587.00	4274.4	0.0
4	-340.00	587.00	4274.4	0.0
5	-275.00	587.00	4274.4	0.0

```
   6     -238.00      615.00       2527.2          0.0
   7     -156.00      654.55         59.1          0.0
   8     -154.04      655.50          0.0          0.0
```

The above data were generated automatically from piezometric
line number 1.
♀
UTEXAS4 S/N:10001 - Version: 4.1.0.8 - Latest Revision: 11/9/2009
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Time and date of run: Tue Dec 04 16:48:11 2012
Name of input data file: G:\Projects\AEP Big
Sandy\10.0_Calculations_Analysis_Data\10.03_Geotechnical\Stability\UTEXAS4\Earthquak
e\BigSandy_EL656_EQ_circular.txt

```
PROJECT:        BIG SANDY DAM
PROJECT NO:     13815153
ANALYSIS:       Max Cross-section - Seismic (pseudo-static) Analysis
                PGA=0.07g
BY:             C.Helm
Filename:       BigSandy_EL656_EQ_circular.dat
```

TABLE NO. 28

```
222222222222222222222222222222222222222222222222222222222222222
2 NEW DISTRIBUTED LOAD DATA - SECOND/THIRD-STAGE COMPUTATIONS 2
222222222222222222222222222222222222222222222222222222222222222
                                  Normal        Shear
Point       X           Y        Pressure       Stress

   1     -525.00      540.00       7207.2          0.0
   2     -425.00      540.00       7207.2          0.0
   3     -360.00      587.00       4274.4          0.0
   4     -340.00      587.00       4274.4          0.0
   5     -275.00      587.00       4274.4          0.0
   6     -238.00      615.00       2527.2          0.0
   7     -156.00      654.55         59.1          0.0
   8     -154.04      655.50          0.0          0.0
```

The above data were generated automatically from piezometric
line number 2.

Search will be conducted for RIGHT face of slope
♀
UTEXAS4 S/N:10001 - Version: 4.1.0.8 - Latest Revision: 11/9/2009
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Time and date of run: Tue Dec 04 16:48:11 2012
Name of input data file: G:\Projects\AEP Big
Sandy\10.0_Calculations_Analysis_Data\10.03_Geotechnical\Stability\UTEXAS4\Earthquak
e\BigSandy_EL656_EQ_circular.txt

```
PROJECT:        BIG SANDY DAM
PROJECT NO:     13815153
ANALYSIS:       Max Cross-section - Seismic (pseudo-static) Analysis
                PGA=0.07g
BY:             C.Helm
Filename:       BigSandy_EL656_EQ_circular.dat
```

TABLE NO. 30

```
***************************************************
* OUTPUT FOR TYPE 1 AUTOMATIC SEARCH WITH CIRCLES *
***************************************************

------ Output for Circles Tangent to a Given Horizontal Line ------
------ Tangent line elevation, Y: 510.00
```

Center Coordinates X	Y	Radius	2-Stage Factor of Safety	Side Force Inclination (degrees)	Iterations	Messages
320.00	845.00	335.00	1.377	-18.577	6	
350.00	845.00	335.00	1.371	-20.216	6	
380.00	845.00	335.00	1.336	-20.073	6	
320.00	875.00	365.00	1.371	-18.551	6	
350.00	875.00	365.00	1.365	-19.989	6	
380.00	875.00	365.00	1.333	-20.036	6	
320.00	905.00	395.00	1.368	-18.507	6	
350.00	905.00	395.00	1.367	-19.778	6	
380.00	905.00	395.00	1.340	-19.910	6	
- - - - - - New 9-Point Grid (only new points calculated) - - - - - -						
410.00	845.00	335.00	Center rejected as follows:			UTEXAS ERROR NUMBER 9280 The side force inclination fell outside the range of values allowed. The minimum value allowed is: -8.00000e+001 degrees. The maximum value allowed is: 1.00000e+001 degrees. Error occurred while computing factor of safety for 2nd stage.
410.00	875.00	365.00	Center rejected as follows:			UTEXAS ERROR NUMBER 9280 The side force inclination fell outside the range of values allowed. The minimum value allowed is: -8.00000e+001 degrees. The maximum value allowed is: 1.00000e+001 degrees. Error occurred while computing factor of safety for 2nd stage.
410.00	905.00	395.00	1.351	-18.896	6	
- - - - - - New 9-Point Grid (only new points calculated) - - - - - -						
375.00	870.00	360.00	1.334	-20.133	6	
380.00	870.00	360.00	1.332	-20.053	6	
385.00	870.00	360.00	1.332	-19.932	6	
375.00	875.00	365.00	1.334	-20.113	6	
385.00	875.00	365.00	1.332	-19.925	6	
375.00	880.00	370.00	1.335	-20.093	6	
380.00	880.00	370.00	1.333	-20.017	6	
385.00	880.00	370.00	1.332	-19.912	6	
- - - - - - New 9-Point Grid (only new points calculated) - - - - - -						
390.00	870.00	360.00	1.335	-19.770	6	
390.00	875.00	365.00	1.334	-19.771	6	
390.00	880.00	370.00	1.333	-19.768	6	
- - - - - - New 9-Point Grid (only new points calculated) - - - - - -						
382.00	872.00	362.00	1.332	-20.003	6	
385.00	872.00	362.00	1.332	-19.930	6	
388.00	872.00	362.00	1.333	-19.839	6	
382.00	875.00	365.00	1.332	-19.994	6	
388.00	875.00	365.00	1.333	-19.837	6	
382.00	878.00	368.00	1.332	-19.984	6	
385.00	878.00	368.00	1.332	-19.917	6	
388.00	878.00	368.00	1.333	-19.834	6	
- - - - - - New 9-Point Grid (only new points calculated) - - - - - -						
384.00	874.00	364.00	1.332	-19.952	6	

```
    385.00    874.00    364.00   1.332    -19.927         6
    386.00    874.00    364.00   1.332    -19.899         6
    384.00    875.00    365.00   1.332    -19.949         6
    386.00    875.00    365.00   1.332    -19.898         6
    384.00    876.00    366.00   1.332    -19.947         6
    385.00    876.00    366.00   1.332    -19.922         6
    386.00    876.00    366.00   1.332    -19.896         6
- - - - - - New 9-Point Grid (only new points calculated) - - - - - -
    383.00    873.00    363.00   1.332    -19.978         6
    384.00    873.00    363.00   1.332    -19.954         6
    385.00    873.00    363.00   1.332    -19.929         6
    383.00    874.00    364.00   1.332    -19.975         6
    383.00    875.00    365.00   1.332    -19.973         6
- - - - - - New 9-Point Grid (only new points calculated) - - - - - -
    383.00    872.00    362.00   1.332    -19.980         6
    384.00    872.00    362.00   1.332    -19.956         6
----- Critical Circle After the Current Mode of Search -----
X: 384.00     Y: 873.00     Radius: 363.000
Factor of safety: 1.332     Side force inclination: -19.954
♀
UTEXAS4 S/N:10001  - Version: 4.1.0.8 - Latest Revision: 11/9/2009
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Time and date of run: Tue Dec 04 16:48:11 2012
Name of input data file: G:\Projects\AEP Big
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e\BigSandy_EL656_EQ_circular.txt

PROJECT:        BIG SANDY DAM
PROJECT NO:     13815153
ANALYSIS:       Max Cross-section - Seismic (pseudo-static) Analysis
                PGA=0.07g
BY:             C.Helm
Filename:       BigSandy_EL656_EQ_circular.dat

TABLE NO. 31
***************************************************
* OUTPUT FOR TYPE 1 AUTOMATIC SEARCH WITH CIRCLES *
***************************************************

------ Output for Circles with a Given, Constant Radius ------
------ Radius: 363.00

                                2-Stage
Center Coordinates              Factor  Side Force
                                  of    Inclination
     X         Y       Radius   Safety   (degrees)   Iterations          Messages

    354.00    843.00    363.00 Center rejected as follows:      UTEXAS WARNING
NUMBER 8080
                                                                Circle passes below
the limiting depth of: 490.000

    384.00    843.00    363.00 Center rejected as follows:      UTEXAS NOTICE NUMBER
8060
                                                                Circle does not
intersect the slope.

    414.00    843.00    363.00 Center rejected as follows:      UTEXAS NOTICE NUMBER
8060
                                                                Circle does not
intersect the slope.

    354.00    873.00    363.00   1.369    -20.171         6
```

```
    414.00    873.00    363.00 Center rejected as follows:      UTEXAS ERROR NUMBER
9280
                                                                The side force
inclination fell outside the range of values allowed.
                                                                The minimum value
allowed is:   -8.00000e+001 degrees.
                                                                The maximum value
allowed is:    1.00000e+001 degrees.
                                                                Error occurred while
computing factor of safety for 2nd stage.

    354.00    903.00    363.00   1.747    -20.187         5
    384.00    903.00    363.00   1.813    -20.587         6
    414.00    903.00    363.00 Center rejected as follows:      UTEXAS ERROR NUMBER
9280
                                                                The side force
inclination fell outside the range of values allowed.
                                                                The minimum value
allowed is:   -8.00000e+001 degrees.
                                                                The maximum value
allowed is:    1.00000e+001 degrees.
                                                                Error occurred while
computing factor of safety for 2nd stage.

- - - - - - New 9-Point Grid (only new points calculated) - - - - - -
    379.00    868.00    363.00   1.330    -19.059         6
    384.00    868.00    363.00   1.323    -18.860         6
    389.00    868.00    363.00   1.325    -18.686         6
    379.00    873.00    363.00   1.333    -20.061         6
    389.00    873.00    363.00   1.333    -19.805         6
    379.00    878.00    363.00   1.349    -21.142         6
    384.00    878.00    363.00   1.350    -21.102         6
    389.00    878.00    363.00   1.352    -21.014         7
- - - - - - New 9-Point Grid (only new points calculated) - - - - - -
    379.00    863.00    363.00   2.141    -18.017         4
    384.00    863.00    363.00   2.170    -17.926         4
    389.00    863.00    363.00   2.199    -17.794         4
- - - - - - New 9-Point Grid (only new points calculated) - - - - - -
    381.00    865.00    363.00   1.972    -18.312         4
    384.00    865.00    363.00   1.983    -18.237         4
    387.00    865.00    363.00   1.993    -18.150         4
    381.00    868.00    363.00   1.326    -18.976         6
    387.00    868.00    363.00   1.324    -18.759         6
    381.00    871.00    363.00   1.328    -19.586         6
    384.00    871.00    363.00   1.328    -19.512         6
    387.00    871.00    363.00   1.329    -19.421         6
- - - - - - New 9-Point Grid (only new points calculated) - - - - - -
    383.00    867.00    363.00   1.637    -18.630         5
    384.00    867.00    363.00   1.636    -18.593         5
    385.00    867.00    363.00   1.635    -18.559         5
    383.00    868.00    363.00   1.323    -18.890         6
    385.00    868.00    363.00   1.323    -18.828         6
    383.00    869.00    363.00   1.324    -19.104         6
    384.00    869.00    363.00   1.325    -19.075         6
    385.00    869.00    363.00   1.325    -19.044         6
- - - - - - New 9-Point Grid (only new points calculated) - - - - - -
    382.00    867.00    363.00   1.639    -18.665         5
    382.00    868.00    363.00   1.324    -18.929         6
    382.00    869.00    363.00   1.324    -19.132         6
----- Critical Circle After the Current Mode of Search -----
X: 383.00     Y: 868.00     Radius: 363.000
Factor of safety: 1.323     Side force inclination: -18.890
♀
```

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Name of input data file: G:\Projects\AEP Big Sandy\10.0_Calculations_Analysis_Data\10.03_Geotechnical\Stability\UTEXAS4\Earthquake\BigSandy_EL656_EQ_circular.txt

PROJECT: BIG SANDY DAM
PROJECT NO: 13815153
ANALYSIS: Max Cross-section - Seismic (pseudo-static) Analysis
PGA=0.07g
BY: C.Helm
Filename: BigSandy_EL656_EQ_circular.dat

TABLE NO. 30
****
* OUTPUT FOR TYPE 1 AUTOMATIC SEARCH WITH CIRCLES *
****

------ Output for Circles Tangent to a Given Horizontal Line ------
------ Tangent line elevation, Y: 505.00

Center Coordinates X	Center Coordinates Y	Radius	2-Stage Factor of Safety	Side Force Inclination (degrees)	Iterations	Messages
353.00	838.00	333.00	1.332	-19.333	6	
383.00	838.00	333.00	1.328	-18.913	6	
413.00	838.00	333.00	1.395	-16.446	6	
353.00	868.00	363.00	1.330	-19.102	6	
413.00	868.00	363.00	1.364	-17.177	6	
353.00	898.00	393.00	1.332	-18.872	6	
383.00	898.00	393.00	1.343	-18.875	6	
413.00	898.00	393.00	Center rejected as follows:			UTEXAS NOTICE NUMBER 8060 Circle does not intersect the slope.
- - - - - - New 9-Point Grid (only new points calculated) - - - - - -						
378.00	863.00	358.00	1.329	-19.099	6	
383.00	863.00	358.00	1.323	-18.906	6	
388.00	863.00	358.00	1.325	-18.728	6	
378.00	868.00	363.00	1.332	-19.096	6	
388.00	868.00	363.00	1.324	-18.722	6	
378.00	873.00	368.00	1.335	-19.091	6	
383.00	873.00	368.00	1.325	-18.885	6	
388.00	873.00	368.00	1.324	-18.712	6	
- - - - - - New 9-Point Grid (only new points calculated) - - - - - -						
380.00	865.00	360.00	1.326	-19.019	6	
383.00	865.00	360.00	1.322	-18.900	6	
386.00	865.00	360.00	1.324	-18.801	6	
380.00	868.00	363.00	1.328	-19.019	6	
386.00	868.00	363.00	1.324	-18.794	6	
380.00	871.00	366.00	1.330	-19.018	6	
383.00	871.00	366.00	1.323	-18.883	6	
386.00	871.00	366.00	1.324	-18.786	6	
- - - - - - New 9-Point Grid (only new points calculated) - - - - - -						
380.00	862.00	357.00	1.325	-19.017	6	
383.00	862.00	357.00	1.323	-18.908	6	
386.00	862.00	357.00	1.324	-18.806	6	
- - - - - - New 9-Point Grid (only new points calculated) - - - - - -						
382.00	864.00	359.00	1.322	-18.933	6	

```
    383.00    864.00    359.00   1.323    -18.903         6
    384.00    864.00    359.00   1.323    -18.871         6
    382.00    865.00    360.00   1.322    -18.929         6
    384.00    865.00    360.00   1.323    -18.868         6
    382.00    866.00    361.00   1.322    -18.927         6
    383.00    866.00    361.00   1.322    -18.897         6
    384.00    866.00    361.00   1.323    -18.866         6
- - - - - - New 9-Point Grid (only new points calculated) - - - - - -
    381.00    863.00    358.00   1.323    -18.972         6
    382.00    863.00    358.00   1.322    -18.936         6
    381.00    864.00    359.00   1.324    -18.973         6
    381.00    865.00    360.00   1.324    -18.974         6
----- Critical Circle After the Current Mode of Search -----
X: 382.00     Y: 864.00     Radius: 359.000
Factor of safety: 1.322     Side force inclination: -18.933
♀
UTEXAS4 S/N:10001  - Version: 4.1.0.8 - Latest Revision: 11/9/2009
Licensed for use by: Craig Helm, URS
Time and date of run: Tue Dec 04 16:48:11 2012
Name of input data file: G:\Projects\AEP Big
Sandy\10.0_Calculations_Analysis_Data\10.03_Geotechnical\Stability\UTEXAS4\Earthquak
e\BigSandy_EL656_EQ_circular.txt

PROJECT:        BIG SANDY DAM
PROJECT NO:     13815153
ANALYSIS:       Max Cross-section - Seismic (pseudo-static) Analysis
                PGA=0.07g
BY:             C.Helm
Filename:       BigSandy_EL656_EQ_circular.dat

TABLE NO. 33
*********************************************
* 2-STAGE FINAL CRITICAL CIRCLE INFORMATION *
*********************************************
X Coordinate of Center . . . . . . . . . . . . . .  382.00
Y Coordinate of Center . . . . . . . . . . . . . .  864.00
Radius . . . . . . . . . . . . . . . . . . . . . .  359.00
Factor of Safety . . . . . . . . . . . . . . . . . 1.322
Side Force Inclination (degrees) . . . . . . . . . -18.93
Number of Circles Tried  . . . . . . . . . . . . .  123
Number of Circles F Calculated for . . . . . . . .  115
Time Required for Search (seconds) . . . . . . . .  0.2

TABLE NO. 34
***************************************************************
* Summary of the 10 Circles with the Lowest Factors of Safety *
***************************************************************
     Center Coordinates            Elevation    Factor    Side
                                   of Bottom      of     Force
        X           Y     Radius   of Circle    Safety   Inclin.    X-Left
X-Right

     382.00     864.00     359.00     505.00    1.322   -18.93        89.40
525.67
     382.00     865.00     360.00     505.00    1.322   -18.93        88.88
525.87
     382.00     863.00     358.00     505.00    1.322   -18.94        89.91
525.46
     382.00     866.00     361.00     505.00    1.322   -18.93        88.37
526.08
     383.00     866.00     361.00     505.00    1.322   -18.90        89.37
527.08
     383.00     865.00     360.00     505.00    1.322   -18.90        89.88
```

526.87
383.00 864.00 359.00 505.00 1.323 -18.90 90.40
526.67
383.00 868.00 363.00 505.00 1.323 -18.89 88.34
527.50
383.00 863.00 358.00 505.00 1.323 -18.91 90.91
526.46
383.00 862.00 357.00 505.00 1.323 -18.91 91.43
526.25
♀
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PROJECT: BIG SANDY DAM
PROJECT NO: 13815153
ANALYSIS: Max Cross-section - Seismic (pseudo-static) Analysis
PGA=0.07g
BY: C.Helm
Filename: BigSandy_EL656_EQ_circular.dat

TABLE NO. 43
```
11111111111111111111111111111111111111111111111111111111111
1 Coordinate, Weight, Strength and Pore Water Pressure     1
1 Information for Individual Slices for Conventional       1
1 Computations or First Stage of Multi-Stage Computations. 1
1 (Information is for the critical shear surface in the    1
1 case of an automatic search.)                            1
11111111111111111111111111111111111111111111111111111111111
```

Slice No.	X	Y	Slice Weight	Matl. No.	Cohesion	Friction Angle	Pore Pressure
	89.40	656.00					
1	89.70	655.58	28	1	0.0	32.00	0.0
	90.00	655.15					
2	95.67	647.66	10400	1	0.0	32.00	0.0
	101.33	640.16					
3	103.67	637.29	9610	1	0.0	32.00	0.0
	106.00	634.42					
4	107.00	633.23	5009	1	0.0	32.00	0.0
	108.00	632.04					
5	114.00	625.30	40518	1	0.0	32.00	0.0
	120.00	618.57					
6	126.60	611.88	58958	1	0.0	32.00	0.0
	133.20	605.19					
7	140.15	598.86	70836	1	0.0	32.00	0.0
	147.09	592.53					
8	154.35	586.56	81636	1	0.0	32.00	0.0
	161.62	580.60					
9	169.19	575.03	91037	1	0.0	32.00	0.0
	176.75	569.46					
10	184.60	564.29	98746	1	0.0	32.00	0.0
	192.45	559.12					
11	200.56	554.37	104501	1	0.0	32.00	0.0
	208.67	549.62					
12	216.83	545.38	105780	1	0.0	32.00	0.0
	225.00	541.15					
13	226.00	540.67	12912	1	0.0	32.00	0.0
	227.00	540.19					
14	227.19	540.09	2500	1	0.0	32.00	0.0

```
          227.39    540.00
  15      232.86    537.50         68974    3          0.0    38.00        156.0
          238.33    535.00
  16      244.67    532.38         77724    4          0.0    25.00        475.7
          251.00    529.75
  17      259.84    526.56        115755    4          0.0    25.00        838.9
          268.67    523.36
  18      277.66    520.62        132189    4          0.0    25.00       1209.0
          286.66    517.89
  19      295.78    515.63        146400    4          0.0    25.00       1520.4
          304.90    513.38
  20      314.13    511.60        158150    4          0.0    25.00       1772.2
          323.36    509.82
  21      326.68    509.30         58990    4          0.0    25.00       1915.4
          330.00    508.79
  22      339.33    507.67        160976    4          0.0    25.00       2017.5
          348.66    506.55
  23      358.04    505.92        148371    4          0.0    25.00       2126.4
          367.41    505.30
  24      374.71    505.15        104579    4          0.0    25.00       2174.8
          382.00    505.00
  25      391.39    505.25        118553    4          0.0    25.00       2168.6
          400.79    505.49
  26      404.39    505.72         40284    4          0.0    25.00       2139.2
          408.00    505.94
  27      417.35    506.87         98152    4          0.0    25.00       2067.4
          426.70    507.79
  28      430.85    508.36         41862    4          0.0    25.00       1974.1
          435.00    508.93
  29      442.50    510.22         63687    4          0.0    25.00       1858.5
          450.00    511.50
  30      455.00    512.54         32074    4          0.0    25.00       1557.7
          460.00    513.58
  31      469.12    515.86         47119    4          0.0    25.00       1194.5
          478.23    518.14
  32      487.22    520.89         34220    4          0.0    25.00        880.2
          496.20    523.65
  33      505.03    526.87         19377    4          0.0    25.00        507.3
          513.86    530.09
  34      519.76    532.55          3912    4          0.0    25.00        153.1
          525.67    535.00
```

No water in crack.
♀
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```
PROJECT:        BIG SANDY DAM
PROJECT NO:     13815153
ANALYSIS:       Max Cross-section - Seismic (pseudo-static) Analysis
                PGA=0.07g
BY:             C.Helm
Filename:       BigSandy_EL656_EQ_circular.dat
```

TABLE NO. 44

```
1111111111111111111111111111111111111111111111111111111111
1 Seismic Forces and Forces Due to Distributed Loads for 1
1 Individual Slices for Conventional Computations or the 1
1 First Stage of Multi-Stage Computations.               1
```

```
1 (Information is for the critical shear surface in the  1
1 case of an automatic search.)                          1
1111111111111111111111111111111111111111111111111111111111

There are no seismic forces or forces due to distributed loads
for the current shear surface
♀
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e\BigSandy_EL656_EQ_circular.txt

PROJECT:         BIG SANDY DAM
PROJECT NO:      13815153
ANALYSIS:        Max Cross-section - Seismic (pseudo-static) Analysis
                 PGA=0.07g
BY:              C.Helm
Filename:        BigSandy_EL656_EQ_circular.dat

TABLE NO. 47
11111111111111111111111111111111111111111111111111111111111111
1  Information for the Iterative Solution for the Factor of  1
1  Safety and Side Force Inclination by Spencer's Procedure  1
1  Stage 1 of Multi-Stage Computations                       1
11111111111111111111111111111111111111111111111111111111111111
Allowable force imbalance for convergence:  24
Allowable moment imbalance for convergence: 6943

        Trial     Trial
       Factor   Side Force     Force       Moment                  Delta
Iter-    of    Inclination   Imbalance   Imbalance    Delta-F      Theta
ation  Safety   (degrees)      (lbs.)    (ft.-lbs.)              (degrees)

   1   3.00000   -17.1887  -2.471e+005   1.451e+008
First-order corrections to F and Theta ............   -2.1667     3.3734
Reduced values - Deltas were too large ............   -0.5000     0.7785

   2   2.50000   -16.4103  -1.784e+005   1.048e+008
First-order corrections to F and Theta ............   -1.0837     1.8346
Reduced values - Deltas were too large ............   -0.5000     0.8464

   3   2.00000   -15.5638  -7.546e+004   4.432e+007
First-order corrections to F and Theta ............   -0.2932     0.4866
Second-order corrections to F and Theta ...........   -0.2596     0.4242

   4   1.74038   -15.1396   1.256e+003  -7.343e+005
First-order corrections to F and Theta ............    0.0037    -0.0088
Second-order corrections to F and Theta ...........    0.0037    -0.0088

   5   1.74412   -15.1484  -5.571e-003   3.234e+000
First-order corrections to F and Theta ............   -0.0000     0.0000
♀
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Sandy\10.0_Calculations_Analysis_Data\10.03_Geotechnical\Stability\UTEXAS4\Earthquak
e\BigSandy_EL656_EQ_circular.txt

PROJECT:         BIG SANDY DAM
PROJECT NO:      13815153
```

ANALYSIS: Max Cross-section - Seismic (pseudo-static) Analysis
PGA=0.07g
BY: C.Helm
Filename: BigSandy_EL656_EQ_circular.dat

TABLE NO. 48

```
2222222222222222222222222222222222222222222222222222222222
2 Detailed Information from Evaluation of Shear Strengths 2
2 for Stage 2 - Information is Only Given for Slices with 2
2 Two-Stage Shear Strengths (Strength Options 6 and 7).   2
2 (Information is for the critical shear surface in the   2
2 case of an automatic search.)                           2
2222222222222222222222222222222222222222222222222222222222
```

Slice No.	Effective Normal Stress at Consol.	Shear Stress at Consol.	Strength (Kc = 1)	Strength (Kc = Kf)	Kc	Kf
16	4874.0	1303.1	2136.0	2272.8	1.711	2.464
17	5014.9	1340.8	2166.0	2338.5	1.711	2.464
18	5537.6	1480.5	2277.1	2582.2	1.711	2.464
19	6035.1	1613.6	2382.8	2814.2	1.711	2.464
20	6501.3	1738.2	2481.9	3031.6	1.711	2.464
21	6804.1	1819.1	2546.3	3172.8	1.711	2.464
22	6595.6	1763.4	2501.9	3075.6	1.711	2.464
23	5963.9	1594.5	2367.7	2781.0	1.711	2.464
24	5309.8	1419.6	2228.6	2476.0	1.711	2.464
25	4550.0	1216.5	2067.1	2121.7	1.711	2.464
26	3896.2	1041.7	1928.2	1816.8	Strength from S envelope.	
27	3700.7	989.4	1886.6	1725.7	Strength from S envelope.	
28	3684.2	985.0	1883.1	1718.0	Strength from S envelope.	
29	2959.9	791.4	1729.2	1380.2	Strength from S envelope.	
30	2129.5	569.3	1552.6	993.0	Strength from S envelope.	
31	1854.7	495.9	1494.2	864.8	Strength from S envelope.	
32	1438.7	384.7	1405.8	670.9	Strength from S envelope.	
33	875.6	234.1	1286.1	408.3	Strength from S envelope.	
34	277.5	74.2	1159.0	129.4	Strength from S envelope.	

♀
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PROJECT: BIG SANDY DAM
PROJECT NO: 13815153
ANALYSIS: Max Cross-section - Seismic (pseudo-static) Analysis
PGA=0.07g
BY: C.Helm
Filename: BigSandy_EL656_EQ_circular.dat

TABLE NO. 49

```
22222222222222222222222222222222222222222222222222222222
2 Coordinate, Weight, Strength and Pore Water Pressure  2
2 Information for Individual Slices for Second          2
2 Stage of Multi-Stage Computations.                    2
2 (Information is for the critical shear surface in the 2
2 case of an automatic search.)                         2
22222222222222222222222222222222222222222222222222222222
```

Slice Slice Matl. Friction Pore

No.	X	Y	Weight	No.	Cohesion	Angle	Pressure
	89.40	656.00					
1	89.70	655.58	28	1	0.0	32.00	0.0
	90.00	655.15					
2	95.67	647.66	10400	1	0.0	32.00	0.0
	101.33	640.16					
3	103.67	637.29	9610	1	0.0	32.00	0.0
	106.00	634.42					
4	107.00	633.23	5009	1	0.0	32.00	0.0
	108.00	632.04					
5	114.00	625.30	40518	1	0.0	32.00	0.0
	120.00	618.57					
6	126.60	611.88	58958	1	0.0	32.00	0.0
	133.20	605.19					
7	140.15	598.86	70836	1	0.0	32.00	0.0
	147.09	592.53					
8	154.35	586.56	81636	1	0.0	32.00	0.0
	161.62	580.60					
9	169.19	575.03	91037	1	0.0	32.00	0.0
	176.75	569.46					
10	184.60	564.29	98746	1	0.0	32.00	0.0
	192.45	559.12					
11	200.56	554.37	104501	1	0.0	32.00	0.0
	208.67	549.62					
12	216.83	545.38	105780	1	0.0	32.00	0.0
	225.00	541.15					
13	226.00	540.67	12912	1	0.0	32.00	0.0
	227.00	540.19					
14	227.19	540.09	2500	1	0.0	32.00	0.0
	227.39	540.00					
15	232.86	537.50	68974	3	0.0	38.00	156.0
	238.33	535.00					
16	244.67	532.38	77724	4	2202.5	0.00	0.0
	251.00	529.75					
17	259.84	526.56	115755	4	2249.8	0.00	0.0
	268.67	523.36					
18	277.66	520.62	132189	4	2425.3	0.00	0.0
	286.66	517.89					
19	295.78	515.63	146400	4	2592.4	0.00	0.0
	304.90	513.38					
20	314.13	511.60	158150	4	2748.9	0.00	0.0
	323.36	509.82					
21	326.68	509.30	58990	4	2850.6	0.00	0.0
	330.00	508.79					
22	339.33	507.67	160976	4	2780.6	0.00	0.0
	348.66	506.55					
23	358.04	505.92	148371	4	2568.4	0.00	0.0
	367.41	505.30					
24	374.71	505.15	104579	4	2348.8	0.00	0.0
	382.00	505.00					
25	391.39	505.25	118553	4	2093.6	0.00	0.0
	400.79	505.49					
26	404.39	505.72	40284	4	1816.8	0.00	0.0
	408.00	505.94					
27	417.35	506.87	98152	4	1725.7	0.00	0.0
	426.70	507.79					
28	430.85	508.36	41862	4	1718.0	0.00	0.0
	435.00	508.93					
29	442.50	510.22	63687	4	1380.2	0.00	0.0
	450.00	511.50					
30	455.00	512.54	32074	4	993.0	0.00	0.0
	460.00	513.58					
31	469.12	515.86	47119	4	864.8	0.00	0.0

	478.23	518.14					
32	487.22	520.89	34220	4	670.9	0.00	0.0
	496.20	523.65					
33	505.03	526.87	19377	4	408.3	0.00	0.0
	513.86	530.09					
34	519.76	532.55	3912	4	129.4	0.00	0.0
	525.67	535.00					

♀

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PROJECT: BIG SANDY DAM
PROJECT NO: 13815153
ANALYSIS: Max Cross-section - Seismic (pseudo-static) Analysis
PGA=0.07g
BY: C.Helm
Filename: BigSandy_EL656_EQ_circular.dat

TABLE NO. 50

```
22222222222222222222222222222222222222222222222222222222222222222222222
2 Seismic Forces and Forces Due to Distributed Loads for Individual 2
2 Slices for the Second Stage of Multi-Stage Computations.          2
2 (Information is for the critical shear surface in the             2
2 case of an automatic search.)                                     2
22222222222222222222222222222222222222222222222222222222222222222222222
```

				FORCES DUE TO DISTRIBUTED LOADS			
Slices No.	X	Seismic Force	Y for Seismic Force	Normal Force	Shear Force	X	Y
1	89.70	2	655.79	0	0	89.70	656.00
2	95.67	728	651.83	0	0	95.67	656.00
3	103.67	673	646.65	0	0	103.67	656.00
4	107.00	351	644.62	0	0	107.00	656.00
5	114.00	2836	640.65	0	0	114.00	656.00
6	126.60	4127	632.18	0	0	126.60	652.47
7	140.15	4959	622.05	0	0	140.15	645.23
8	154.35	5715	612.10	0	0	154.35	637.64
9	169.19	6373	602.37	0	0	169.19	629.72
10	184.60	6912	592.88	0	0	184.60	621.48
11	200.56	7315	583.66	0	0	200.56	612.95
12	216.83	7405	574.82	0	0	216.83	604.26
13	226.00	904	570.01	0	0	226.00	599.36
14	227.19	175	569.41	0	0	227.19	598.72
15	232.86	4828	567.04	0	0	232.86	595.69
16	244.67	5441	561.35	0	0	244.67	589.38
17	259.84	8103	556.03	0	0	259.84	586.00
18	277.66	9253	552.50	0	0	277.66	586.00
19	295.78	10248	549.61	0	0	295.78	586.00
20	314.13	11070	547.32	0	0	314.13	586.00
21	326.68	4129	546.04	0	0	326.68	586.00
22	339.33	11268	543.00	0	0	339.33	581.69
23	358.04	10386	537.79	0	0	358.04	573.06
24	374.71	7321	533.61	0	0	374.71	565.37
25	391.39	8299	529.93	0	0	391.39	557.66
26	404.39	2820	527.28	0	0	404.39	551.66
27	417.35	6871	527.08	0	0	417.35	550.00
28	430.85	2930	527.86	0	0	430.85	550.00

29	442.50	4458	526.41	0	0	442.50	545.00
30	455.00	2245	524.45	0	0	455.00	537.50
31	469.12	3298	525.43	0	0	469.12	535.00
32	487.22	2395	527.95	0	0	487.22	535.00
33	505.03	1356	530.94	0	0	505.03	535.00
34	519.76	274	533.77	0	0	519.76	535.00

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PROJECT: BIG SANDY DAM
PROJECT NO: 13815153
ANALYSIS: Max Cross-section - Seismic (pseudo-static) Analysis PGA=0.07g
BY: C.Helm
Filename: BigSandy_EL656_EQ_circular.dat

TABLE NO. 51

```
22222222222222222222222222222222222222222222222222222222222222
2  Information for the Iterative Solution for the Factor of  2
2  Safety and Side Force Inclination by Spencer's Procedure  2
2  Second Stage of Multi-Stage Computations                  2
22222222222222222222222222222222222222222222222222222222222222
```

Allowable force imbalance for convergence: 24
Allowable moment imbalance for convergence: 6943

```
        Trial     Trial
       Factor   Side Force     Force       Moment                   Delta
Iter-    of     Inclination   Imbalance   Imbalance    Delta-F      Theta
ation  Safety   (degrees)      (lbs.)    (ft.-lbs.)               (degrees)

   1   3.00000   -17.1887  -4.168e+005   2.506e+008
First-order corrections to F and Theta ............   -3.6105    -5.8928
Reduced values - Deltas were too large ............   -0.5000    -0.8161

   2   2.50000   -18.0048  -3.483e+005   2.094e+008
First-order corrections to F and Theta ............   -2.1588    -1.9677
Reduced values - Deltas were too large ............   -0.5000    -0.4557

   3   2.00000   -18.4605  -2.485e+005   1.493e+008
First-order corrections to F and Theta ............   -1.0060    -0.7229
Reduced values - Deltas were too large ............   -0.5000    -0.3593

   4   1.50000   -18.8199  -8.590e+004   5.161e+007
First-order corrections to F and Theta ............   -0.2006    -0.1264
Second-order corrections to F and Theta ...........   -0.1799    -0.1137

   5   1.32006   -18.9336   1.153e+003  -6.927e+005
First-order corrections to F and Theta ............    0.0021     0.0010
Second-order corrections to F and Theta ...........    0.0021     0.0010

   6   1.32217   -18.9326  -2.699e-003   1.620e+000
First-order corrections to F and Theta ............   -0.0000    -0.0000
```

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PROJECT: BIG SANDY DAM
PROJECT NO: 13815153
ANALYSIS: Max Cross-section - Seismic (pseudo-static) Analysis
PGA=0.07g
BY: C.Helm
Filename: BigSandy_EL656_EQ_circular.dat

TABLE NO. 55

```
*********************************************************************
* Check of Computations by Spencer's Procedure (Results are for the *
* critical shear surface in the case of an automatic search.)       *
*********************************************************************
```

Summation of Horizontal Forces: 1.91735e-010

Summation of Vertical Forces: 1.87359e-010

Summation of Moments: 5.55301e-008

Mohr Coulomb Shear Force/Shear Strength Check Summation: 4.47944e-011
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PROJECT: BIG SANDY DAM
PROJECT NO: 13815153
ANALYSIS: Max Cross-section - Seismic (pseudo-static) Analysis
PGA=0.07g
BY: C.Helm
Filename: BigSandy_EL656_EQ_circular.dat

TABLE NO. 58

```
*************************************************************************
* Final Results for Stresses Along the Shear Surface                    *
* (Results are for the critical shear surface in the case of a search.) *
*************************************************************************
```

SPENCER'S PROCEDURE USED TO COMPUTE THE FACTOR OF SAFETY
Factor of Safety: 1.322 Side Force Inclination: -18.93

-------- VALUES AT CENTER OF BASE OF SLICE --------

Slice No.	X-Center	Y-Center	Total Normal Stress	Effective Normal Stress	Shear Stress
1	89.70	655.58	22.9	22.9	10.8
2	95.67	647.66	467.2	467.2	220.8
3	103.67	637.29	1092.1	1092.1	516.1
4	107.00	633.23	1350.5	1350.5	638.3
5	114.00	625.30	1879.2	1879.2	888.1
6	126.60	611.88	2618.8	2618.8	1237.7
7	140.15	598.86	3147.4	3147.4	1487.5
8	154.35	586.56	3638.5	3638.5	1719.6
9	169.19	575.03	4081.2	4081.2	1928.8
10	184.60	564.29	4465.2	4465.2	2110.3
11	200.56	554.37	4779.9	4779.9	2259.0
12	216.83	545.38	5013.5	5013.5	2369.4

13	226.00	540.67	5116.9	5116.9	2418.3
14	227.19	540.09	5126.9	5126.9	2423.0
15	232.86	537.50	5033.4	4877.4	2882.1
16	244.67	532.38	5140.3	5140.3	1665.8
17	259.84	526.56	5657.8	5657.8	1701.6
18	277.66	520.62	6562.1	6562.1	1834.3
19	295.78	515.63	7391.4	7391.4	1960.7
20	314.13	511.60	8138.4	8138.4	2079.1
21	326.68	509.30	8608.5	8608.5	2156.0
22	339.33	507.67	8537.7	8537.7	2103.0
23	358.04	505.92	8072.9	8072.9	1942.6
24	374.71	505.15	7518.9	7518.9	1776.5
25	391.39	505.25	6804.1	6804.1	1583.5
26	404.39	505.72	6141.0	6141.0	1374.1
27	417.35	506.87	5898.9	5898.9	1305.2
28	430.85	508.36	5823.3	5823.3	1299.3
29	442.50	510.22	4972.1	4972.1	1043.9
30	455.00	512.54	3815.9	3815.9	751.0
31	469.12	515.86	3183.5	3183.5	654.1
32	487.22	520.89	2445.5	2445.5	507.4
33	505.03	526.87	1474.3	1474.3	308.8
34	519.76	532.55	463.7	463.7	97.9

♀
UTEXAS4 S/N:10001 - Version: 4.1.0.8 - Latest Revision: 11/9/2009
Licensed for use by: Craig Helm, URS
Time and date of run: Tue Dec 04 16:48:11 2012
Name of input data file: G:\Projects\AEP Big Sandy\10.0_Calculations_Analysis_Data\10.03_Geotechnical\Stability\UTEXAS4\Earthquake\BigSandy_EL656_EQ_circular.txt

PROJECT: BIG SANDY DAM
PROJECT NO: 13815153
ANALYSIS: Max Cross-section - Seismic (pseudo-static) Analysis PGA=0.07g
BY: C.Helm
Filename: BigSandy_EL656_EQ_circular.dat

TABLE NO. 59

```
***************************************************************************
* Final Results for Side Forces and Stresses Between Slices               *
* (Results are for the critical shear surface in the case of a search.)   *
***************************************************************************
```

--------------- VALUES AT RIGHT SIDE OF SLICE ---------------

Slice No.	X-Right	Side Force	Y-Coord. of Side Force Location	Fraction of Height	Sigma at Top	Sigma at Bottom
1	90.00	16	655.50	0.411	8.2	26.9
2	101.33	5548	646.31	0.388	109.1	553.4
3	106.00	10330	641.55	0.330	-8.8	914.5
4	108.00	12754	639.68	0.319	-44.3	1051.3
5	120.00	31252	629.74	0.299	-164.7	1744.2
6	133.20	55370	620.02	0.339	39.9	2354.2
7	147.09	80922	610.83	0.374	376.9	2747.4
8	161.62	106409	602.08	0.404	803.1	2983.9
9	176.75	130374	593.81	0.433	1313.6	3073.9
10	192.45	151465	586.07	0.463	1920.4	3005.9
11	208.67	168490	578.93	0.497	2648.4	2753.8
12	225.00	180281	572.59	0.535	3516.9	2289.1
13	227.00	181344	571.88	0.540	3634.9	2215.6
14	227.39	181540	571.74	0.541	3658.1	2200.9

15	238.33	179898	569.12	0.591	4546.7	1344.7
16	251.00	191850	563.49	0.600	5157.4	1295.4
17	268.67	206893	556.18	0.524	3573.7	2674.4
18	286.66	219710	549.54	0.465	2403.9	3698.9
19	304.90	228012	543.77	0.419	1518.5	4421.1
20	323.36	229732	539.05	0.384	860.7	4844.5
21	330.00	228385	537.64	0.374	676.9	4918.7
22	348.66	218977	534.31	0.392	1026.6	4821.6
23	367.41	202154	531.70	0.416	1499.6	4529.1
24	382.00	184857	530.14	0.441	1982.8	4152.5
25	400.79	158638	528.64	0.484	2834.9	3438.9
26	408.00	148217	528.13	0.504	3249.9	3114.4
27	426.70	118127	527.55	0.468	2141.7	3153.1
28	435.00	102812	527.94	0.463	1840.4	2895.9
29	450.00	77487	528.73	0.605	4187.4	956.0
30	460.00	63542	528.93	0.717	6450.2	-839.3
31	478.23	39065	529.76	0.689	4677.2	-294.2
32	496.20	17712	531.57	0.698	3229.1	-277.3
33	513.86	3338	534.46	0.891	2152.7	-865.9
34	525.67	0	535.00	0.000	0.0	0.0

Read end-of-file on input while looking for another command word.
End of input data assumed - normal termination.

ATTACHMENT B

Geometry and Material Properties

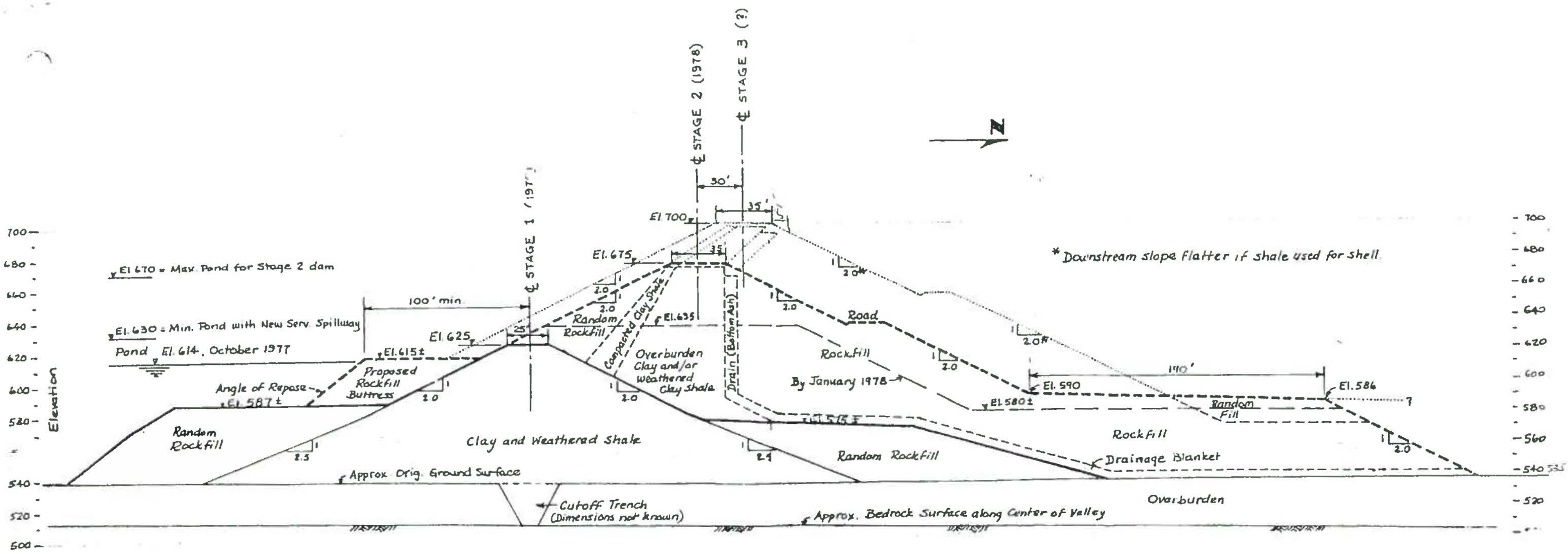
KENTUCKY POWER COMPANY
BIG SANDY PLANT
FLY ASH RETENTION DAM
STAGE 3 RAISING
ENGINEERING REPORT

Prepared by
American Electric Power Service Corporation
Civil Engineering Department
Geotechnical Section

March 1993

Max Cross-Section Used.

From: Stage 3 Raising Engineering Report, 1993

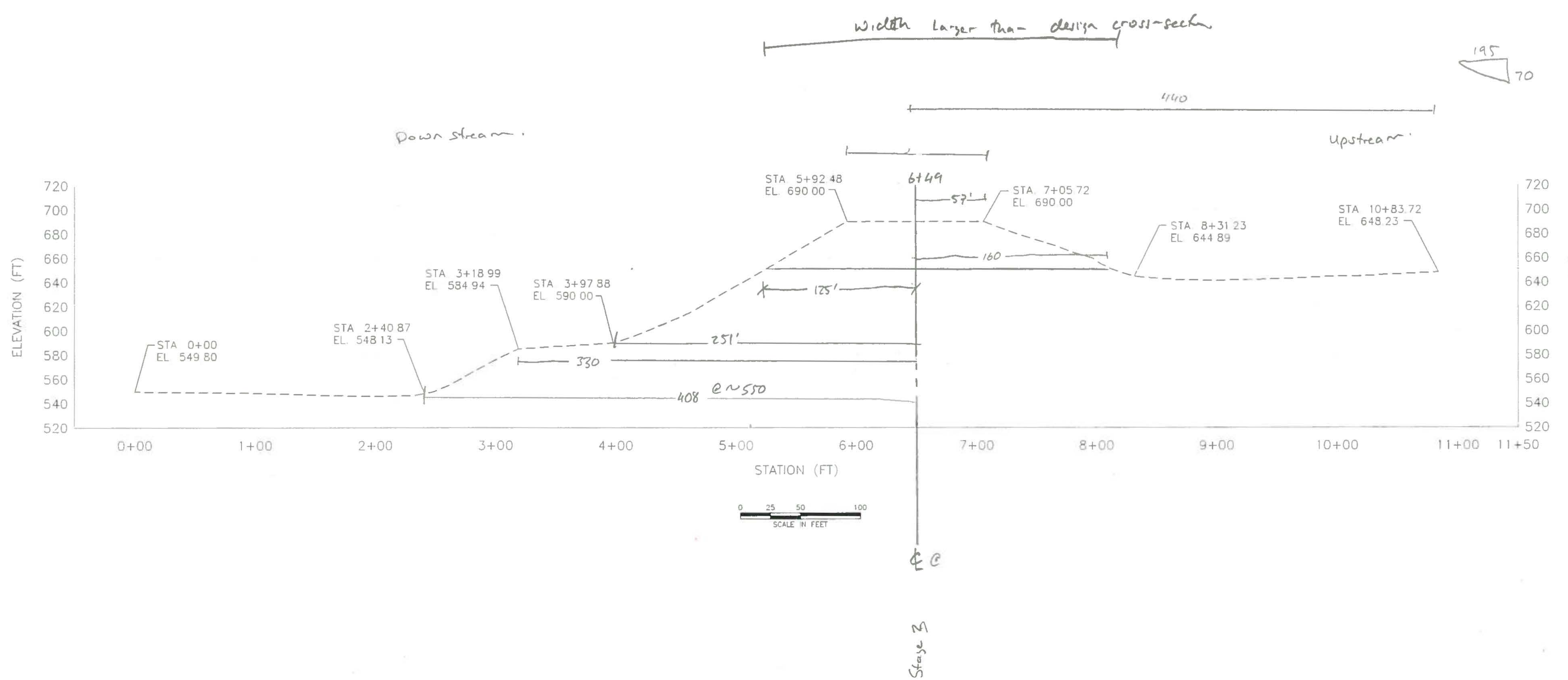


Scale: 1 in. = 60 ft

FIG. 1 - CROSS-SECTION OF BIG SANDY DAM ALONG CENTER OF VALLEY

CC Nov. '77

FIG. B-1



"2010 Survey" from AEP
(topographical)

<u>Not Used</u> in final Geometry for stability.

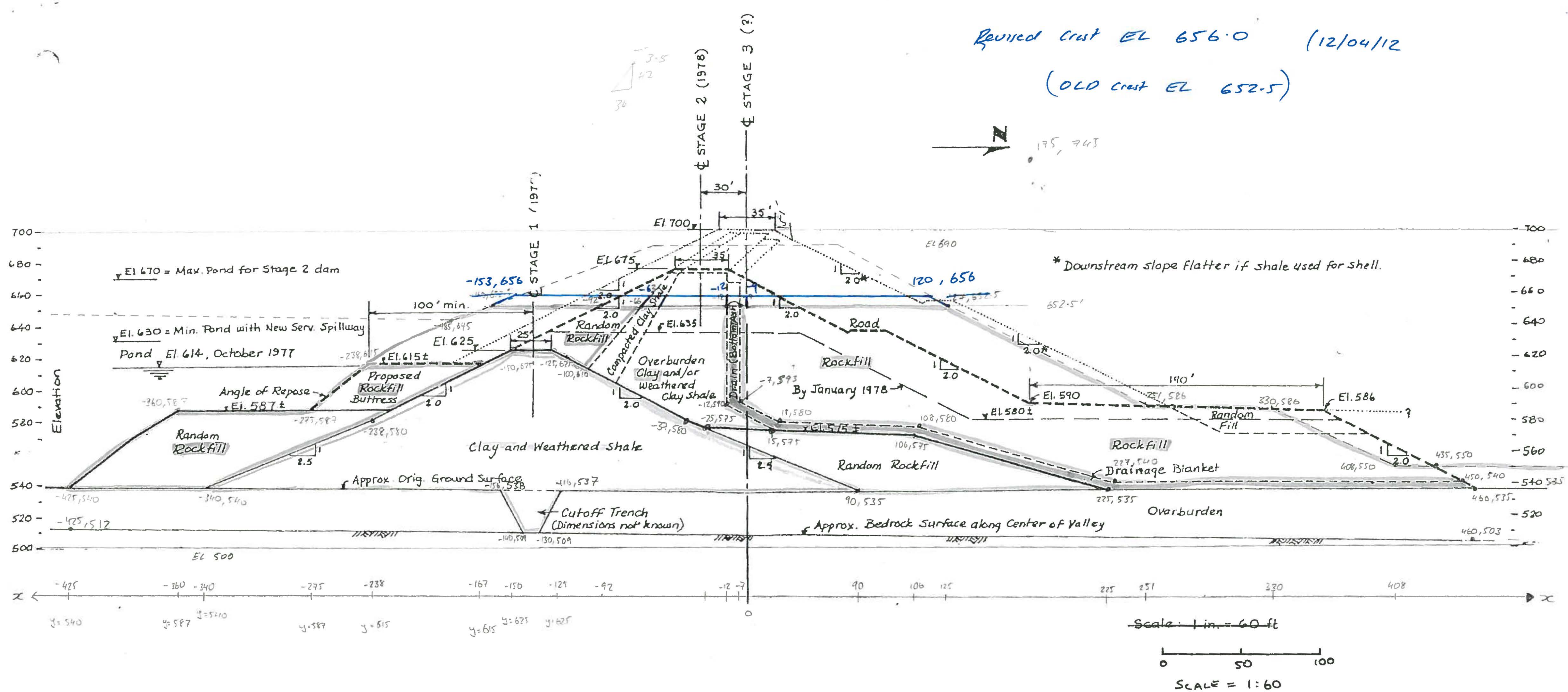


FIG. 1 - CROSS-SECTION OF BIG SANDY DAM ALONG CENTER OF VALLEY

MAX CROSS-SECTION.

(Grid added to define geometry of material zones/geometry)

CC Nov. '77

FIG. B-1

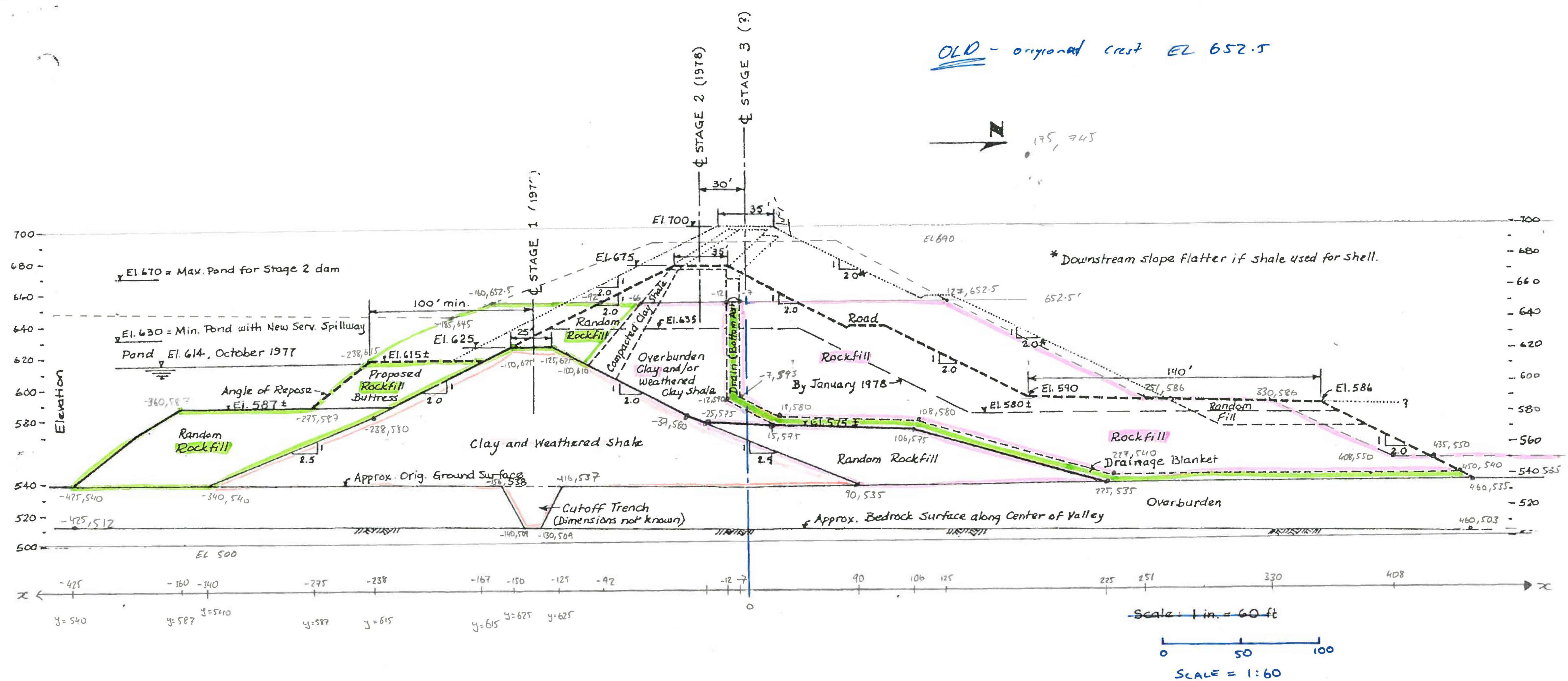


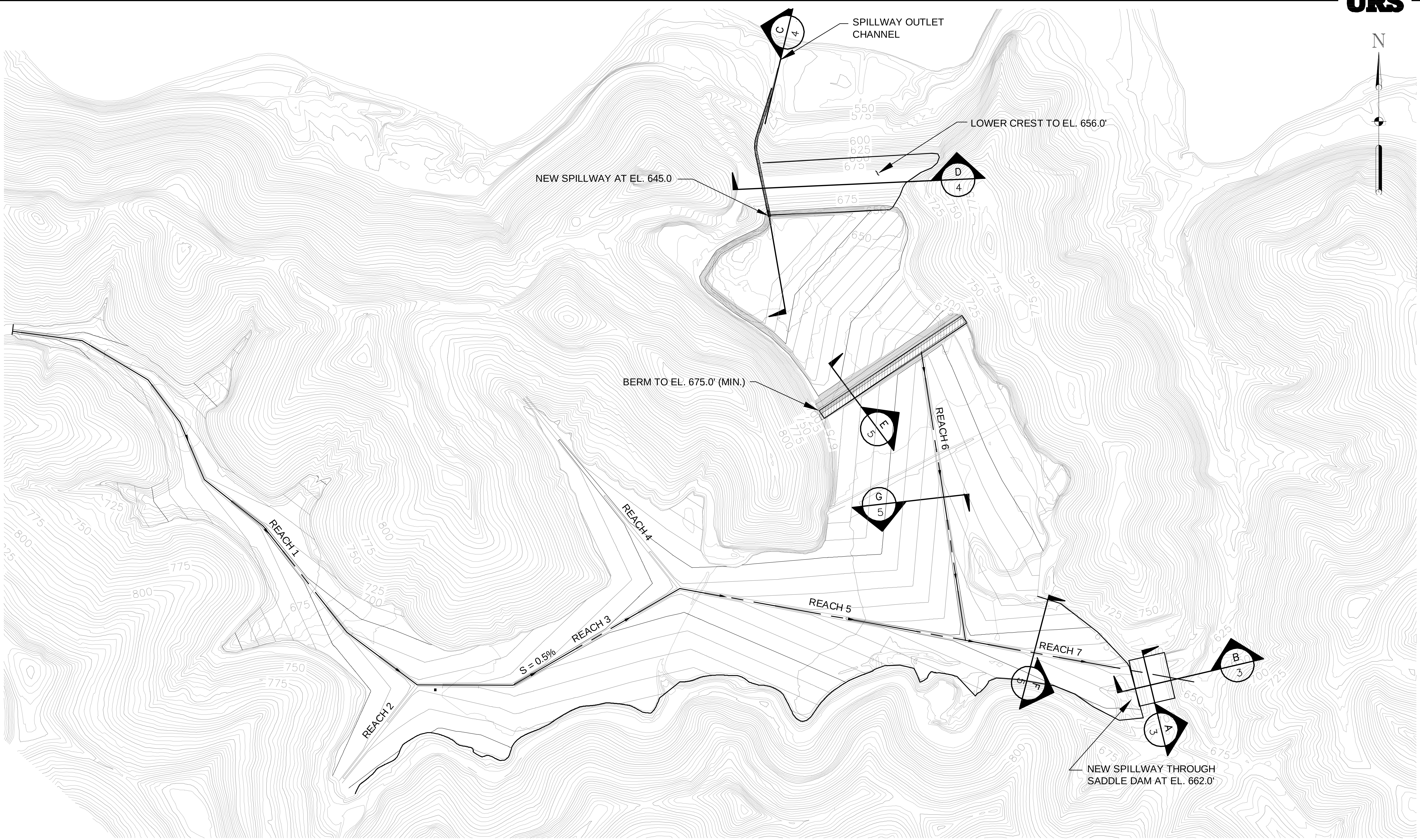
FIG. 1 - CROSS-SECTION OF BIG SANDY DAM ALONG CENTER OF VALLEY

Max Cross-Section.

(Grid added to define geometry of material zones/geometry)

CC Nov. '77

FIG. B-1



Job No. :	13815153
Prepared By :	SWR
Date :	11/20/12

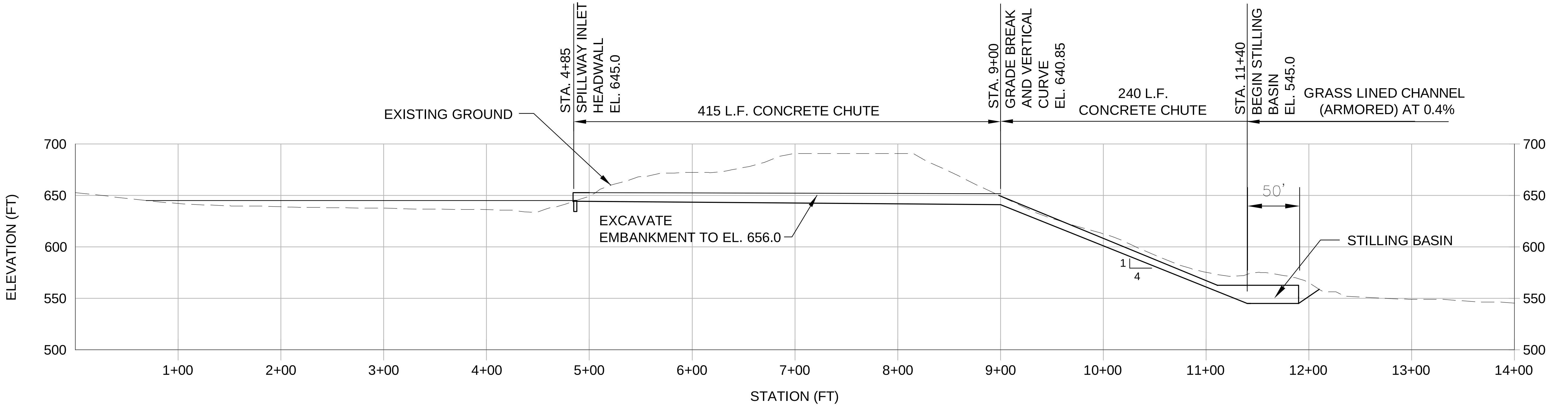
PROPOSED PLAN

AEP BIG SANDY ASH POND CLOSURE

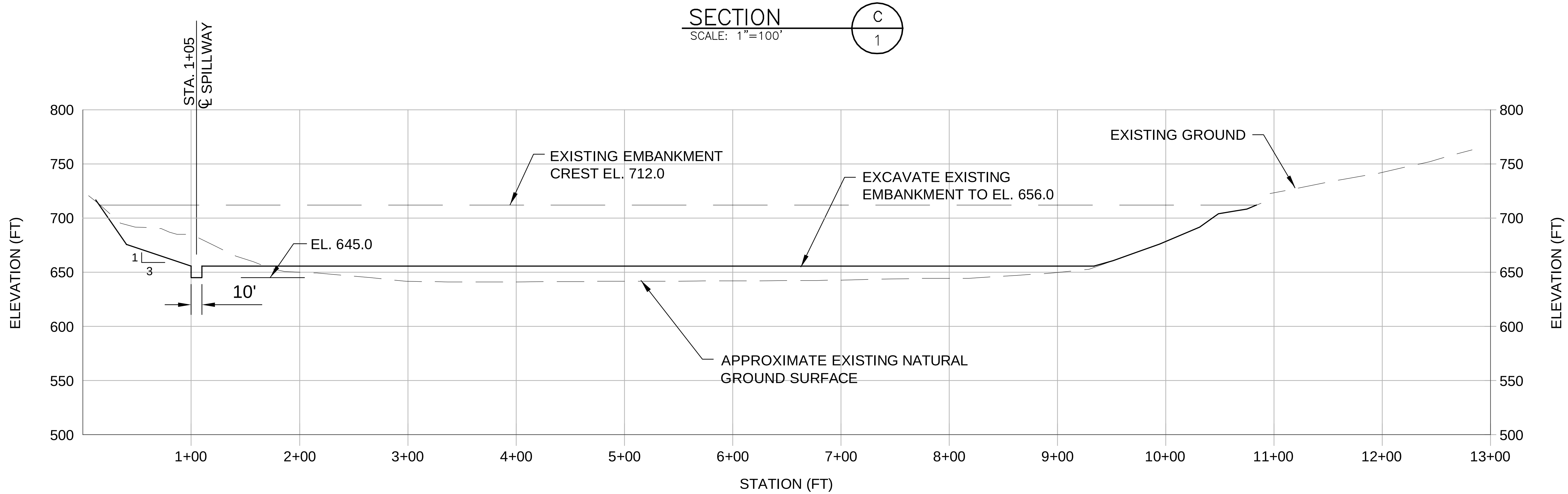
FIG. 1

DRAWING PATH: T:\Projects\Glenwood\AEP Big Sandy\AutoCAD Files\PROPOSED PLAN.dwg

MILLER, RANDY A. , 11/20/2012 4:00 PM



SECTION C/1
SCALE: 1"=100'



SECTION D/1
SCALE: 1"=100'

0 50 100 200
SCALE IN FEET

Job No. :	13815153
Prepared By :	SWR
Date :	11/20/12

SECTIONS C AND D

AEP BIG SANDY ASH POND CLOSURE

FIG. 4

URS DRAWING PATH: T:\Projects\Glenwood\AEP Big Sandy\AutoCAD Files\PROPOSED PLAN.dwg

MILLER, RANDY A . 11/20/2012 4:00 PM

From: Appendix F - Geotechnical Analysis's → "Stage 3 Raising Engineering Report" March 1993

9	185.0	70.0	205.0	80.0	4
10	205.0	80.0	345.0	80.0	4
11	345.0	80.0	455.0	135.0	4
12	455.0	135.0	490.0	135.0	4
13	490.0	135.0	491.0	170.0	5
14	490.0	135.0	500.0	135.0	5
15	500.0	135.0	501.0	175.0	2
16	535.0	175.0	570.0	135.0	2
17	570.0	135.0	615.0	135.0	4
18	615.0	135.0	645.0	125.0	4
19	645.0	125.0	720.0	87.0	2
20	35.0	40.0	275.0	45.0	5
21	275.0	45.0	345.0	80.0	4
22	275.0	45.0	380.0	80.0	5
23	380.0	80.0	465.0	80.0	5
24	465.0	80.0	489.0	90.0	5
25	489.0	90.0	490.0	135.0	5
26	260.0	38.0	380.0	75.0	4
27	380.0	75.0	465.0	75.0	4
28	465.0	75.0	499.0	90.0	2
29	499.0	90.0	500.0	135.0	2
30	465.0	75.0	515.0	75.0	4
31	515.0	75.0	615.0	125.0	2
32	615.0	125.0	645.0	125.0	2
33	260.0	38.0	415.0	38.0	1
34	415.0	38.0	430.0	45.0	1
35	430.0	45.0	515.0	75.0	2
36	430.0	45.0	720.0	45.0	1
37	.0	35.0	30.0	35.0	3
38	30.0	35.0	260.0	35.0	6
39	260.0	35.0	400.0	35.0	6
40	400.0	35.0	720.0	35.0	7

ISOTROPIC Soil Parameters Used in 1993 Stability Analysis.
"STEADY STATE SEEPAGE"

8 type(s) of soil

Soil Unit No.	Unit Weight Moist (pcf)	Unit Weight Sat. (pcf) γ_T	Cohesion Intercept (psf) c'	Friction Angle (deg) ϕ'	Pore Pressure Parameter Ru	Pore Pressure Constant (psf)	Water Surface No.
1	128.0	135.0	1000.0	23.00	.000	.0	1
2	128.0	135.0	.0	25.00 - Foundation Soils	.000	.0	1
3	125.0	130.0	.0	25.00 - Embankment	.000	.0	1
4	105.0	110.0	.0	24.00	.000	.0	1
5	70.0	70.0	.0	38.00 - Drain	.000	.0	1
6	128.0	135.0	.0	25.00	.000	.0	1
7	128.0	135.0	1000.0	23.00	.000	.0	1
8	128.0	135.0	.0	27.00	.000	.0	1

Rockfill
Very Low → Revise.
→ use $\gamma = 110$ pcf $c' = 0$ phi' (ϕ') = 32 degrees.

1 Water surface(s) have been specified

Unit weight of water = 62.40 (pcf)

From: Stage 3 Raising Engineering Report, 1993

TABLE 7.2

SUMMARY OF DESIGN SHEAR STRENGTHS AT MAIN DAM

CASE IV: PARTIAL POOL
* CASE V: STEADY SEEPAGE WITH MAXIMUM STORAGE POOL *
CASE VI: STEADY SEEPAGE WITH SURCHARGE POOL

DAM ZONE	DESIGN SHEAR STRENGTH (TSF)	COMMENTS
* FOUNDATION SOIL: UNDER EXISTING BERM UNDER EXISTING DAM*	 σ' tan 25° 0.5 + σ' tan 23°	Foundation Soils. Kc=1; α = 90°
* FOUNDATION SOIL: UNDER EXISTING BERM UNDER EXISTING DAM*	 σ' tan 27° 0.4 + σ' tan 23°	Kc=1; α = 45°
* FOUNDATION SOIL: UNDER EXISTING BERM UNDER EXISTING DAM*	 σ' tan 30° σ' tan 30°	Kc=1.75; α =45°
* CLAY IN EXISTING DAM	σ' tan 25°	Embankment
* RANDOM ROCKFILL	~~σ' tan 24°~~	Low → Use phi'=32
* BOTTOM ASH	σ' tan 38°	Drain
* COMPACTED CLAY	σ' tan 27°	Mc= -1% to+2%

* (Confining pressure $\sigma \geq 5$ tsf)

Bedrock (Sandstone) c'=8000psf ϕ'= 0 deg

Case VII: Earthquake. Much research is in progress on the behavior of earth dams subjected to earthquake shocks, and new analytical methods for evaluating seismic effects are being developed. However, for this design, the traditional approach was used. This assumes that the earthquake imparts an additional horizontal force F_h acting in the direction of potential failure. The arc or set of planes found to be critical without earthquake loading is used with this added driving force to determine the factor of safety for Cases I, VI, V, VI. The horizontal seismic force is equal to the mass involved times the horizontal acceleration, i.e.

$$F_h = \frac{W}{g} a_h$$

The total weight of the sliding soil mass W should be based on saturated unit weights below the saturation line and moist unit weights above the line. Selection of the seismic coefficient should be based on the degree of seismic

A summary of the shear strength obtained from each of these tests is presented on Table No. 5.1.

TABLE NO. 5.1

SUMMARY OF SHEAR STRENGTH OF FOUNDATION SOILS AT MAIN DAM

			Shear Strength (TSF)			
	Con. Rat.		Total Stress		Effec. Stress	
α	Kc	Test	20% Strain	15% Strain	20% Strain	15% Strain
90°	--	UU (Q)	$\sigma \leq 2.5$ $S = \sigma \tan 16°$ $2.5 < \sigma < 5.0$ $S = 0.4 + \sigma \tan 6°$ $5.0 \leq \sigma$ $S = 1.0$	$S = \sigma \tan 14°$ $S = 0.35 + \sigma \tan 7°$ $S = 0.85$	---	---
90°	--	UU (Q)	$5.0 \leq \sigma$ $S = 1.1$	$S = 1.0$	---	---
90°	1	Cu (R)	$1.1 + \sigma \tan 16°$	$0.8 + \sigma \tan 17°$	$\sigma' \tan 28°$	$\sigma' \tan 31°$
90°	1	Cu (R)	$0.6 + \sigma \tan 18°$	$0.8 + \sigma \tan 19°$	$\sigma' \tan 25°$	$\sigma' \tan 27°$
90°	1	DS	---	---	$\sigma' \tan 26°$	$\sigma' \tan 27°$
90°	1	DS	---	---	$\sigma' \tan 35°$+	$\sigma' \tan 37°$+
30°	1	Cu (R)	$1.1 + \sigma \tan 12°$	$1.3 + \sigma \tan 12°$	$\sigma' \tan 23°$*	$\sigma' \tan 25°$
45°	1	Cu (R)	$0.65 + \sigma \tan 18°$	$1.0 + \sigma \tan 15°$	$\sigma' \tan 27°$	$\sigma' \tan 25°$
45°	1.75	Cu (R)	$4.8 + \sigma \tan 6°$**	---	$\sigma' \tan 34°$**+	---
45°	1.75	Cu (R)	$1.9 + \sigma \tan 12°$**	---	$\sigma' \tan 30°$**	---

** At approximately 6% strain.
* Appears to be low.
+ Appears to be high.

Select from these two tests

Selected design strengths for Earthquake analysis

Strength of Clay in Existing Dam:

Five triaxial compression tests under unconsolidated undrained, UU, Conditions were performed on samples from Borings BSFD-1, BSFD-2, BSFD-3 and BSFD-3A from depths above the elevation of the original ground surface. These soils consisted of very stiff to hard brown silty clay. Total stress circles for these tests are

Foundation Soils. - 2-Stage Analysis. (earthquake)

Drained
$c' = 0$ psf
$\phi' = 25$ degrees

Undrained
$c = 1.1$ tsf
$\phi = 12$ degreee

URS

Page ____ of ____

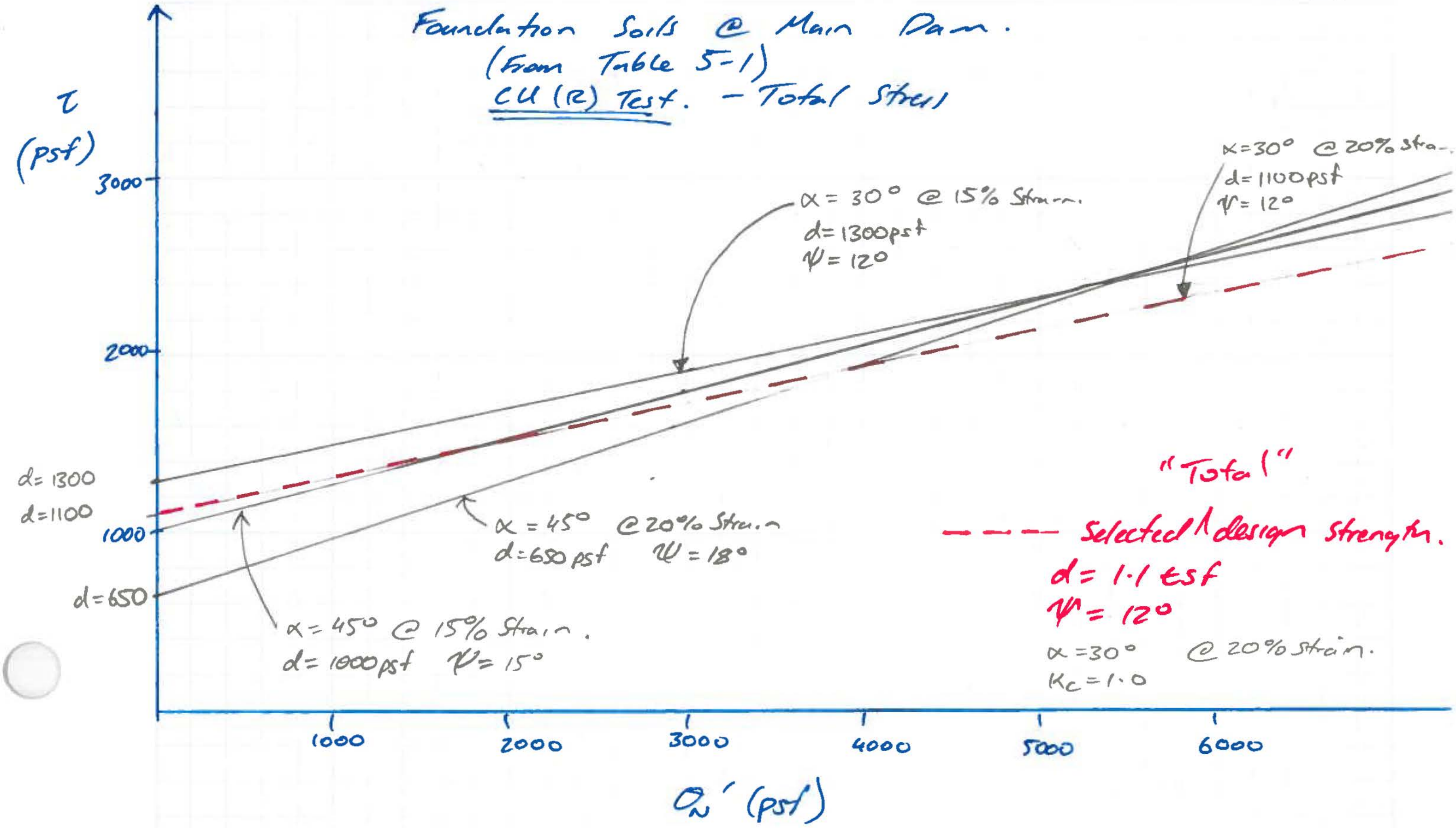
Job: Big Sandy | Project No. ____ | Sheet ____ of ____

Description: ~~Main Dam~~ Undrained Strength -

Computed by ____ Date ____

Checked by ____ Date ____

Reference



Undrained $d = 1100\,psf$

$\psi = 12°$

Drained $c' = 0\ psf$

$\phi = 25°$

Bilinear Envelope: use b/c @ lower confining stresses, drained strengths are lower (conservative) than undrained strengths. Drained strengths may represent foundation materials if pore pressures do not increase during earthquake or drainage is sufficient to prevent excess pore pressures being generated.

σ_N critical

$$d + \sigma_N \tan\psi = c + \sigma_N \tan\phi$$

$$1100 + \sigma_N \tan 12 = 0 + \sigma_N \tan 25$$

$$\frac{1100}{(\tan 25 - \tan 12)} = \sigma_N = 4922\,psf.$$

~~A~~ Bilinear Envelope: $\phi = 25$ degrees $\sigma_N < 4920$ psf; $\phi = 12$ degrees $\sigma_N > 4920$ psf. } Not Used.

TABLE NO. 5.2

SUMMARY OF SHEAR STRENGTH OF CLAY IN EXISTING MAIN DAM

TEST	SHEAR STRENGTH (TSF) TOTAL STRESS 20% STRAIN	SHEAR STRENGTH (TSF) TOTAL STRESS 15% STRAIN	SHEAR STRENGTH (TSF) EFFECTIVE STRESS 20% STRAIN	SHEAR STRENGTH (TSF) EFFECTIVE STRESS 15% STRAIN
UU (Q)	$\sigma \leq 3.0$ tsf $S=\sigma \tan 28^\circ$+ $3.0<\sigma<6.0$ $S=0.4+\sigma \tan 24^\circ$ $6.0 \leq \sigma$ $S = 3.0$	$S=\sigma \tan 22^\circ$ $S=\sigma \tan 22^\circ$ $S=2.7$	---	---
UU (Q)	$\sigma \leq 4$+sf $S=\sigma \tan 20^\circ$ $4<\sigma<8.3$tsf $S=0.6+\sigma \tan 11^\circ$ $8.3<\sigma<15$tsf $S=0.7+\sigma \tan 10^\circ$ $\sigma<15$tsf $S=3.5$	$S=\sigma \tan 20^\circ$ $S=0.6+\sigma \tan 11^\circ$ $S=0.6+\sigma \tan 11^\circ$ $S=3.8$	---	---
Cu (R)	$0.8+\sigma \tan 20^\circ$	$0.5+\sigma \tan 23^\circ$	$\sigma' \tan 25^\circ$	$\sigma' \tan 27^\circ$
Cu (R)	$1.1+\sigma \tan 18^\circ$	$1.2+\sigma \tan 16^\circ$	$\sigma' \tan 26^\circ$	$\sigma' \tan 29^\circ$
DS	---	---	$\sigma' \tan 23^\circ$	$\sigma' \tan 27^\circ$
DS	---	---	$1.7+\sigma' \tan 17^\circ$	$1.6+\sigma' \tan 17^\circ$

Select from these two tests

+ Appears to be high.

Strength of Clay from Proposed Borrow:

The samples tested came from combining samples obtained from the seven test trenches dug at the location of the proposed borrow site. The samples consisted of brown silty clay, "and" fine to coarse sand, trace fine to coarse gravel. Before testing, all composite samples were remolded by compacting the soil to a dry unit weight equal to 95% of the maximum dry unit weight as obtained in accordance with ASTM D698. Strength tests were conducted on remolded samples having moisture contents from about -1% to +3% of the optimum moisture content.

Six triaxial compression tests under unconsolidated undrained, UU, conditions, were performed on samples remolded at moisture contents of -1% and +3% of the optimum moisture content. Total

Embankment Soil - 2-Stage Analysis (Earthquake)

Drained

$c' = 0$ psf

$\phi' = 25$ degrees

Undrained

$c = 500$psf

$\psi = 23$ degree

URS

Page ____ of ____

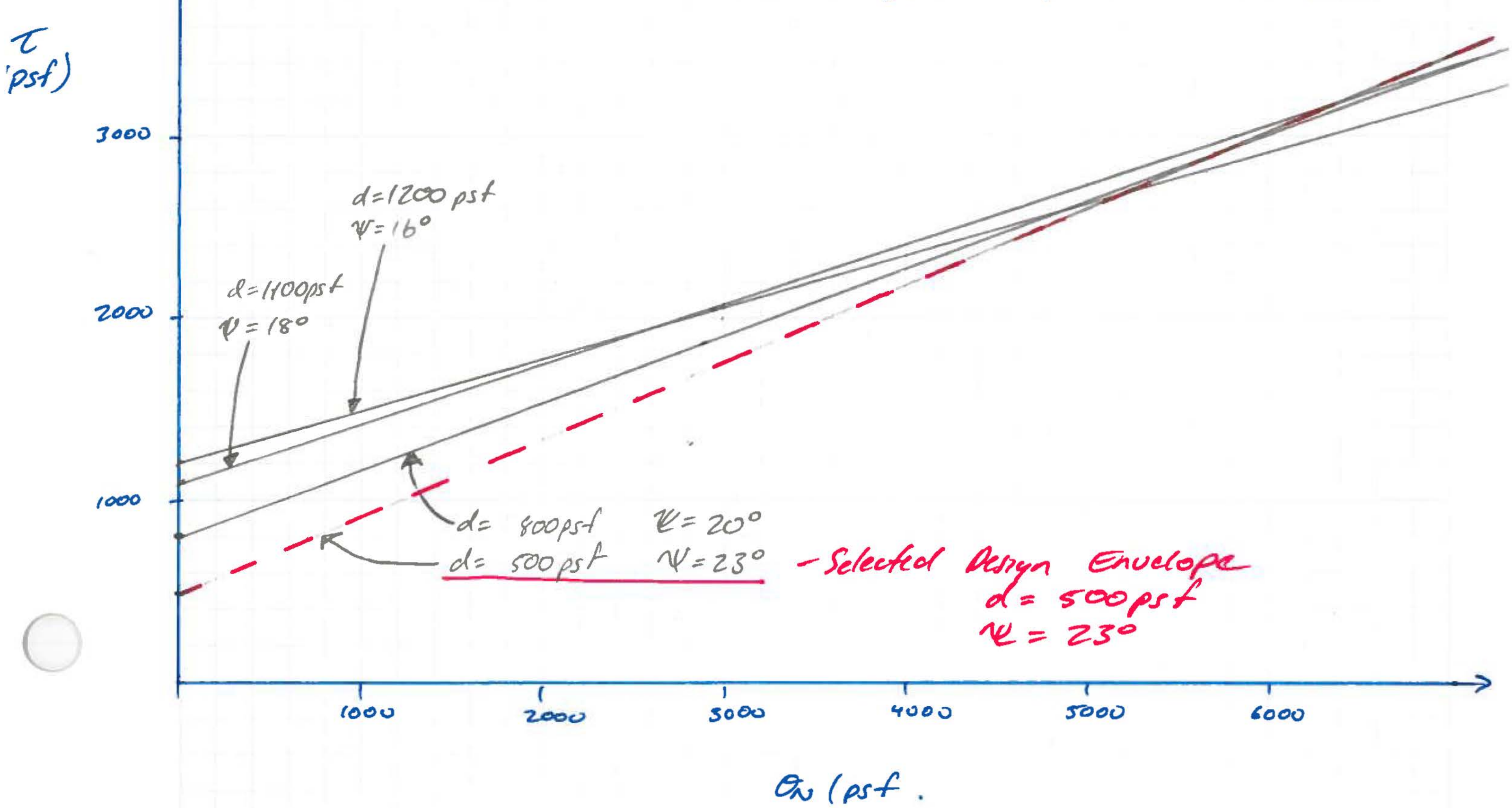
Job ____ Project No. ____ Sheet ____ of ____

Description ~~Plot~~ Computed by ____ Date ____

Undrained Strength - Main dam. Checked by ____ Date ____

Reference

Embankment (CLAY) Soils in Main Dam.
(From TABLE 5.2) CU (R) Test. - Total Stress.



Undrained : $d = 500$ psf
$\Psi = 23$ degrees

Drained : $c' = 0$ psf
$\phi' = 25$ degrees.

Bilinear Envelope : used b/c drained conditions may exist & will have lower strength @ lower confining stress than undrained strength

Critical σ_N = drained = undrained

$c + \sigma_N \tan\phi = d + \sigma_N \tan\Psi$

$0 + \sigma_N \tan 25 = 500 + \sigma_N \tan 23$

$\sigma_N (\tan 25 - \tan 23) = 500$

$\sigma_N = 13{,}757$ psf. →

∴ Bi-Linear Envelope. $\phi = 25$ $\sigma_N < 13{,}800$
$\phi = 23$ $\sigma_N > 13{,}800$

This is above σ_N in embankment, so bilinear envelope Not Needed. Just use drained strengths for conservatism.