Tennessee Valley Authority
Regulatory Submittal for Kingston Fossil Plant

Documents submitted:
Test Embankment Buttress Work Plan, Revision 0

Date Submitted:
02/11/2010

Submitted to whom
Leo Francendese

Concurrence

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Approvals

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EPA Leo Francendese Date 3/3/10

consulted w/ TDEC

cc:

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- ☒ Jeff Gary, Jacobs
- ☒ Robert Pullen, Jacobs

Test Embankment Buttress Work Plan, Revision 0

1.0 Purpose of Work

This work plan is to install a buttress along the west and northwest corner of the Test Embankment to provide additional stability against shallow failure of the slopes. The buttress will be constructed out of ash or sand from the ballfield to the recommendations provided in the attached Stantec Letter Report dated February 2, 2010.

2.0 Design Components

The buttress shall be 50 feet wide and conform to the lines and grades indicated in the attached letter report and on the attached sketch. The construction of the berm will require the installation of a perforated pipe in the existing ditch to facilitate drainage. As a part of this work the general area around the edge of the Test Embankment will be filled with sand or ash to provide a more gentle slope to help in the minimization of erosion from this area.

The installation of the pipe will require personnel to be within the existing ditch. The use of spotters or other appropriate safety measures, as directed by Civil Projects safety professionals, will be used when working within the ditch.

The existing instrumentation in the area will be extended above the fill if required.

3.0 Construction Management

The construction will be accomplished with amphibious track hoes, dozers and trucks.

4.0 Schedule

The activity would begin as soon as possible and continue until the buttress has been completed.

5.0 Waste Management

No waste will be generated by this work.

6.0 Health and Safety

All construction activities will be done in accordance with the site-wide Health and Safety Plan.

REVISION NO.	DATE	DESCRIPTION	DRAWN BY	CHECKED BY	APPROVAL
A		ISSUED FOR CONSTRUCTION			

0 40' 80' 120'

SEAL

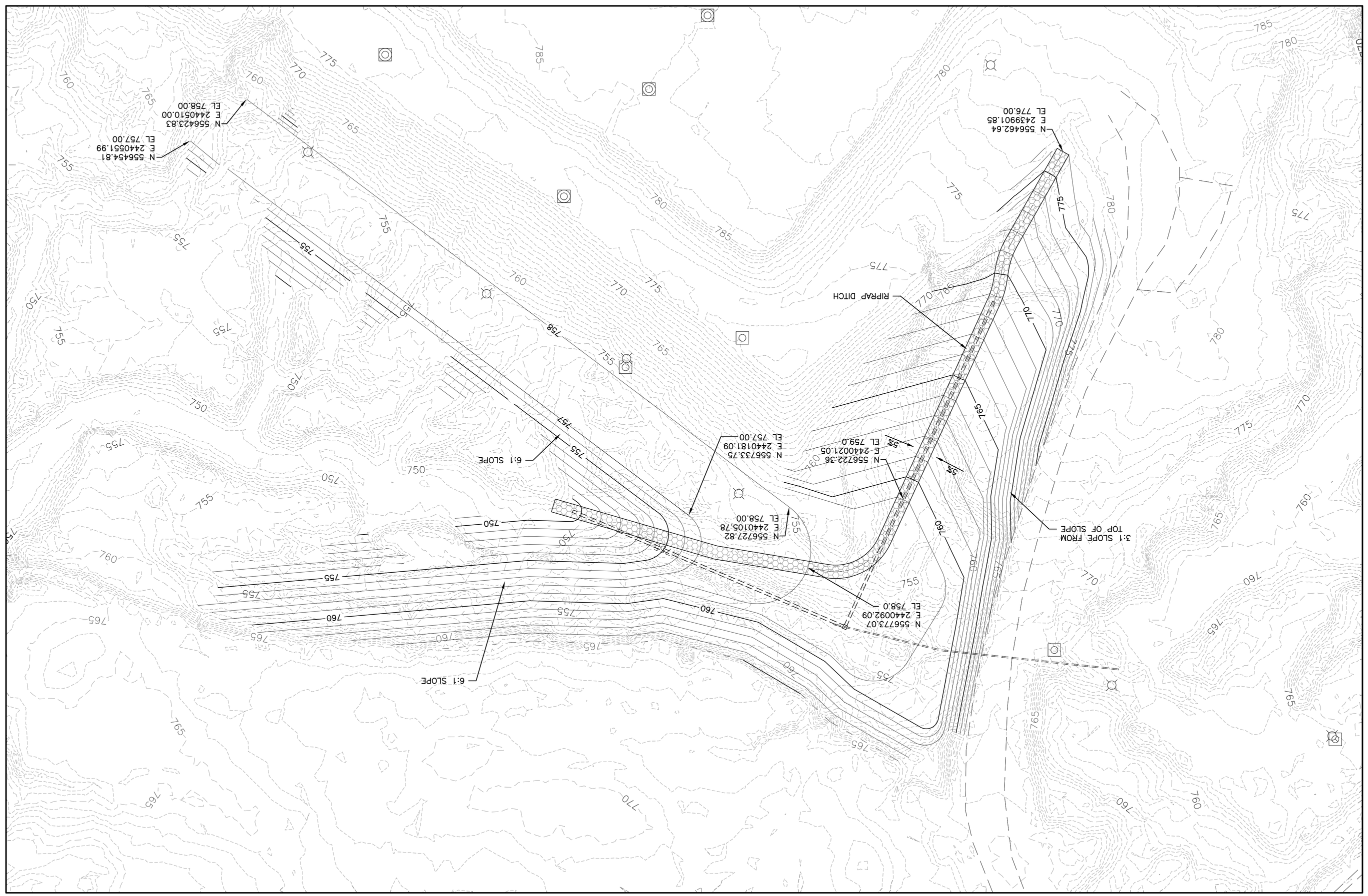
JACOBS

KINGSTON FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY

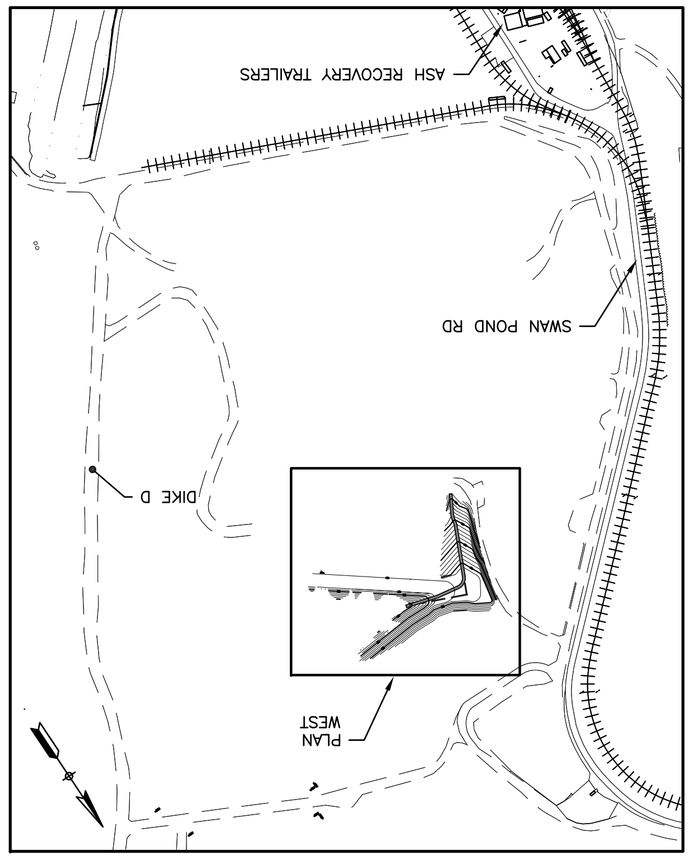
TEST EMBANKMENT
SAND BUTTRESS

SCALE: SHOWN | DRAWING NO. INF-SK-062 | REV. A

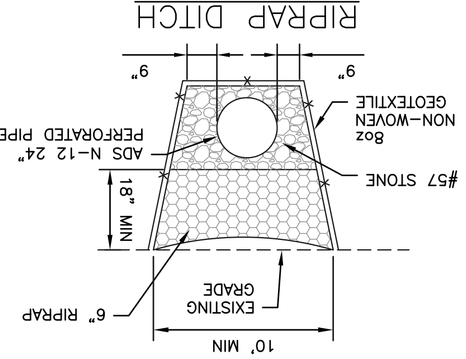
PLAN
SCALE: 1"=40'



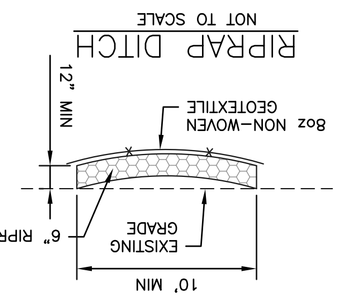
KEY PLAN
NOT TO SCALE



RIPRAP DITCH W/PERFORATED PIPE
NOT TO SCALE



RIPRAP DITCH
NOT TO SCALE



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February 2, 2010 let_010_kif_175669015

Mr. Michael T. Scott, PE
General Manager
TVA Kingston Fossil Ash Recovery Operations
1134 Swan Pond Road, KFP 1T-KST
Harriman, Tennessee 37748

Re: Updated Assessment and Recommendations to Improve Stability
Dredge Cell Test Embankment
Kingston Fossil Plant
Harriman, Roane County, Tennessee

Dear Mr. Scott:

This letter transmits results from updated slope stability analyses, recently completed to evaluate current factors of safety within the Kingston Dredge Cell Test Embankment. The analyses represent the height of fill and subsurface pore water pressures, as measured with the field instrumentation, on or about January 13, 2010.

The results indicate drained factors of safety greater than 1.5, as required by Tennessee Valley Authority (TVA) design criteria for long term conditions, on the three analyzed cross sections. However, on the western side of the embankment, factors of safety for undrained slope stability are inadequate (less than 1.3). This assessment is based on advanced, three-stage stability analyses for the potential of an undrained failure.

Data from the test embankment instrumentation continue to show pore pressure and displacement ratios that are below pre-established threshold or trigger limits. Data obtained from slope inclinometer surveys at the toe of the slope show the accumulation of measurable shearing strains across the depth of the underlying ash on the western half of the test embankment. The greater deformations in this area are consistent with the lower safety factors indicated by the stability analyses.

To address and improve slope stability, we recommend constructing a sand berm around the western end of the test embankment. The berm will average about 5 feet in thickness (maximum thickness of 7 feet), will be 50 feet wide at the top, and may be constructed from river sand (from ongoing dredging at the site). Until this berm is completed, filling operations should be directed to areas east of Section 2. Placement of fill may resume across the full

width of the test embankment once the recommended berm is completed. Construction of the test embankment may otherwise proceed, as long as monitoring is continued in accordance with the approved work plan.

With the recommended sand berm in place, analyses of the three cross sections indicate that the test embankment will meet current stability criteria through the end of planned construction. This conclusion is based on refined analyses and target factors of safety, which have been recently established (in collaboration with TVA and AECOM) to protect against the development of sudden undrained failures in saturated ash.

Details of these analyses, results, and recommendations are provided below. Supporting documentation is provided in the attached exhibits:

- Exhibit A Geometry of the Kingston Dredge Cell Test Embankment mid-January 2010
- Exhibit B Instrumentation Data Report from January 22, 2010
- Exhibit C Stability of Section 2 at Completion of Tier 2
- Exhibit D Recommended Sand Berm at the Western End of the Test Embankment
- Exhibit E Drained Embankment Stability with Recommended Sand Berm upon Completion of Tier 2
- Exhibit F Undrained Embankment Stability with Recommended Sand Berm upon Completion of Tier 2
- Exhibit G Undrained Embankment Stability with Recommended Sand Berm upon Completion of Tier 3

Background

The test embankment within the Kingston dredge cell is being constructed to obtain field data that will support design of the selected closure plan. The design, instrumentation, and monitoring plan for the test embankment are described in an approved document by Stantec Consulting Services Inc. (Stantec) dated June 3, 2009. Roughly 250,000 cubic yards of compacted ash are planned for placement to a height of about 45 feet, which corresponds to a peak elevation of about 800 feet. Depending on location, between 15 and 23 feet of fill material was placed within the test embankment between August 5, 2009, and January 14, 2010. Conditions within and beneath the embankment are currently being monitored using data from 32 vibrating wire piezometers (installed at eight locations), four settlement plates, and six slope inclinometer installations (additional inclinometers are being installed as the embankment is built).

On October 30, 2009, placement of fill in the test embankment was temporarily halted due to pore pressure ratios above pre-selected thresholds. At that time, Stantec completed an engineering evaluation of the instrument readings and analyzed the stability of the test embankment. These analyses, documented in a Stantec report dated November 9, 2009, indicated that adequate factors of safety existed for stability of the embankment slopes. The

high water pressures measured at that time appear to have been driven by other activities and drainage from adjacent areas of the site. The temporary stop in field operations and the re-evaluation of stability followed the written plans for the test embankment. Filling operations were allowed to resume on November 3, 2009; monitoring and evaluation of data from the on-site instrumentation have since continued in accordance with the test plan.

When completed, the planned test embankment will have three 15-foot thick tiers, with two 50-foot wide setbacks (benches) in the outslopes. As of January 13, 2010, the fill was just above the top of Tier 2. Based on survey data, the surface of the active fill varied between elevations of about 787 and 792 feet, with an average elevation of 790 feet. The topography of the test embankment at the completion of Tier 2 is depicted in Exhibit A.

Current Instrument Readings

Field data collected from the test embankment instrumentation are processed and reported (via FTP) weekly; the test embankment data report from January 22, 2010, is included here as Exhibit B. The instrumentation system, measurement locations, and data presentation have been previously described in the approved “Test Embankment Program” (Stantec, June 3, 2009) and the “Assessment of Current Stability” (Stantec, November 9, 2009). In the weekly report (Exhibit B), data were reported from 48 piezometers, six inclinometers, and four settlement plates. Inclinometers SI-7 and SI-8 were recently installed, but the data were not reported because elevations had not been established. Measurements from additional slope inclinometers and piezometers, installed to the north and east of the test embankment and shown on Exhibit A, are not reported as part of the test embankment program.

Based on the January 22 data from the piezometers, the computed pore pressure ratios are all below 10%, which is the lower “Action” threshold. The current displacement ratios, computed from slope inclinometers and settlement plate data, are also below the “Action” threshold of 20%. The thresholds (trigger limits) for the instrument readings were recently clarified in a January 6 letter distributed by Stantec. The variations in these ratios over time during construction of the test embankment are plotted in Exhibit B.

Looking at the lateral displacement profiles in Exhibit B, maximum deformations have been measured in SI-2 and SI-3, which are located at the toe of Tier 1 on the western half of the test embankment. At these two locations, downslope displacements (toward the north) approaching 0.30 to 0.35 inches have been recorded at roughly elevation 745 feet. The lateral deformations extend to the full depth of the soil and ash (top of the weathered shale is at about elevation 708 feet at SI-2 and 704 feet at SI-3), and appear to result from lateral squeezing and shearing within the foundation soils beneath the embankment. The inclinometer surveys do not show the development of defined failure planes, which would appear as relatively sharp transitions in the plots of displacement versus elevation.

The slope of the inclinometer data represents the magnitude of the shear strains in the soil:

$$\text{Shear Strain} = \arctan\left(\frac{\text{Lateral Displacement}}{\text{Depth Increment}}\right)$$

The maximum strains (maximum slope of displacement profile) have occurred in the saturated, hydraulically placed ash. At SI-2 and SI-3, the bottom of the ash deposits are at about elevations 724 and 727 feet, respectively. Above those levels, the shear strains in the ash are roughly 0.1% at both locations. These measured field strains have developed over time under drained conditions, and do not signal an imminent failure. However, the higher measured strains indicate higher shearing stresses within the ash along Sections 1 and 2, on the western side of the test embankment.

The data from SI-2 and SI-3 show that measurable deformations have accumulated beneath the embankment. The threshold or triggering limits for the instrument data were established to provide guidance for the field monitoring team. While the data are currently below all thresholds, it is prudent to evaluate the embankment stability based on engineering analysis.

Updated Stability Analyses

To assess the current stability of the test embankment, the analyses completed in November 2009 were updated. Two-dimensional, limit equilibrium stability calculations were completed using Spencer's method, as implemented in SLOPE/W (from Geo-Slope International) and UTEXAS4 (from Shinoak Software).

Detailed, vertical cross sections through the subsurface at Sections 1 through 4 (Exhibit A) were previously provided in the November 9 report. The subsurface stratigraphy is based on data from several geotechnical explorations completed at the site. Elevations for strata interfaces within the test embankment were obtained from boring logs collected during installation of the instruments.

Section 1 crosses the western end of the test embankment, on the bench at the top of Tier 1. With the addition of fill in Tier 2, Section 1 is no longer aligned with the weak axis (perpendicular to the maximum slope) on the western end of the embankment. Slope stability calculations for Section 1 would not accurately represent stability in this area; hence, previously reported stability evaluations for Section 1 were not updated here.

For the updated stability analyses, the current ground surface elevations along Sections 2 through 4 were defined using data from surveys completed on January 12, 2010. Because active filling operations are continuing, the top of the embankment was conservatively assumed to be level at elevation 790 feet in the calculations. This resulted in arbitrarily raising the ground surface about 1 to 3 feet at the crest of the slope in the three cross sections.

Pore water pressures throughout each cross section were estimated based on current data from piezometers in the test embankment and surrounding areas. Two piezometric lines were established for each cross section: an upper line for pore pressures within the existing

ash, and a second, lower line for the native foundation soils. The piezometric lines (depicted on the stability sections in Exhibits C, E, F, and G) were drawn to match the piezometer data recorded on January 13, 2010.

Updated Static Stability Criteria

Current TVA design criteria ("TVA Coal Combustion Products Management Program", URS, 23 October 2009) require a factor of safety of 1.3 or greater for slope stability during construction and 1.5 for long-term conditions. This criteria, also used by the U.S. Army Corps of Engineers (USACE EM 1110-2-1902)[1], has been adopted by TVA for the Kingston dredge cell project.

Recently, refinements to this stability criteria have been discussed for conditions where embankments are constructed over saturated ash. Given the significant strength loss past the peak undrained shearing resistance, which occurs at relatively low strains, analytical methods and minimum safety factors must be carefully considered. The refined design criteria was discussed and agreed to by TVA, AECOM, and Stantec at a meeting in Knoxville (December 16 and 17, 2009) and via teleconference (January 20, 2010). A memorandum is being prepared to document this refinement, and to better explain the rationale and proper application.

The updated, static stability criteria can be briefly described as follows. No distinction is made between interim and permanent slopes.

- **FSd ≥ 1.5**

  FSd is computed using effective stress methods, assuming fully drained conditions (steady state seepage) with measured or predicted pore water pressures. The value represents stability of the slope at any time when the pore pressures can be determined.

- **FSu ≥ 1.3**

  FSu represents stability with respect to undrained shearing capacity. For embankment construction, FSu is computed for the current embankment height prior to the placement of the next lift of fill. A total stress model of the soil shearing resistance is used, to predict the reduced capacity available if an undrained failure is triggered.

- **FSul ≥ { 2FSu / (1 + FSu) }**

  FSul is obtained from a total stress, undrained analysis of an embankment and represents conditions just after the next lift of fill is placed. This value is intended to limit the average, incremental increase in shear stress along a slip surface to be less

[1] U.S. Army Corps of Engineers (2003). "Slope Stability". No. EM 1110-2-1902, October 31.

than one half of the incremental shear stress required to reach the peak undrained strength. The value of FSul can be used to determine the maximum safe thickness for the next embankment lift.

The value of FSd is computed using conventional, effective stress methods of analysis and strength characterization. FSu and FSul can be computed using the three-stage stability analysis method adopted by the U.S. Army Corps of Engineers for rapid drawdown analyses (USACE 2003, EM 1110-2-1902, Exhibit G)[1]. In the three-stage method, a total stress model of shearing resistance is determined from the anisotropic consolidation stresses. Measured or predicted pore water pressures are used in the first (consolidation) stage of the analysis. The undrained soil strengths are estimated based on linear interpolation with the principle stress ratio during consolidation (described below). The shear resistance along a given potential failure surface is computed in three stages of the calculation:

- Stage 1. Use an effective stress analysis to compute the normal and shear consolidation stresses at each point on the failure plane.
- Stage 2. Determine the undrained, total stress shearing resistance along the failure plane. Based on the consolidation stresses from Stage 1, use the normal consolidation stress and linear interpolation between limiting strength envelopes to estimate the shearing resistance along the slip surface.
- Stage 3. Compute the undrained factor of safety using the lesser of the drained and undrained shear strengths (from Stages 1 and 2) at each point along the slip surface.

This method of analysis is described in EM 1110-2-1902 (USACE 2003)[1] and Duncan and Wright (2005)[2], and is available within UTEXAS4.

Material Properties

The unit weights and strength properties of each material were previously determined. These properties, and the supporting laboratory and field data, are documented in a draft report prepared by Stantec. That report was submitted to TVA, subjected to review by TVA and AECOM, and discussed at a design team meeting on October 8, 2009. Based on those discussions and subsequent review comments, there is a general consensus that the engineering parameters listed in Table 1 and Table 2 are appropriate for representing the ash materials and foundation soils within the Kingston dredge cell. These properties were thus adopted for this analysis of the test embankment stability.

As discussed below, a berm of material is recommended for improving the stability of the test embankment. The berm may be constructed of loosely compacted river sand, obtained from ongoing dredging work at the site. This material was assumed to have a unit weight of 110 pcf and a friction angle of 30 degrees, as listed in Table 1.

In the three-stage undrained stability analysis, an interpolation scheme is used to determine the undrained shear strength of anisotropically consolidated soils. The interpolation is based

[2] Duncan, J. M., and Wright, S. G. (2005). *Soil Strength and Slope Stability.* John Wiley & Sons.

on two limiting strength envelopes, representing a fully drained strength and the undrained strength of an isotropically consolidated soil sample. Both of these input envelopes represent a relationship between the shear strength and the effective normal consolidation stress on the failure plane. The envelopes correspond to effective principal stress ratios (K_c=σ'$_1$/σ'$_3$) at consolidation of K_f and 1, respectively, and are defined by an intercept and a slope. The envelope corresponding to K_c=K_f is identical to the conventional effective stress shear strength envelope. Thus, its intercept ($d_{Kc=Kf}$) is the same as the effective stress cohesion value (c') and its slope ($\psi_{Kc=kf}$) is the same as the effective stress friction angle (ϕ'). The K_c=1 envelope can be derived from the total stress cohesion value (c) and the total stress friction angle (ϕ), as determined from conventional CU triaxial compression tests. When c and ϕ are obtained from a line drawn tangent to the total stress Mohr's circles, the relationships among the intercept ($d_{Kc=1}$) and slope ($\psi_{Kc=1}$) of the K_c=1 envelope, the total stress c and ϕ, and the effective stress ϕ' are (Duncan and Wright 2005):

$$d_{k_c=1} = c\left(\frac{\cos\phi\cos\phi'}{1-\sin\phi}\right)$$

$$\psi_{k_c=1} = \tan^{-1}\left(\frac{\sin\phi\cos\phi'}{1-\sin\phi}\right)$$

Values of $d_{Kc=1}$ and $\psi_{Kc=1}$ for the test embankment soils are listed in Table 2.

Current Stability

Updated stability analyses for the Kingston test embankment were completed for Sections 2, 3, and 4. Values of FSd and FSu computed for Sections 3 and 4 meet the design criteria (those results are reported in later sections of this letter). For a level embankment at elevation 790 feet, the drained stability analyses gave FSd = 1.65 on Section 2, while the undrained, three-stage stability analysis indicated FSu = 1.19. These results are presented in Exhibit C; the critical failure mechanism is a circular slide through the saturated ash at the toe of the slope. The results from the undrained analyses (FSu = 1.19) show that Section 2 does not currently meet criteria (FSu ≥ 1.3) for stability.

The lower stability at Section 2 can be understood by considering the topography shown on Exhibit A. On the western half of the test embankment, the ground surface north of the slope toe dips toward a drainage channel. Compared to Sections 3 and 4, the embankment height is effectively greater on Section 2, which increases the static shear stresses and lowers the safety factor. Deformation data from the slope inclinometers supports this observation. Furthermore, this suggests that the stability of Section 2 can be improved by constructing a relatively small berm along the slope toe.

Recommended Sand Berm

To stabilize Section 2, a toe berm is recommended for the western half of the test embankment. The dimensions of the berm were selected so that the test embankment, from now through planned completion, should meet the refined stability criteria outlined above. The analyses and computed safety factors are presented below.

For the stability analyses, the berm was assumed to consist of loosely compacted sand. The assumed strength (Table 1) is not critical to the results, as the weight of berm (not its strength) provides the needed stabilizing force. The berm material may be constructed using river sand obtained from ongoing dredging work at the site. Stronger materials, such as crushed stone, could be substituted for the river sand without reducing the effectiveness of the berm.

A plan drawing and a typical cross section through the recommended berm are presented in Exhibit D. The top of the berm should extend 50 feet laterally, and slope from an elevation of 758 feet at the face of the test embankment to 757 feet at the crest of the outslope. The berm will average about 5 feet in thickness, with a maximum thickness of about 7 feet. The outslope of the berm should be constructed at a slope no steeper than 5:1 (horizontal:vertical). The berm should wrap around the western end of the test embankment, and tie into existing grade at both ends.

Site drainage around the test embankment and berm has not been addressed here. The sand berm, as shown in Exhibit D, will block a drainage ditch around the northwest corner of the test embankment. Drainage must be maintained, which can be accomplished by shifting the drainage ditch 100 feet (or more) to the north. Erosion control measures that have been recently proposed by Jacobs Engineering Group, Inc. (Jacobs) along the western perimeter of the test embankment will also need modification. Revised layouts for the drainage and erosion control measures should be developed by Jacobs, in association with formalizing plans for constructing the sand berm.

The western side of the test embankment currently does not meet stability criteria (analyses of the existing slope indicate FSu = 1.19). As the berm material is gradually placed, this part of the test embankment should become increasingly more stable. An undrained, three-stage stability analysis was completed to check if the criteria for FSul would be met during construction. The sand berm was treated as a distributed load at the toe of the slope, and the results indicate an acceptable value of FSul = 1.33 (see Exhibit D). With the berm in place, the embankment will exhibit higher safety factors (FSd and FSu), as reported below.

Caution and careful planning should be exercised during construction of the recommended modifications. Placement of fill at the toe will improve stability, so construction equipment should operate only from the top of the berm as it is built. Given the modest final thickness, we recommend building the berm to full height starting at one or both ends, and progress by filling material ahead of completed sections. Recognizing that the stabilizing force is obtained from the weight of the fill, we have not recommended specific restrictions for building the berm in lifts or for compaction of the added fill.

Excavation of drainage ditches should follow completion of the berm. This will avoid conditions where the effective height of the slope is increased prior to having the stabilizing weight of the berm in place. All construction work plans, including sequencing, should be reviewed and approved by TVA safety professionals.

Filling operations on top of the western portion of the test embankment (area of Section 2) should not resume prior to completion of the recommended sand berm as shown in Exhibit D.

Drained Stability Analyses for Completion of Tier 2

Evaluation of the drained safety factors (FSd) was accomplished using SLOPE/W. Because the surficial materials have no cohesion (c' = 0), an infinite slope mechanism (shallow sloughing of material parallel to the slope face) yields the lowest factor of safety in all cases. Stability calculations for shallow failures were completed by specifying a minimum sliding depth of 1 to 2 feet. The resulting FSd ranged from 0.9 to 1.6 for the three cross sections. Observed at various times in the field, these failures pose little risk to the overall embankment or the safety of construction activities.

The primary concern for stability in the test embankment is the potential to trigger a deep failure, along surfaces that would pass through the bottom of the ash deposits. If a deep failure were to develop in the test embankment, there is a potential for triggering a retrogressive failure that could destabilize the relic areas. The deep sliding mechanism would thus have the worst possible consequences.

Results from the drained stability analyses, for the deep sliding mechanism, are presented in Exhibit E and summarized in Table 3. These analyses assume the test embankment is completed to the top of Tier 2 at a level elevation of 790 feet, corresponding roughly to the currently existing conditions. With the recommended sand berm in place, all three cross sections exhibit values of FSd > 2, well in excess of the required minimum value of 1.5.

Undrained Stability Analyses for Completion of Tier 2

Three-stage, undrained stability analyses were completed using UTEXAS4. The calculations modeled the existing geometry of the test embankment with Tier 2, a level top surface at elevation 790 feet, and the recommended sand berm in place. Piezometric levels were modeled based on the January 13 field measurements. Here, there was no additional distributed load or changes in piezometric level in the different stages of the undrained stability calculation. The output results are presented in Exhibit F and summarized in Table 3.

On all three cross sections, the critical undrained slip surface was found in the 3:1 outslope of Tier 1, at the toe of the test embankment fill. The sand berm increases the undrained stability to FSu = 1.35 on Section 2. The lowest computed value of FSu is 1.31 on Section 3, essentially equal to the design minimum value of 1.3. Hence, with the recommended sand berm in place along the toe on the western side, these analyses indicate that the test embankment will meet criteria for undrained stability.

Undrained Stability Analyses for Completion of Tier 3

The stability of the test embankment with continued construction was checked with the three-stage, undrained analyses presented in Exhibit G. The addition of Tier 3 was modeled as a uniformly distributed load of 1,199 psf (11 feet of fill at a unit weight of 109 pcf), set back 50 feet from the crest of Tier 2. The resulting safety factor for each cross section corresponds to FSul, and represents the expected stability if Tier 3 was rapidly placed in one lift. The results, summarized in Table 3, are all well above the minimum criteria values.

For all three analyzed cross sections, the computed values of FSu = FSul (Table 3). Comparison of the output in Exhibits F and G show that the critical slip surface (the one with the lowest safety factor) occurs at the toe of the slope, in all cases, and is not affected by the Tier 3 load. Hence, failure mechanisms that would engage the added fill in Tier 3 all have higher safety factors. This was verified in trial analyses where higher surface pressures were specified until the critical failure mechanism, with a lower FSul, shifted to the upper slope.

Conclusions and Recommendations

The stability of the test embankment being constructed in the Kingston dredge cell has been re-evaluated. Fill heights and piezometric conditions in mid-January 2010, were analyzed. All instrument readings were below established threshold limits, and there is no known evidence of excessive deformation or instability. However, limit equilibrium slope stability analyses demonstrate that the western side of the test embankment does not meet current, refined criteria for the factor of safety against an undrained failure.

Until this condition is mitigated, no additional fill should be placed in the vicinity of Section 2 (western side of the test embankment). To improve stability, we recommend that a berm of sand should be constructed along the toe of the test embankment. The berm is depicted on the drawings in Exhibit D.

- The berm may be constructed using river sand obtained from the ongoing dredging work at the site. Stronger materials, such as crushed stone, could be substituted for the sand without reducing the effectiveness of the berm.

- The berm will average about 5 feet in thickness, with a maximum thickness of about 7 feet. The top of the berm should extend 50 feet laterally, and slope from an elevation of 758 feet at the face of the test embankment to 757 feet at the crest of the outslope. The outslope of the berm should be constructed at a slope no steeper than 5:1 (horizontal:vertical).

- The berm should wrap around the western end of the test embankment, and tie into existing grade at both ends.

- The sand berm, as shown in Exhibit D, will block a drainage ditch around the northwest corner of the test embankment. Drainage needs to be maintained, which can be accomplished by shifting the drainage ditch 100 feet (or more) to the north.

Revised layouts for the drainage and erosion control measures should be developed by Jacobs.

- Placement of fill at the toe will improve stability, so construction equipment should operate only from the top of the berm as it is built. Excavation of drainage ditches should follow completion of the berm, to avoid conditions where the effective height of the slope is increased prior to having the stabilizing weight of the berm in place.

- Caution and careful planning should be exercised during construction of the recommended modifications. All construction work plans should be reviewed and approved by TVA safety professionals.

Filling operations on top of the western portion of the test embankment (area of Section 2) should not resume prior to completion of the recommended sand berm as shown in Exhibit D. Once the berm is completed, construction may proceed across the full width of the test embankment.

Continued diligence is needed to provide for the safe completion of the test embankment. Monitoring of the test embankment instrumentation must continue as additional fill is placed, in accordance with the approved work plan. Additional, periodic updates to the stability analyses may be warranted, especially if the instrumentation records high pressure or displacement ratios.

Please let us know if you have additional questions.

Sincerely,

STANTEC CONSULTING SERVICES INC.

Alan Rauch

Alan F. Rauch, PhD, PE
Geotechnical Engineer

Michael J. Steele, PE
Associate

/rdr

LEX

Table 1. Unit Weights and Drained Strengths used in the Stability Analyses

Material	Unit Weight		Drained, Effective Stress Strength Parameters	
	γ_m (pcf)	γ_{sat} (pcf)	ϕ' (deg.)	c' (psf)
Landfilled Ash	109	111	30	0
Hydraulically Placed Ash	100	107	25	0
Sensitive Silt/Clay	100	107	28	0
Lean Clay Foundation Soil	--	130	32	0
Sandy Silt to Silty Sand	--	128	30	0
Coarse Stone	115	128	38	0
Sand Berm	110	--	30	0

Table 2. Undrained Strengths used in the Stability Analyses

Material	Undrained, Total Stress Strength Parameters			
	ϕ (deg.)	c (psf)	$\psi_{Kc=1}$ (deg.)	$d_{Kc=1}$ (psf)
Landfilled Ash	**	**	**	**
Hydraulically Placed Ash	10	0	10.78	0
Sensitive Silt/Clay	10*	0*	10.51	0
Lean Clay Foundation Soil	24	0	30.17	0
Sandy Silt to Silty Sand	12	1,000	12.81	1069.45
Coarse Stone	**	**	**	**
Sand Berm	**	**	**	**

* Strength conservatively assumed equal to that of hydraulically placed ash for the analyses reported herein. Design parameters are S = 0.24 and m = 0.71; assuming OCR = 1, c = 0, and measurement in ICU triaxial tests, the equivalent friction angle would be ϕ = 11.2°.

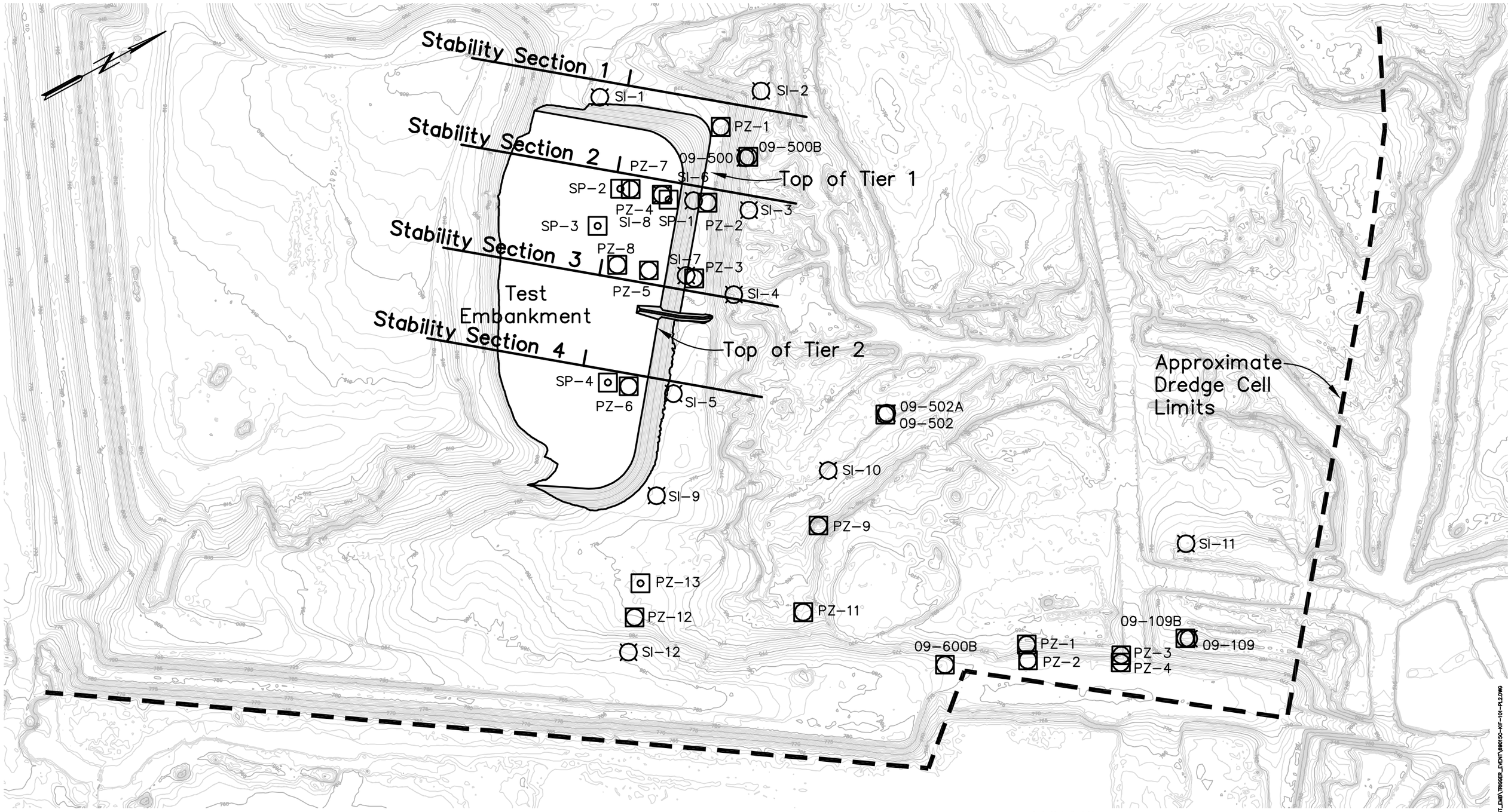
** These materials were assumed to be fully drained or unsaturated in the analyses reported herein; strengths for undrained analyses were thus characterized using ϕ' and c'.

Table 3. Summary of Results from Stability Analyses for Deep Failures, after Completion of the Sand Berm at the Western End of the Test Embankment

Conditions Analyzed	Minimum Safety Factor per Criteria	Computed Safety Factor
Drained Analysis Completion of Tier 2, Top = Elev. 790 ft (see Exhibit E)	**FSd**	
Embankment Section 2	1.5	2.03
Embankment Section 3	1.5	2.20
Embankment Section 4	1.5	3.10
Undrained Analysis Completion of Tier 2, Top = Elev. 790 ft (see Exhibit F)	**FSu**	
Embankment Section 2	1.3	1.35
Embankment Section 3	1.3	1.31
Embankment Section 4	1.3	1.37
Undrained Analysis Completion of Tier 3, Top = Elev. 801 ft (see Exhibit G)	**FSul**	
Embankment Section 2	1.15	1.35
Embankment Section 3	1.13	1.31
Embankment Section 4	1.16	1.37

Exhibit A

Geometry of the Kingston Dredge Cell Test Embankment mid-January 2010



LEGEND

STN-1 Slope Inclinometer (SI)
STN-2 Piezometer (PZ)
STN-3 Settlement Platform (SP)

TEST EMBANKMENT – STABLITY SECTIONS

SCALE: 1"=200'

69015c-kif-101-pl1.DWG

100 0 200 400 FEET

GRAPHIC SCALE: 1" = 200'

NOTES:

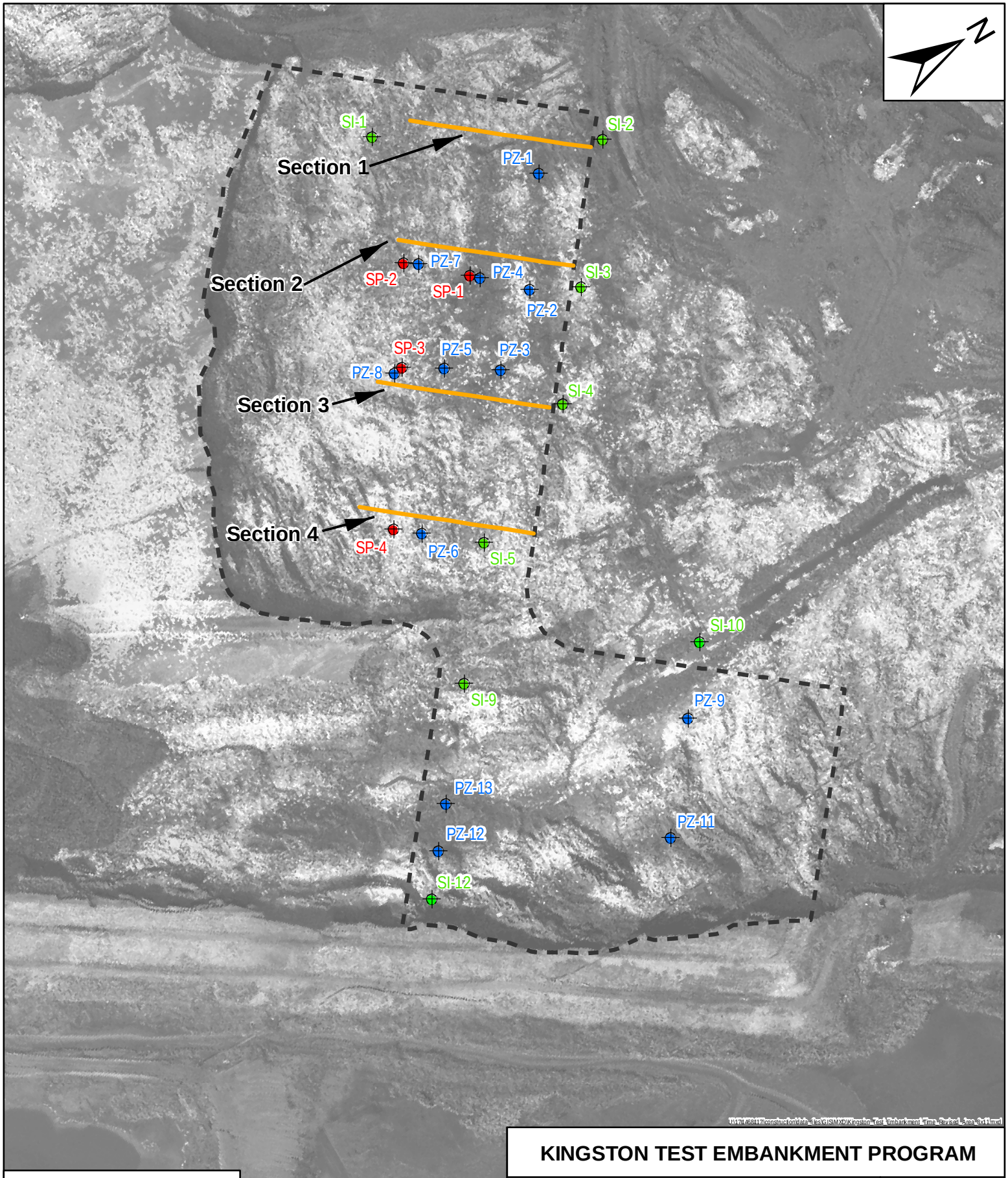
1. Topographic mapping was generated from a LIDAR survey performed on Nov., 25, 2009 by Tuck Mapping Solutions, Inc.
2. Topographic information on the Test Embankment depicts the approximate surface configuration in mid-January, 2010.

A-1

PLOT DATE: 01/25/2010 USER: GRAHAM, DAVE
W:\750-WOTRC\750690015 ENVIRONMENTAL\DRAWING\TEST_EMB\TRIGGER_EVENT\69015C-KIF-101-PL2.DWG

Exhibit B

Instrumentation Data Report from January 22, 2010



Legend

- Cross Section
- Piezometer (PZ)
- Slope Inclinometer (SI)
- Settlement Plate (SP)

KINGSTON TEST EMBANKMENT PROGRAM

Stantec

Stantec Consulting Services, Inc.
100 Westwood Place
Suite 420
Brentwood, TN 37027

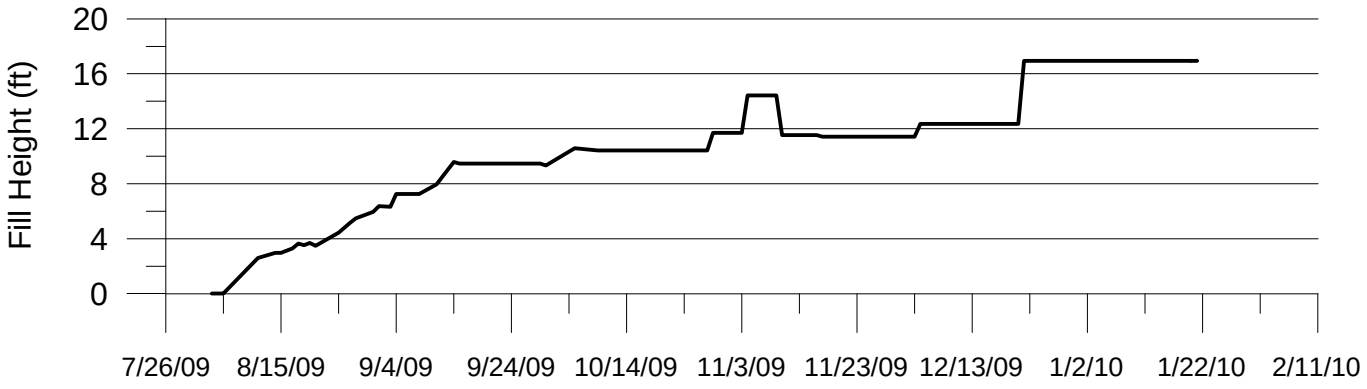
DRAWN BY:	MEP	PROJECT NO.	175669015	DATE:	November 8, 2009
CHECKED BY:	JWA	SCALE:	1" = 200'		

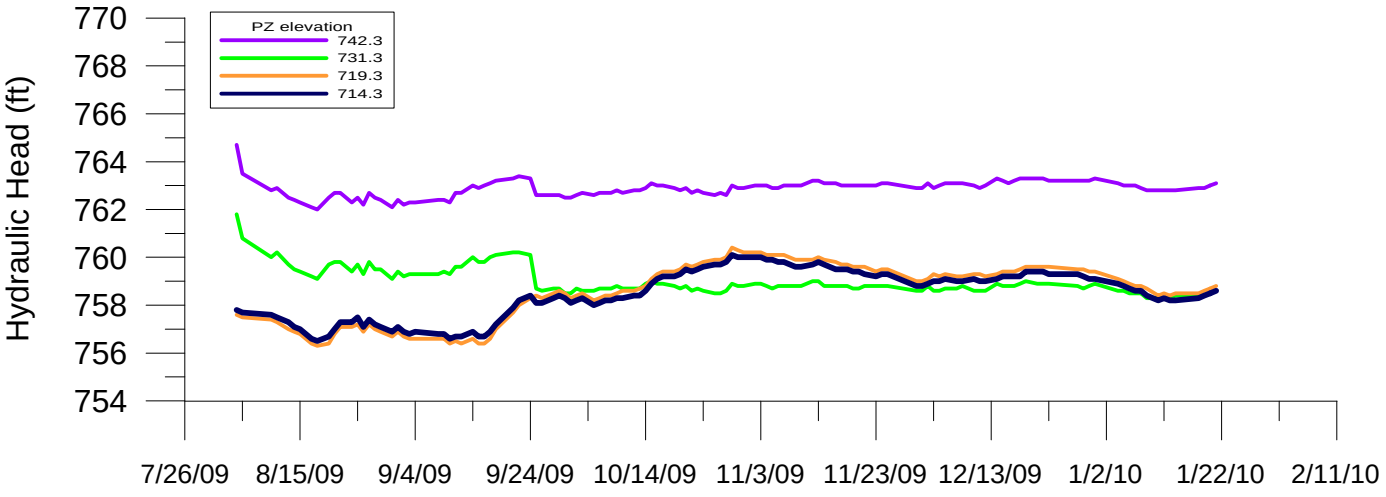
Stantec

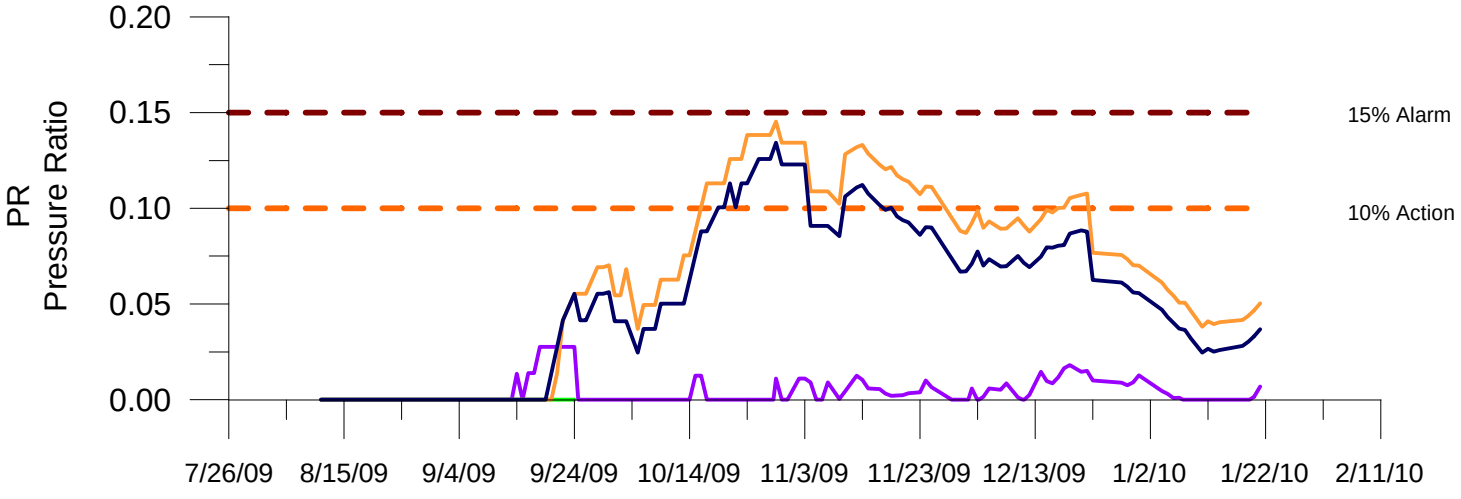
Kingston Test Embankment
175669015

Survey Section 1

PZ-1







* PR(Pressure Ratio) = △U/△Z

* △U= Change in Pore Pressure (psf)

* △Z= Fill Height x 110 (pcf)

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Kingston Test Embankment
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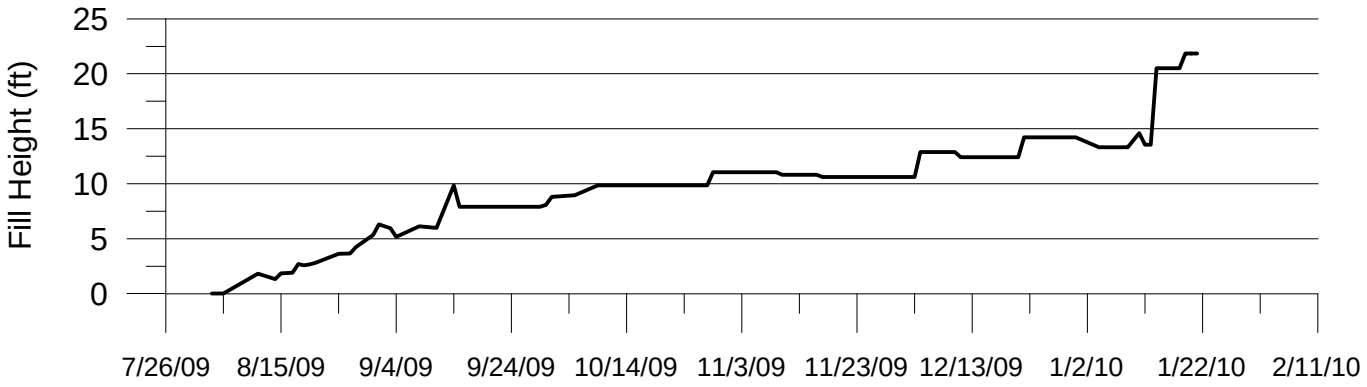
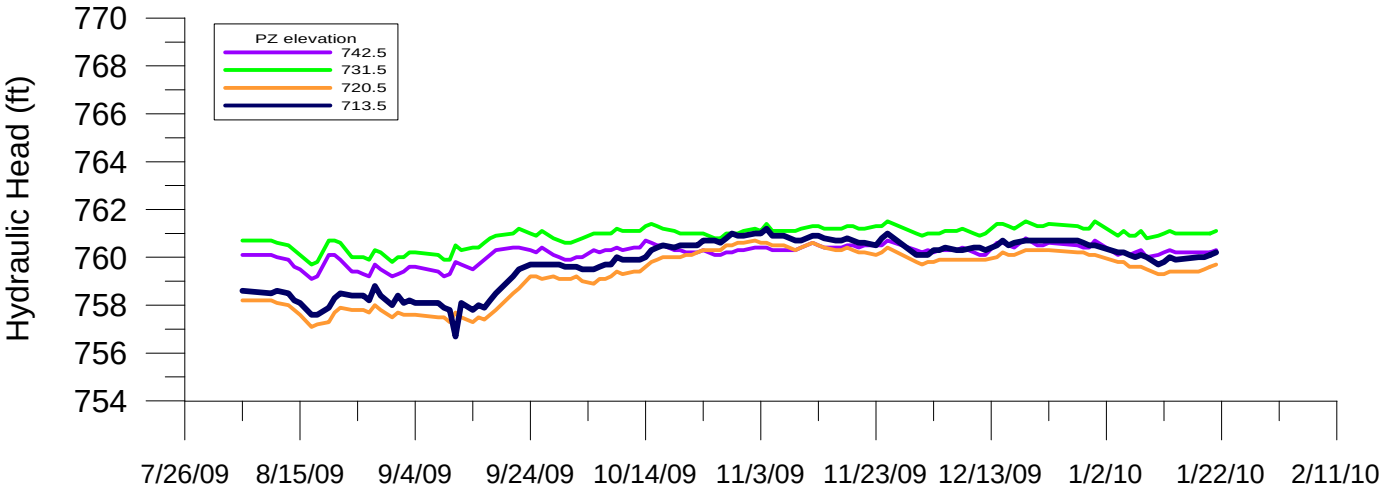
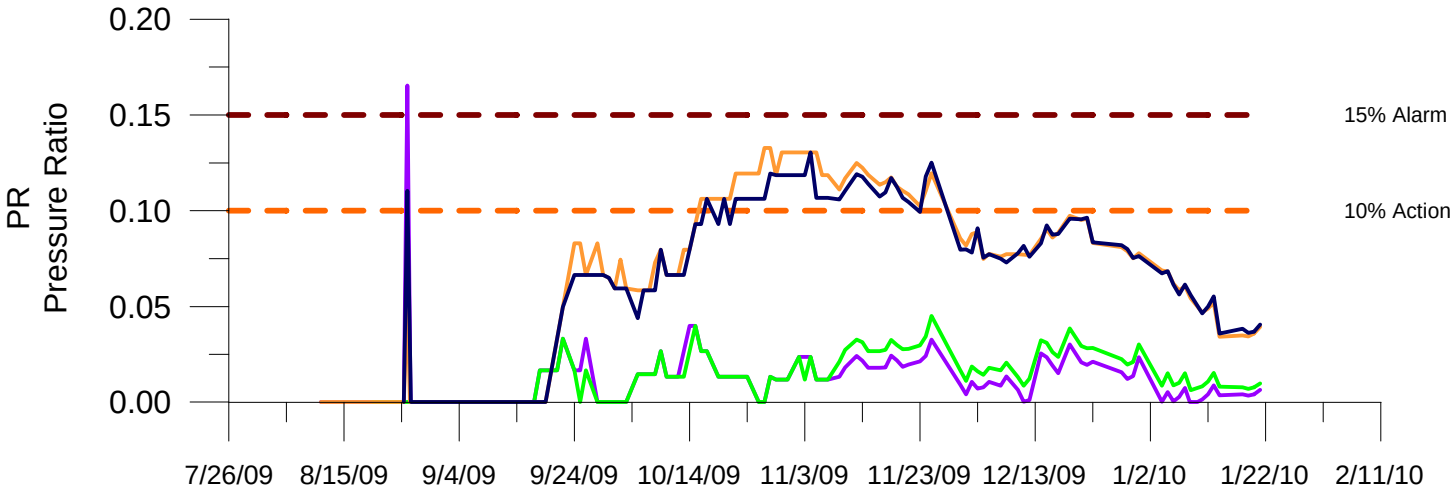
Survey Section 2

PZ-2







* PR(Pressure Ratio) = △U/△Z

* △U= Change in Pore Pressure (psf)

* △Z= Fill Height x 110 (pcf)

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Kingston Test Embankment
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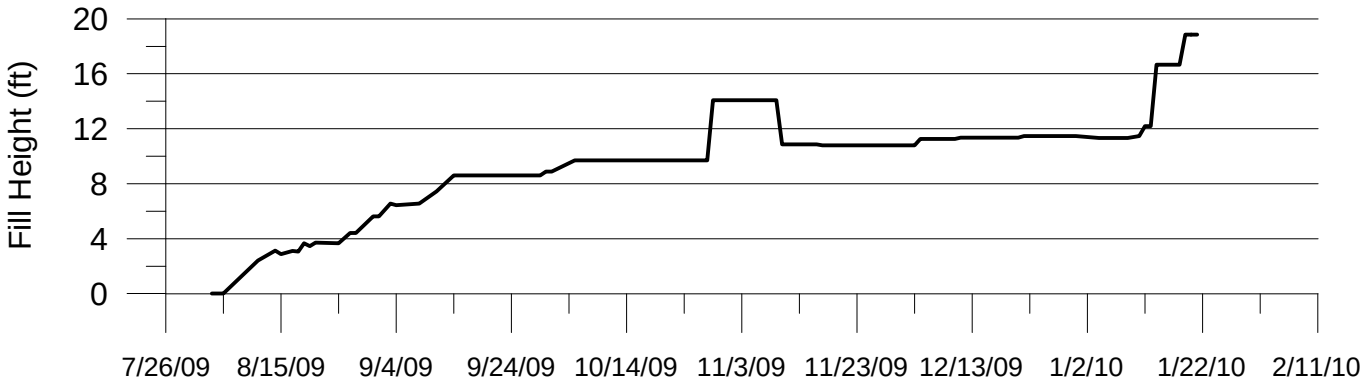
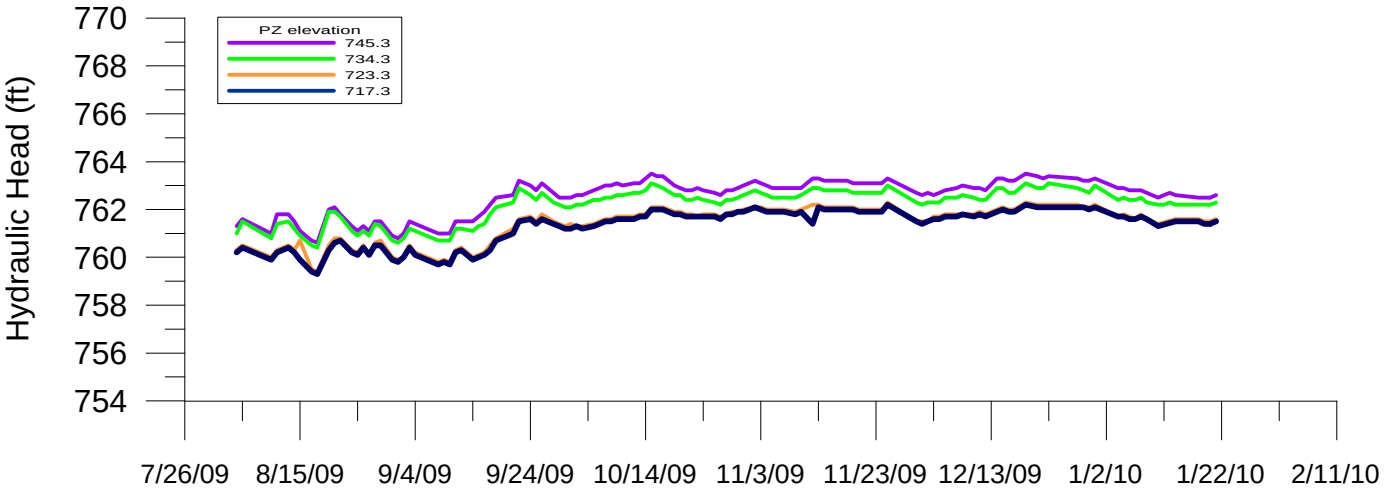
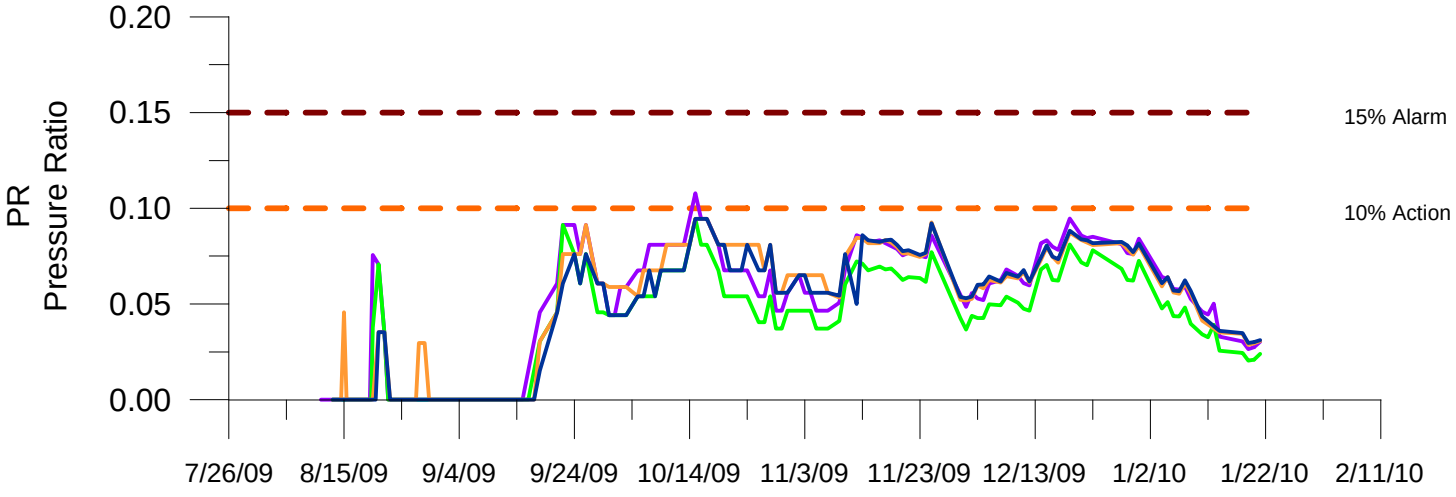
Survey Section 3

PZ-3







* PR(Pressure Ratio) = △U/△Z

* △U= Change in Pore Pressure (psf)

* △Z= Fill Height x 110 (pcf)

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Survey Section 2

Kingston Test Embankment
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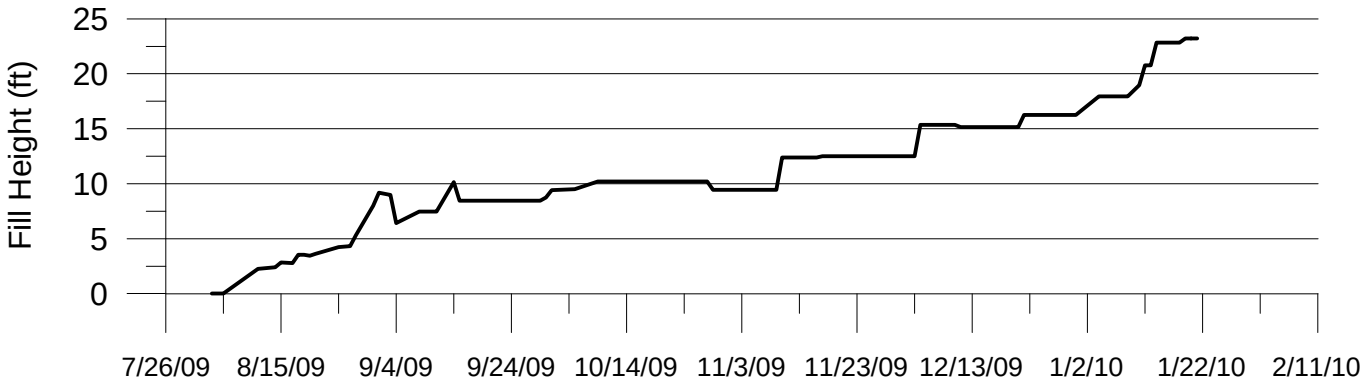
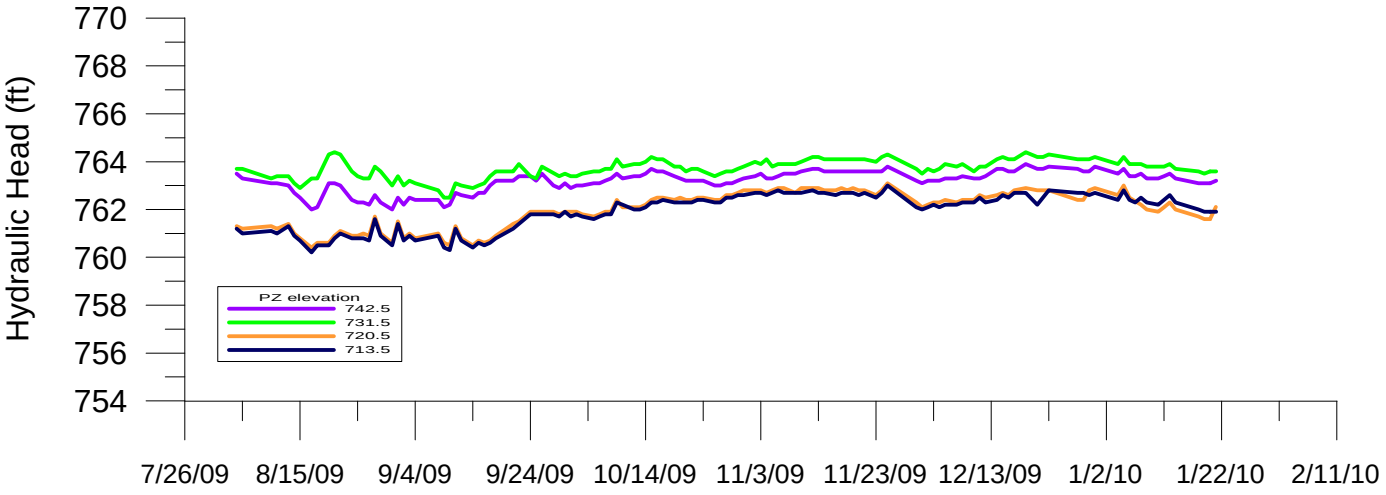
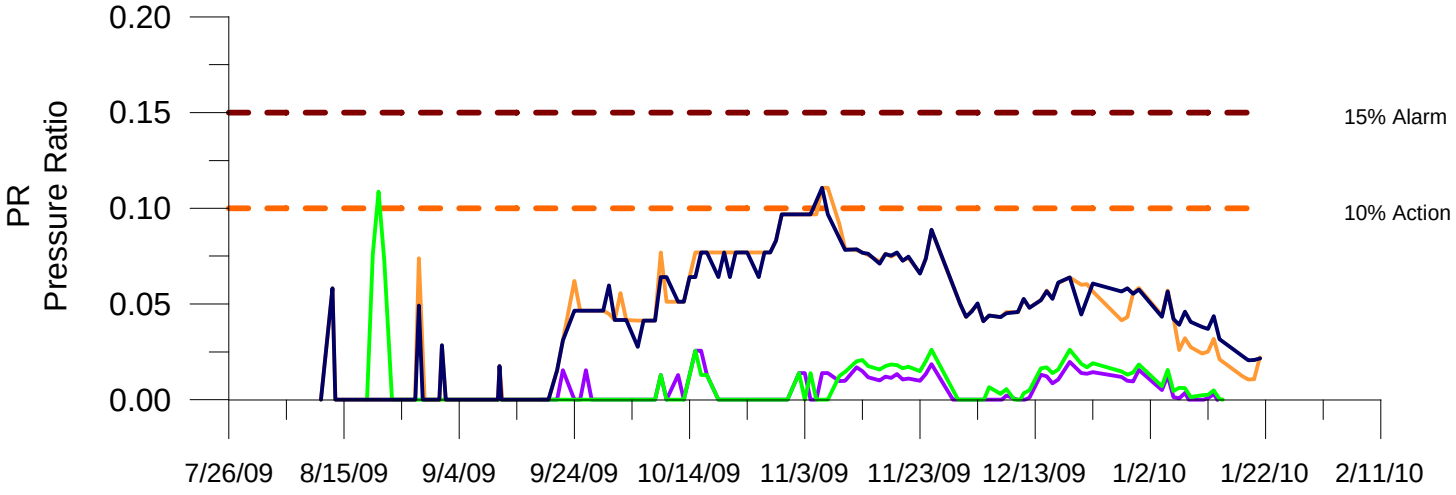
PZ-4







* PR(Pressure Ratio) = △U/△Z

* △U= Change in Pore Pressure (psf)

* △Z= Fill Height x 110 (pcf)

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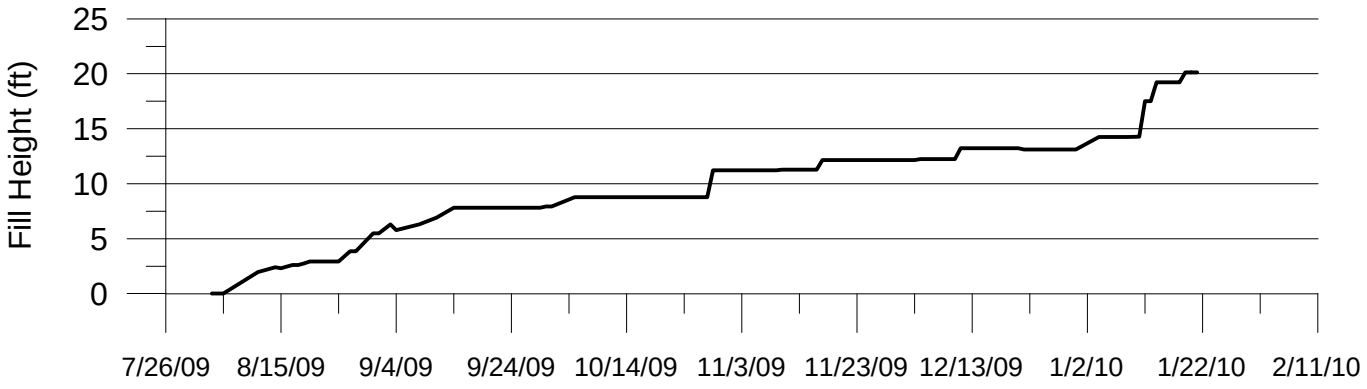
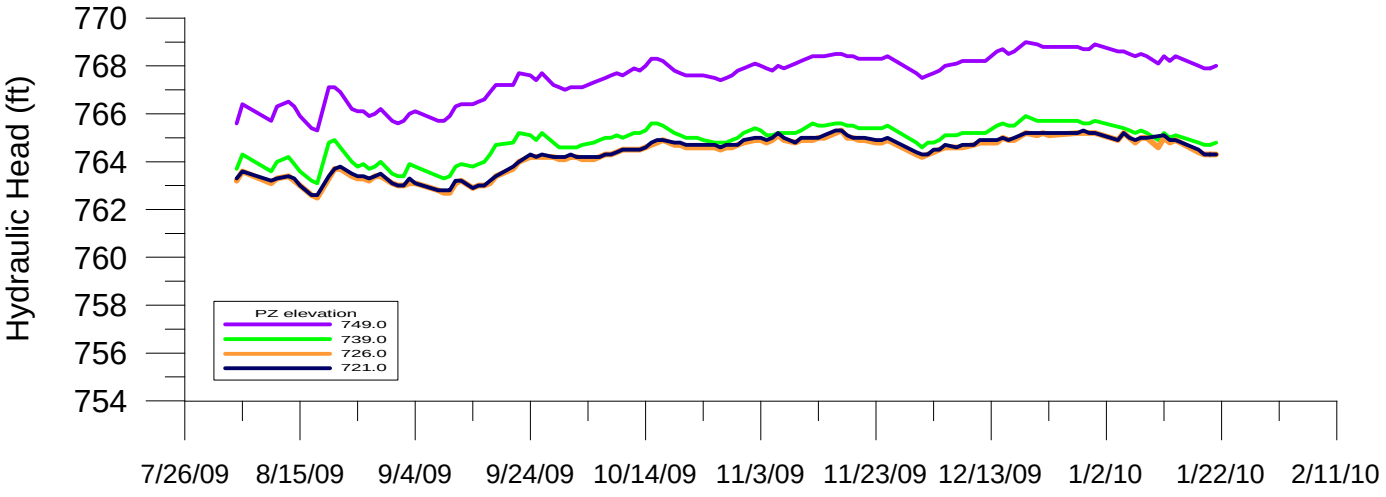
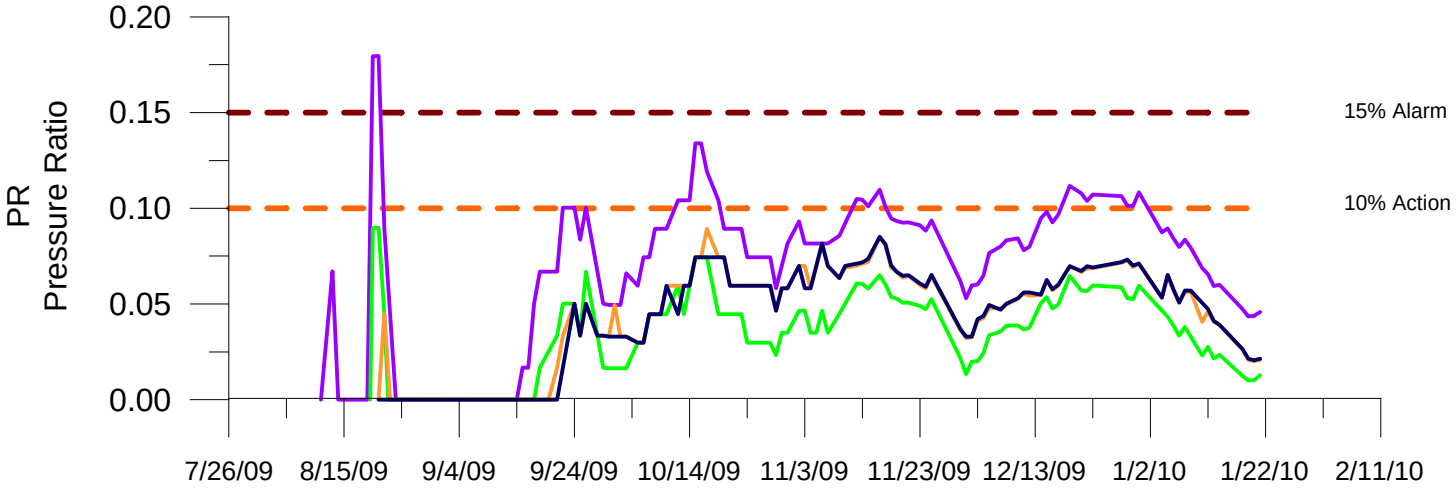
Survey Section 3

PZ-5







* PR(Pressure Ratio) = △U/△Z

* △U= Change in Pore Pressure (psf)

* △Z= Fill Height x 110 (pcf)

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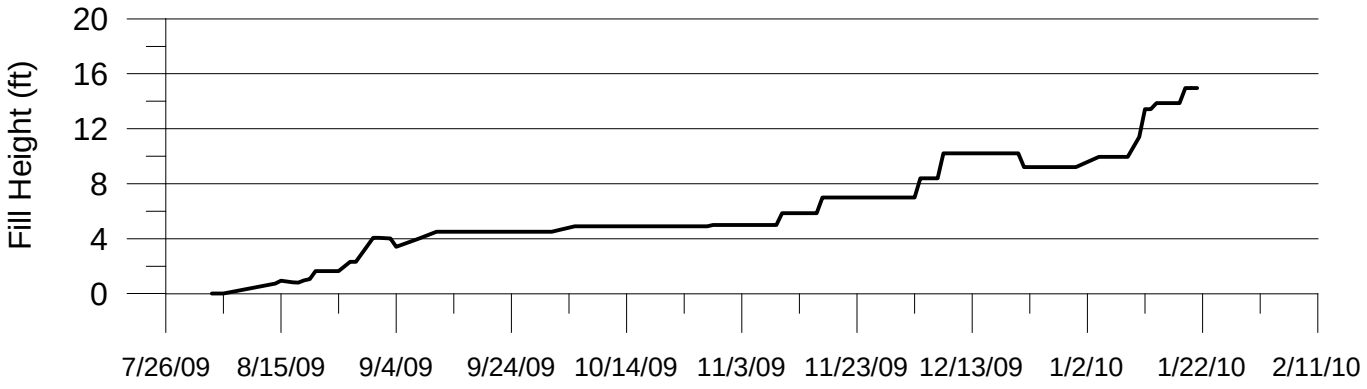
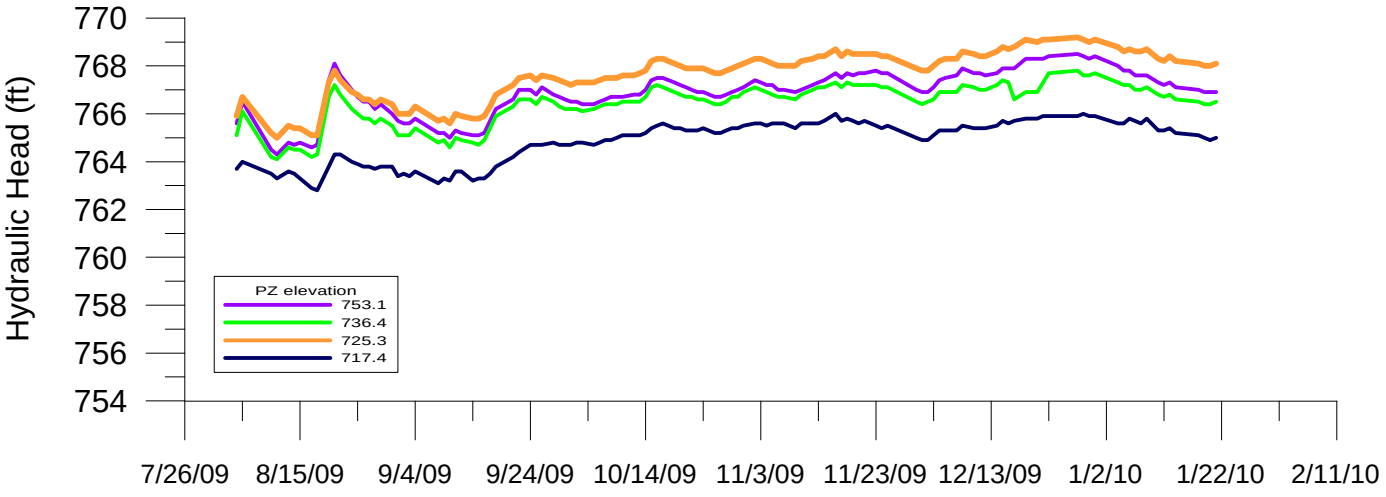
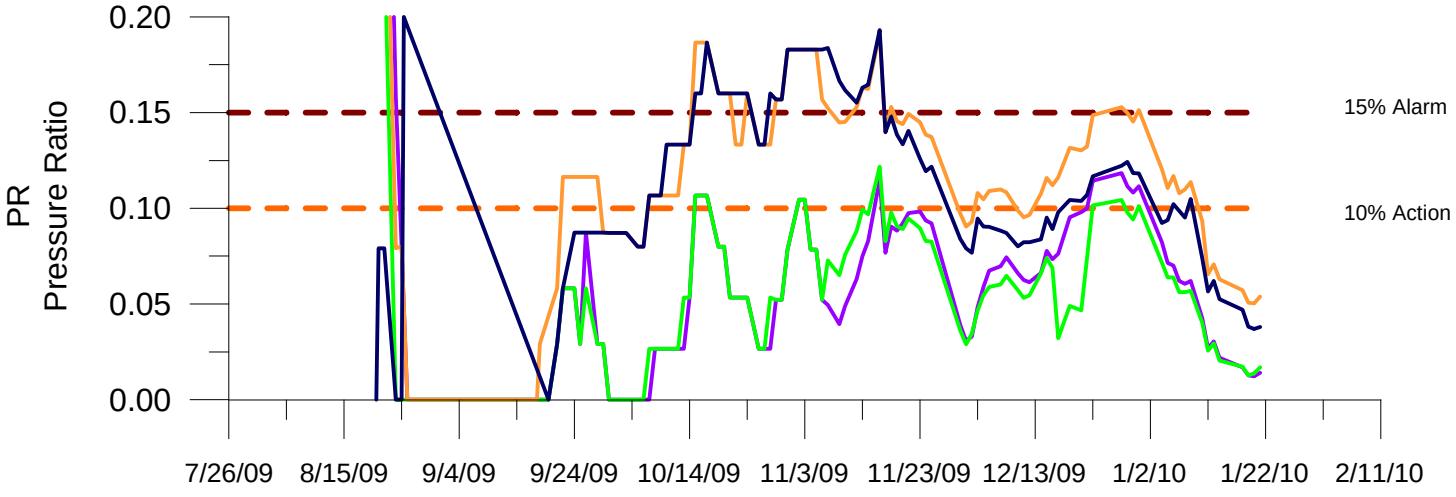
Survey Section 4

PZ-6







* PR(Pressure Ratio) = △U/△Z

* △U= Change in Pore Pressure (psf)

* △Z= Fill Height x 110 (pcf)

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Kingston Test Embankment
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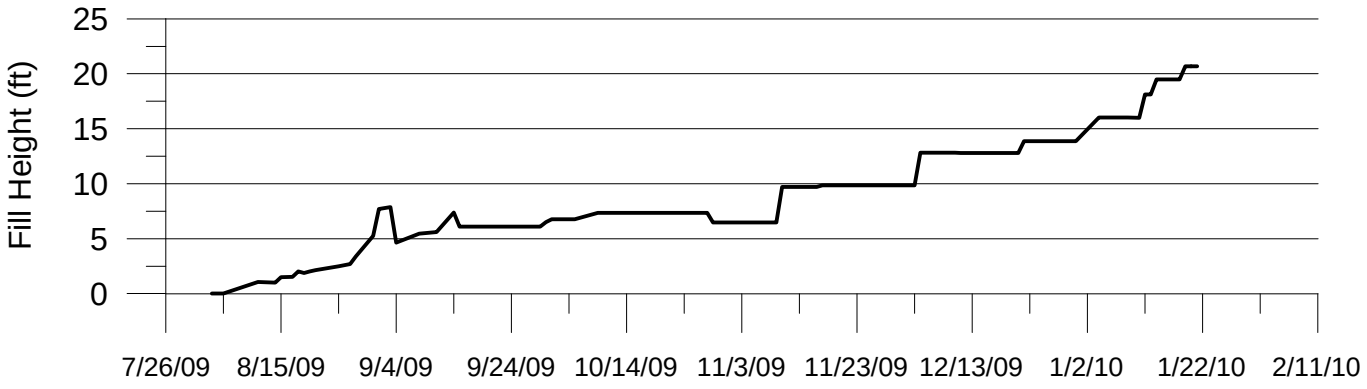
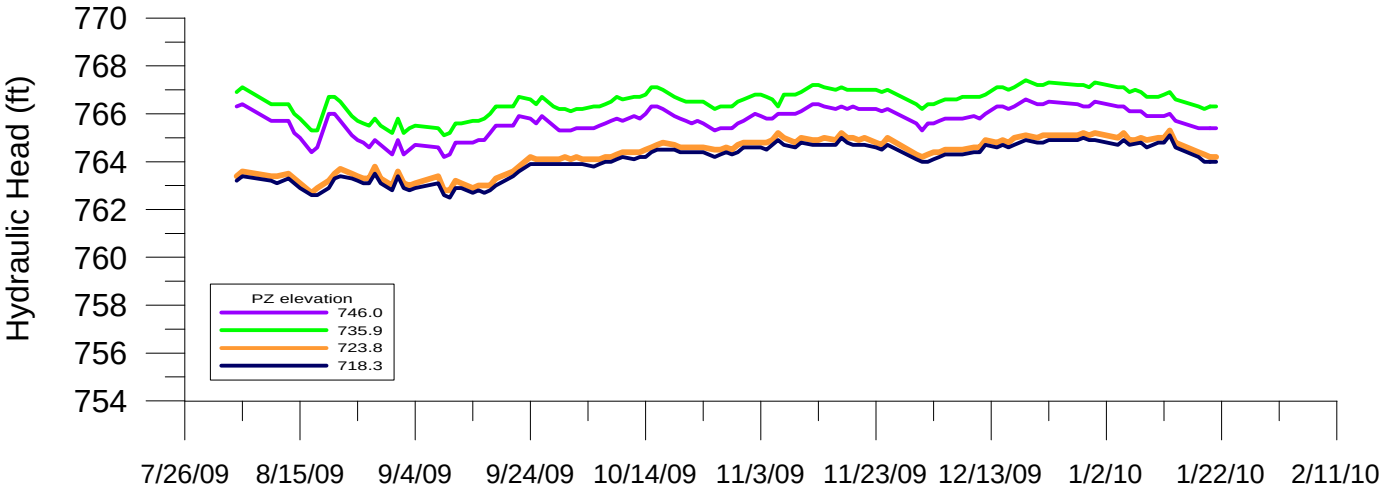
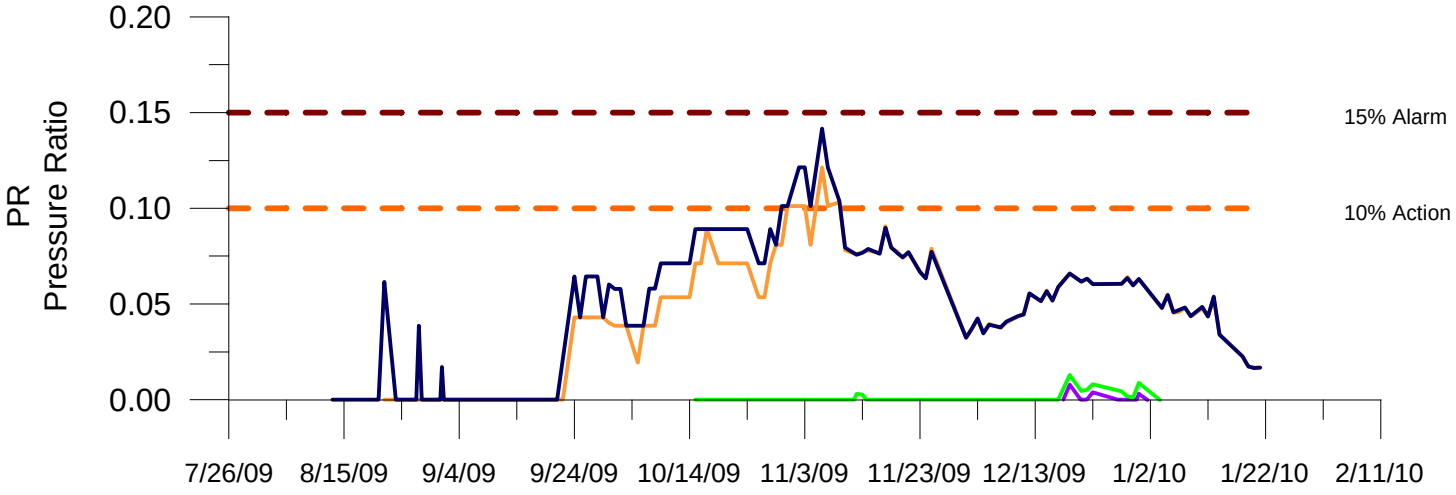
Survey Section 2

PZ-7







* PR(Pressure Ratio) = △U/△Z

* △U= Change in Pore Pressure (psf)

* △Z= Fill Height x 110 (pcf)

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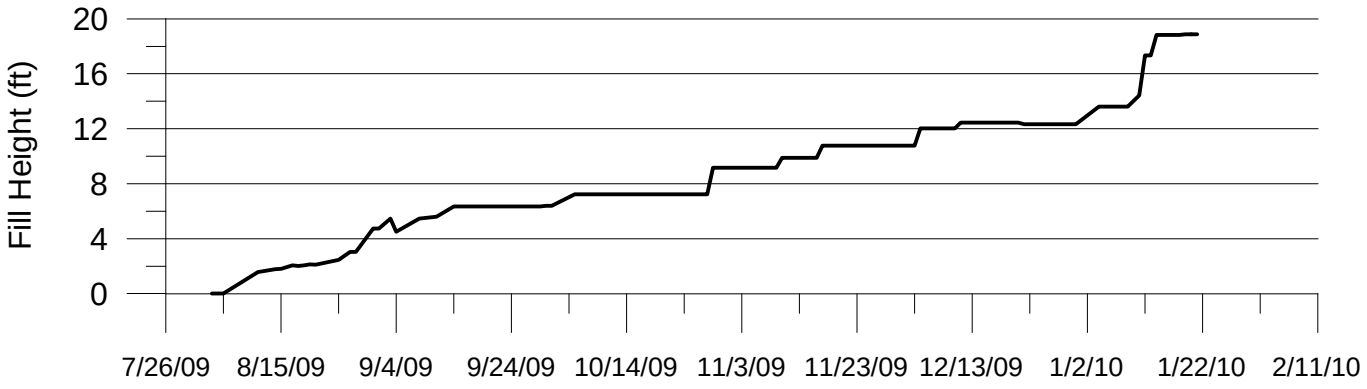
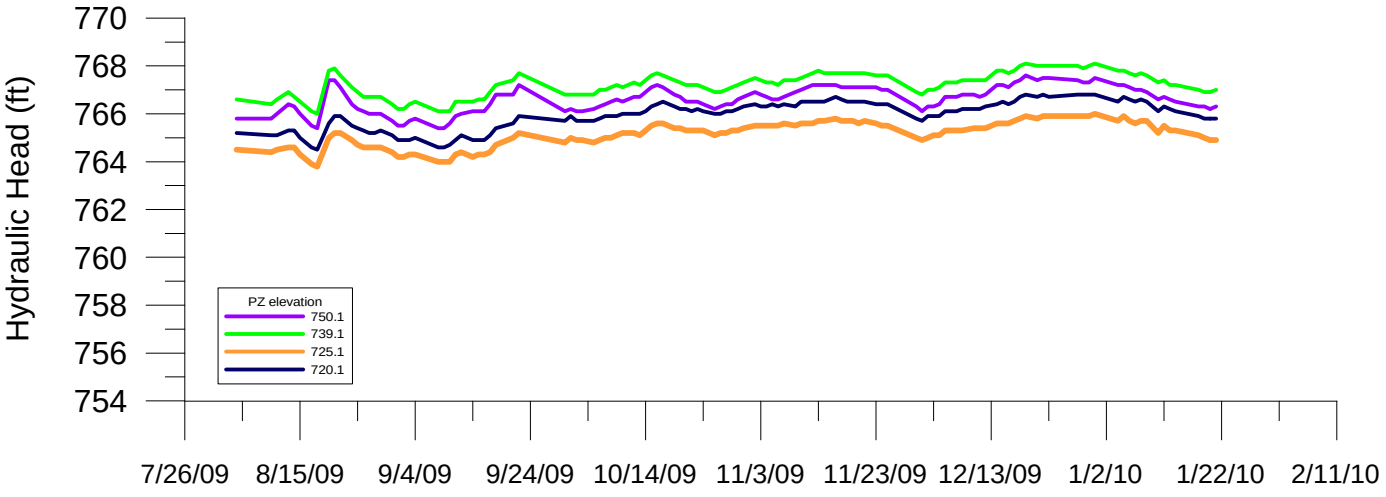
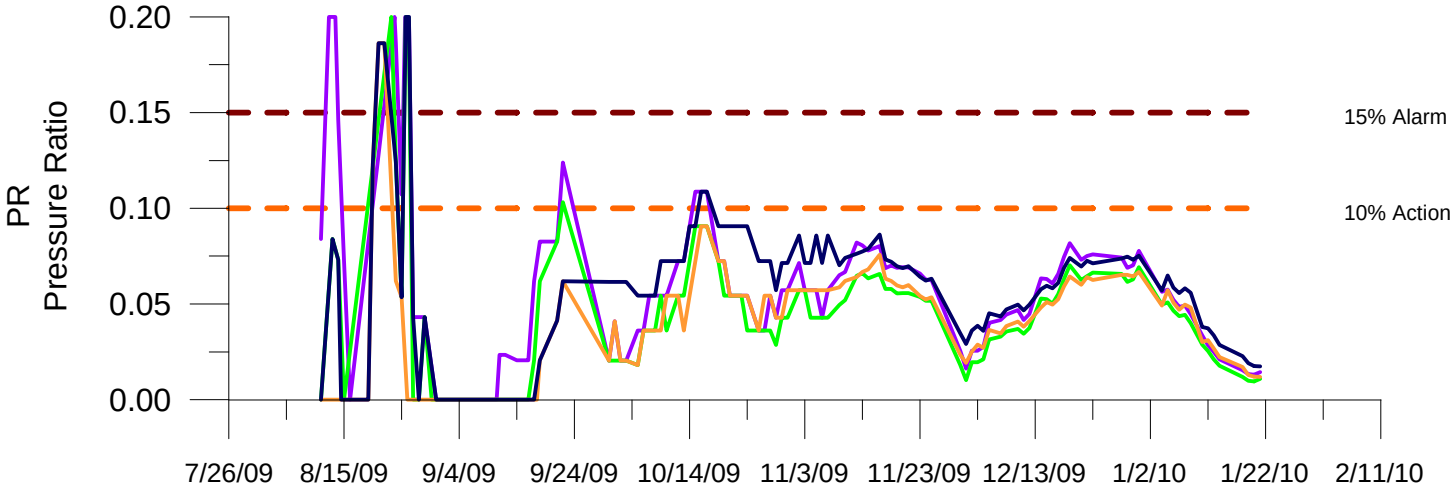
Survey Section 3

PZ-8







* PR(Pressure Ratio) = △U/△Z

* △U= Change in Pore Pressure (psf)

* △Z= Fill Height x 110 (pcf)

Kingston Test Embankment
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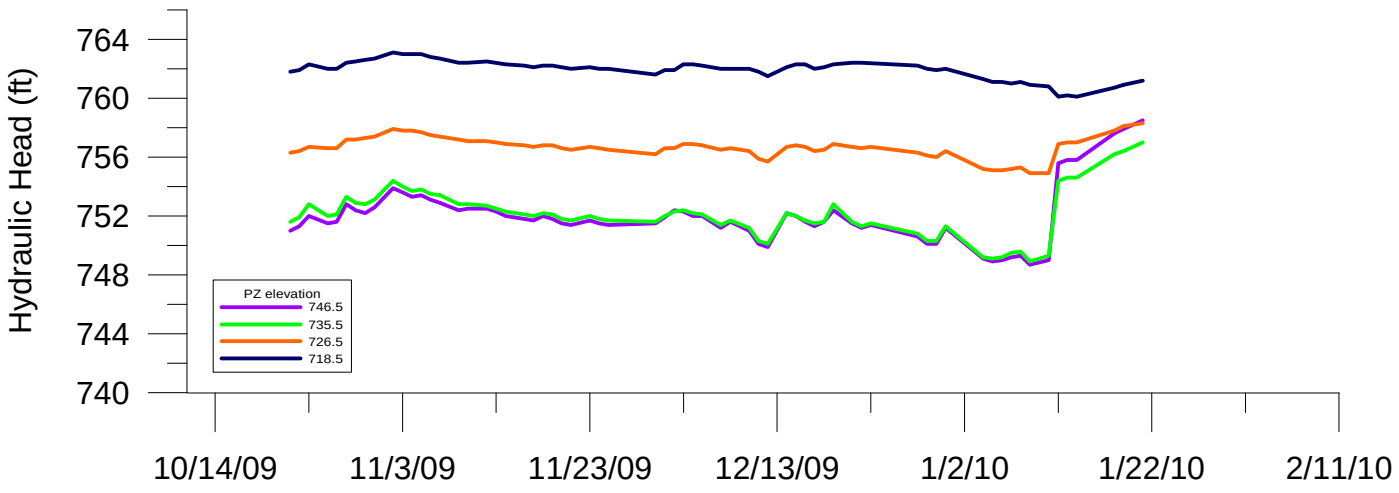
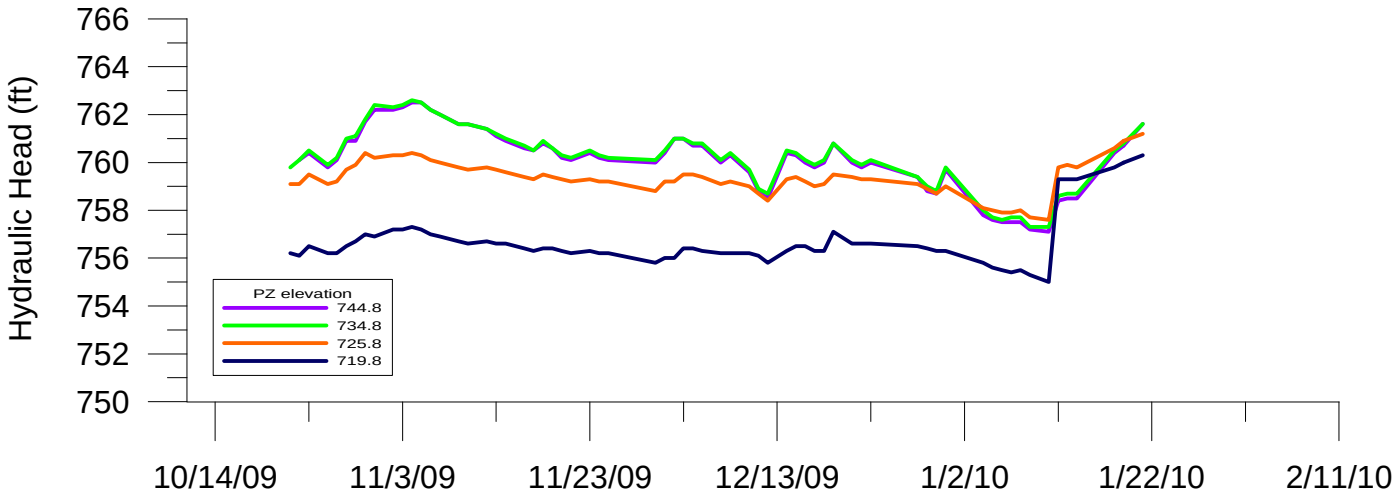
PZ-9



PZ-11



Kingston Test Embankment
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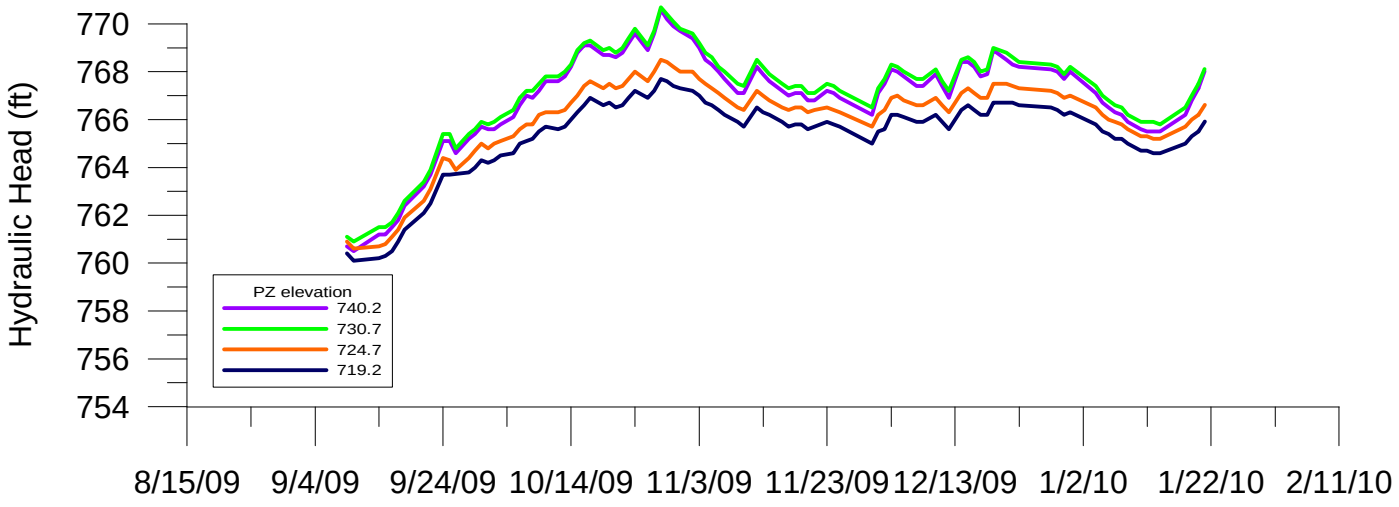
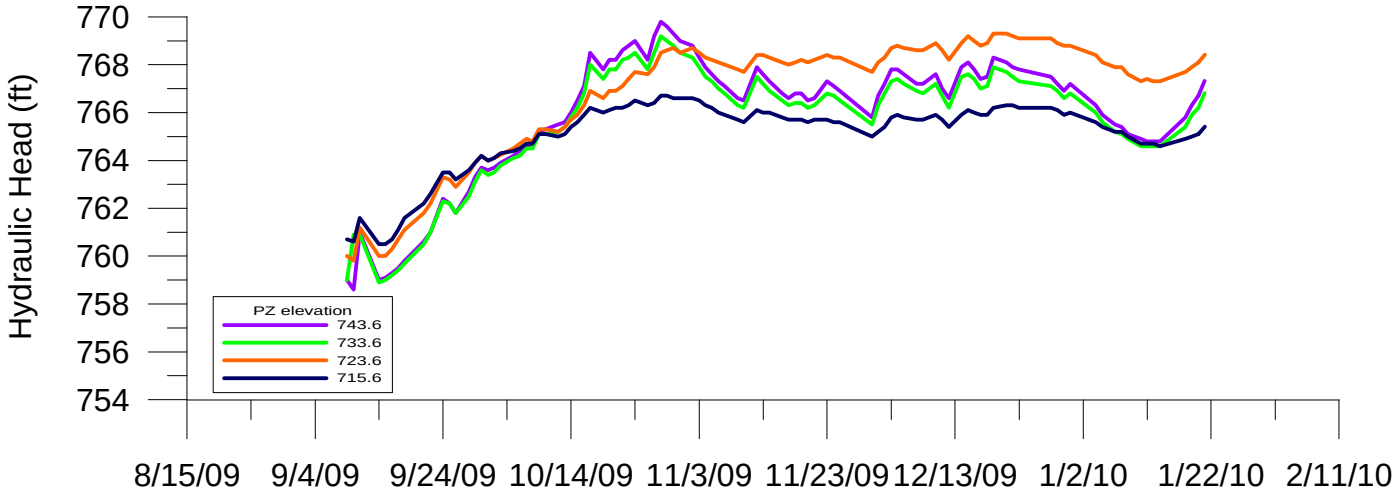
PZ-12



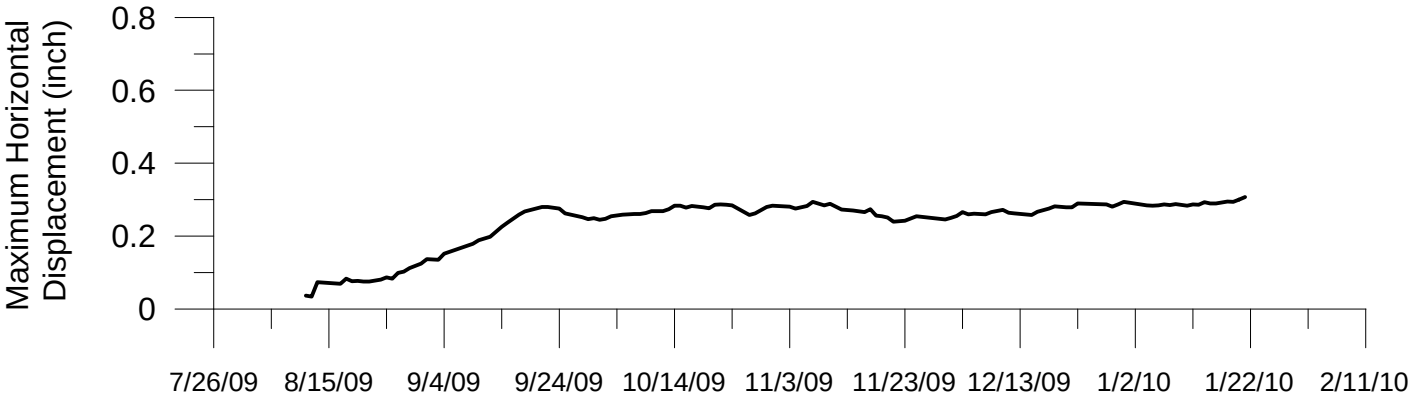
PZ-13

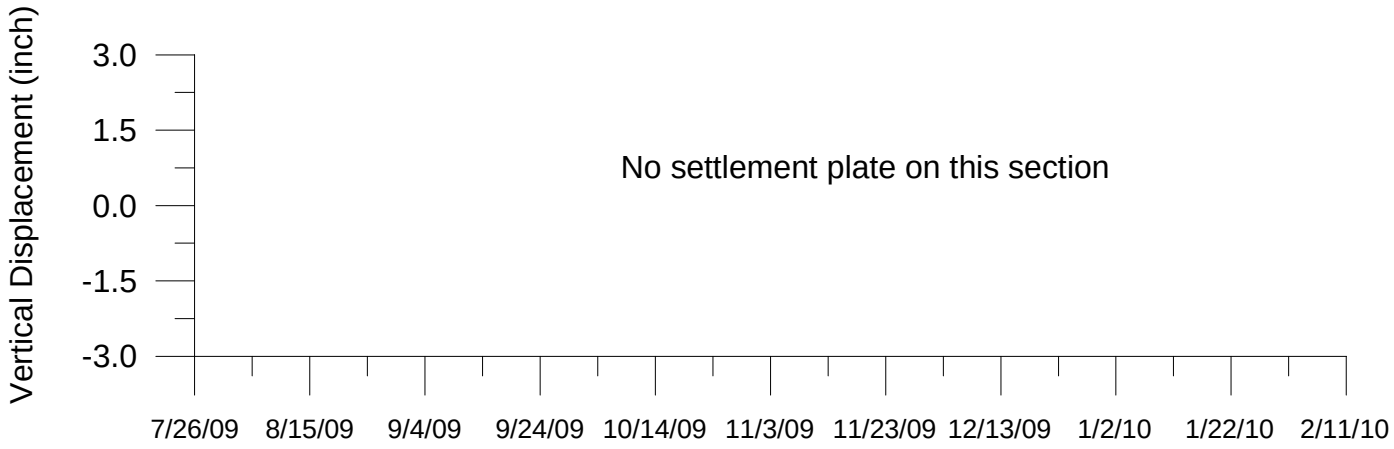


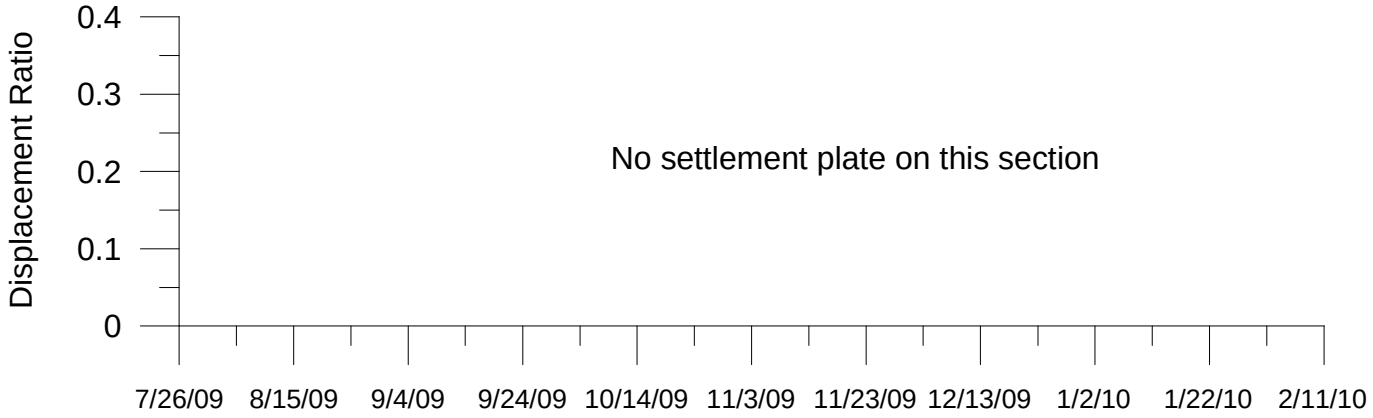
Kingston Test Embankment
175669015

Survey Section 1

SI-2







* DR (Displacement Ratio) = Max Horizontal Displacement / Vertical Displacement

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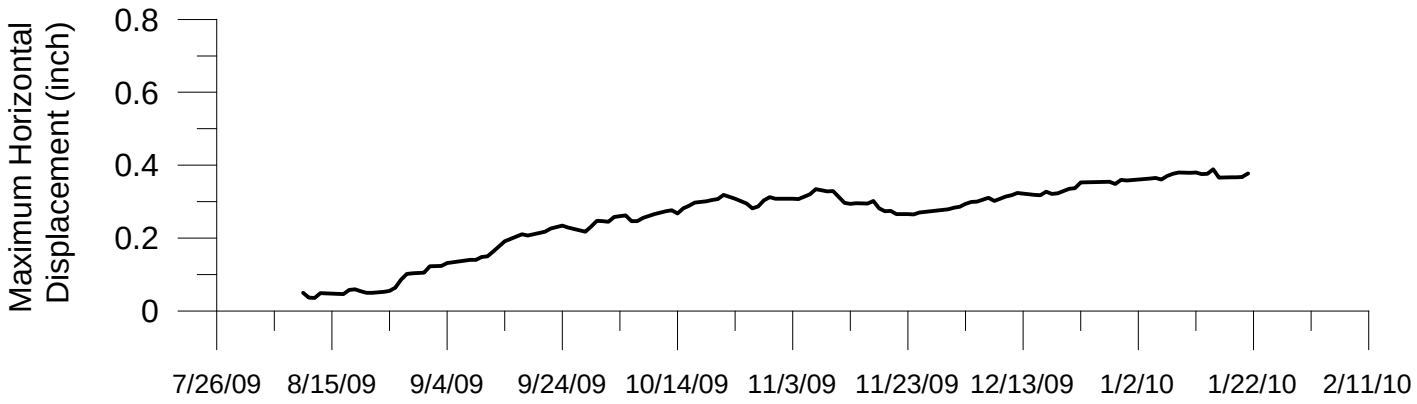
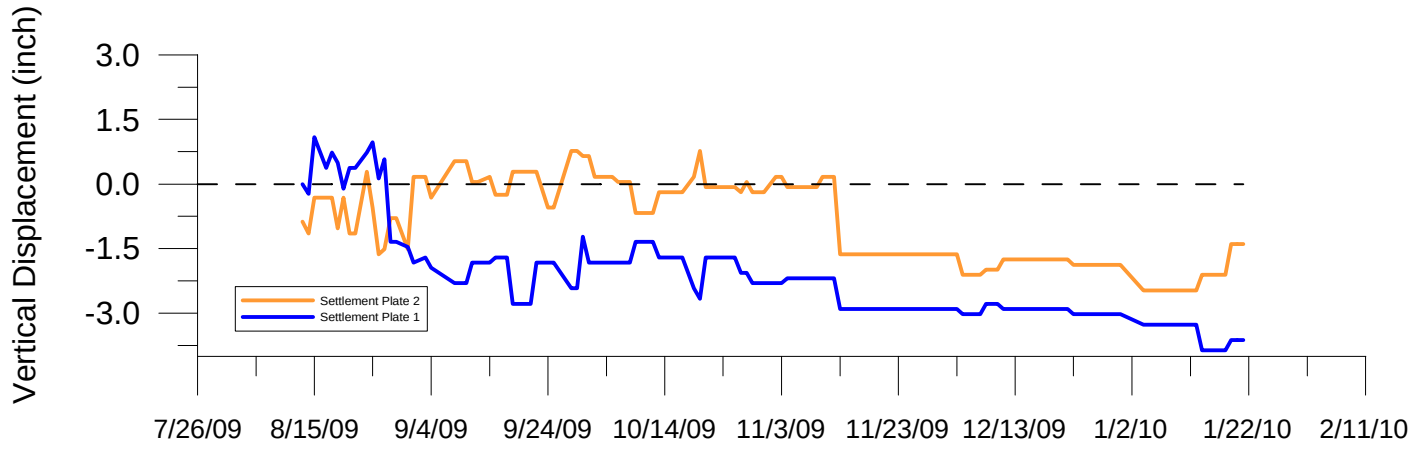
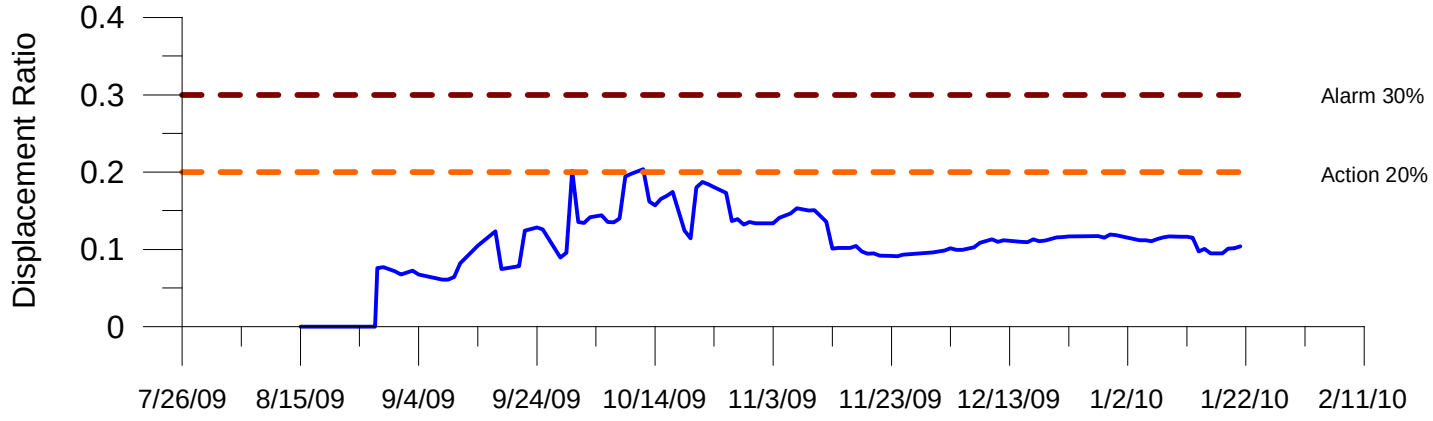
Kingston Test Embankment
175669015

Survey Section 2

SI-3







* DR (Displacement Ratio) = Max Horizontal Displacement / Vertical Displacement

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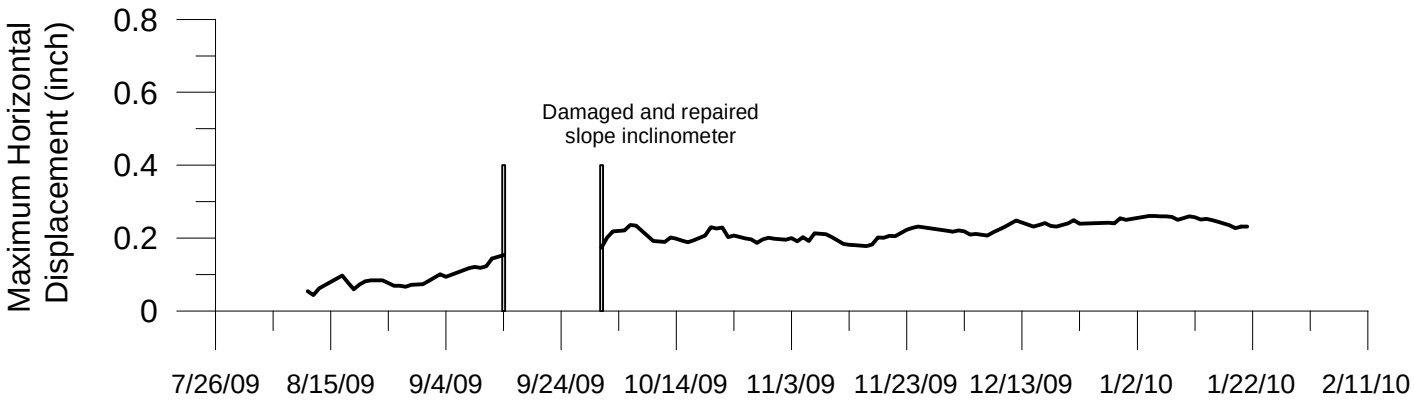
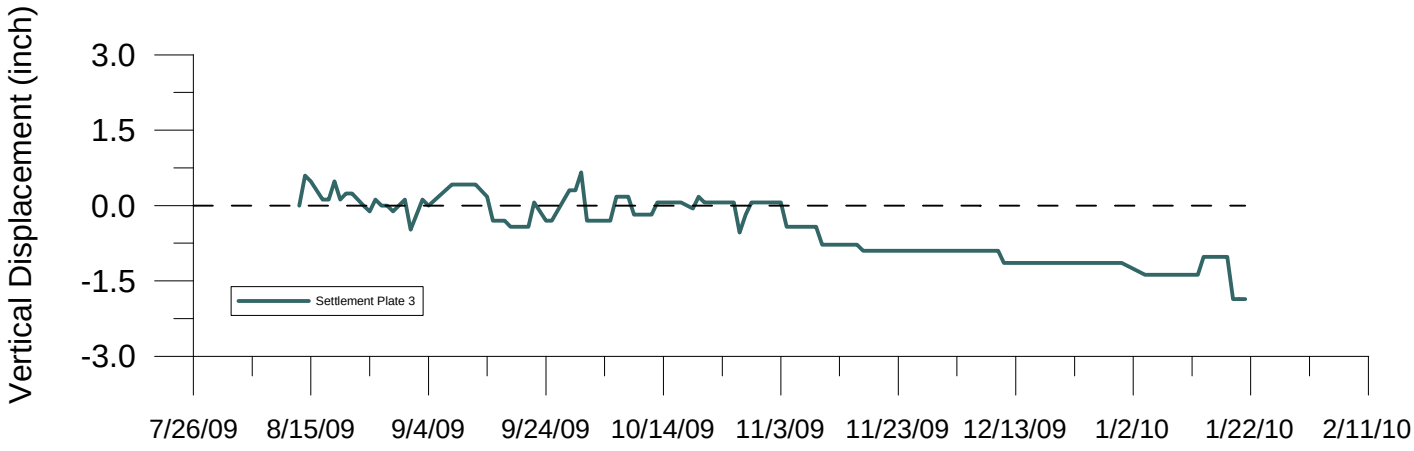
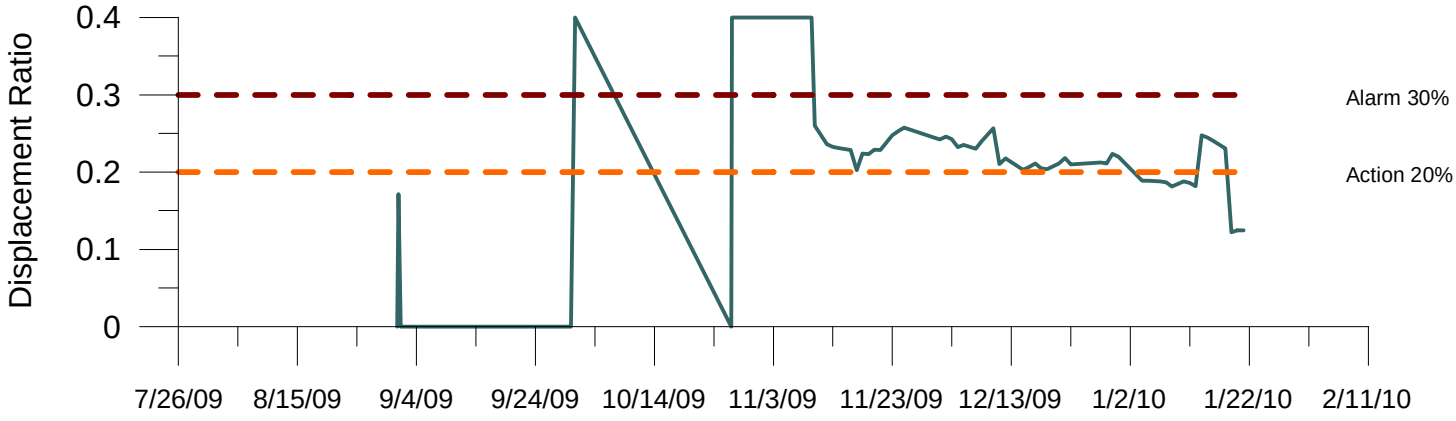
Kingston Test Embankment
175669015

Survey Section 3

SI-4







* DR (Displacement Ratio) = Max Horizontal Displacement / Vertical Displacement

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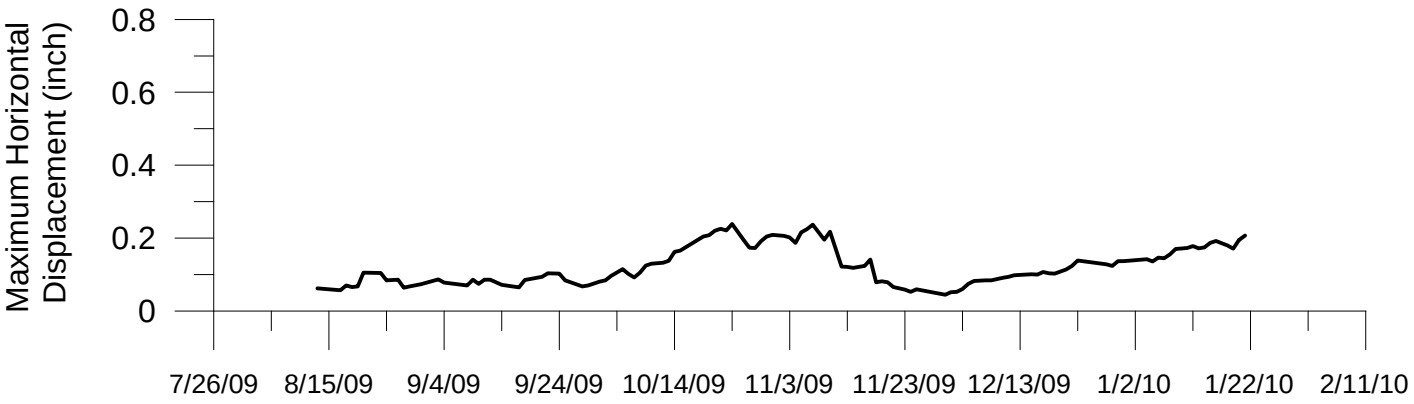
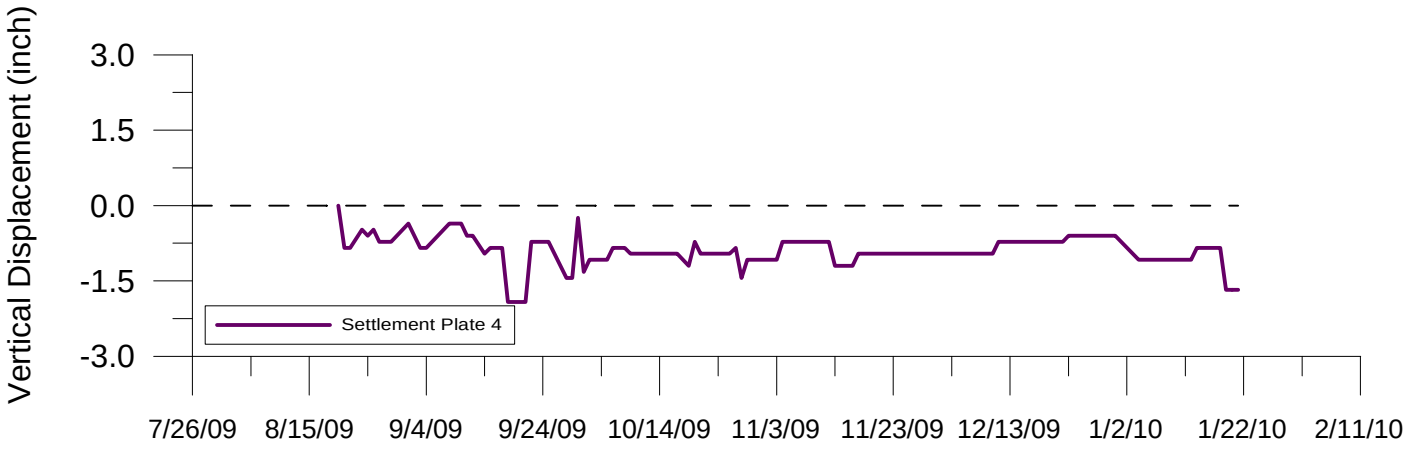
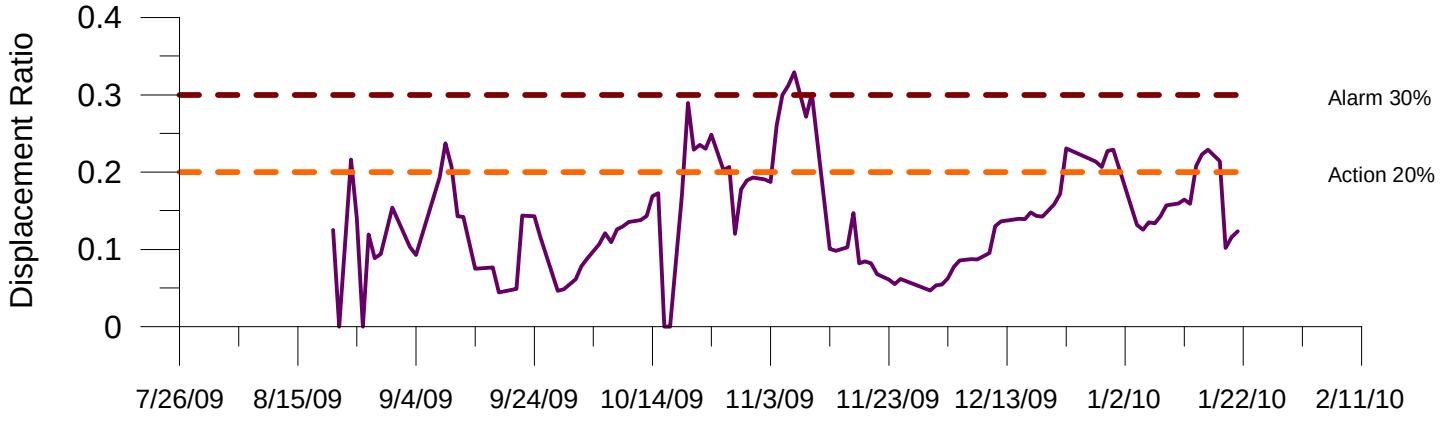
Kingston Test Embankment
175669015

Survey Section 4

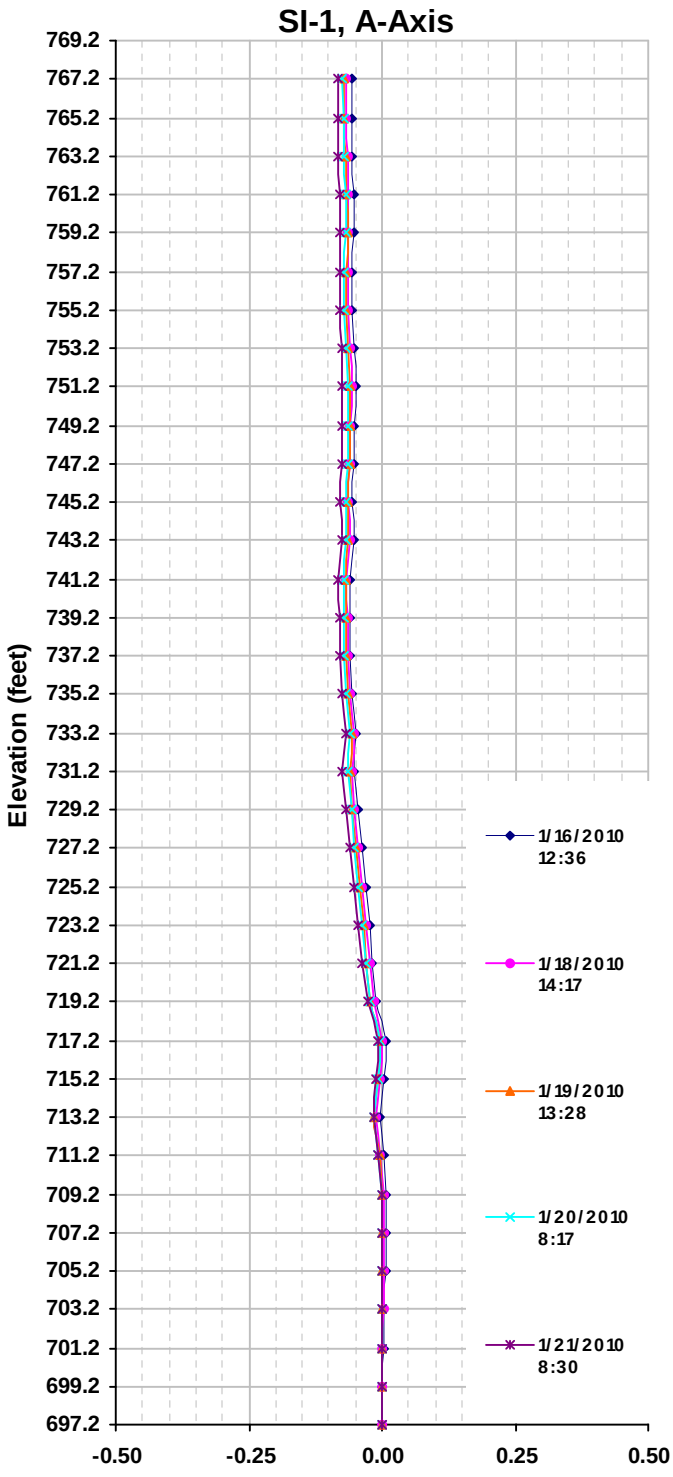
SI-5



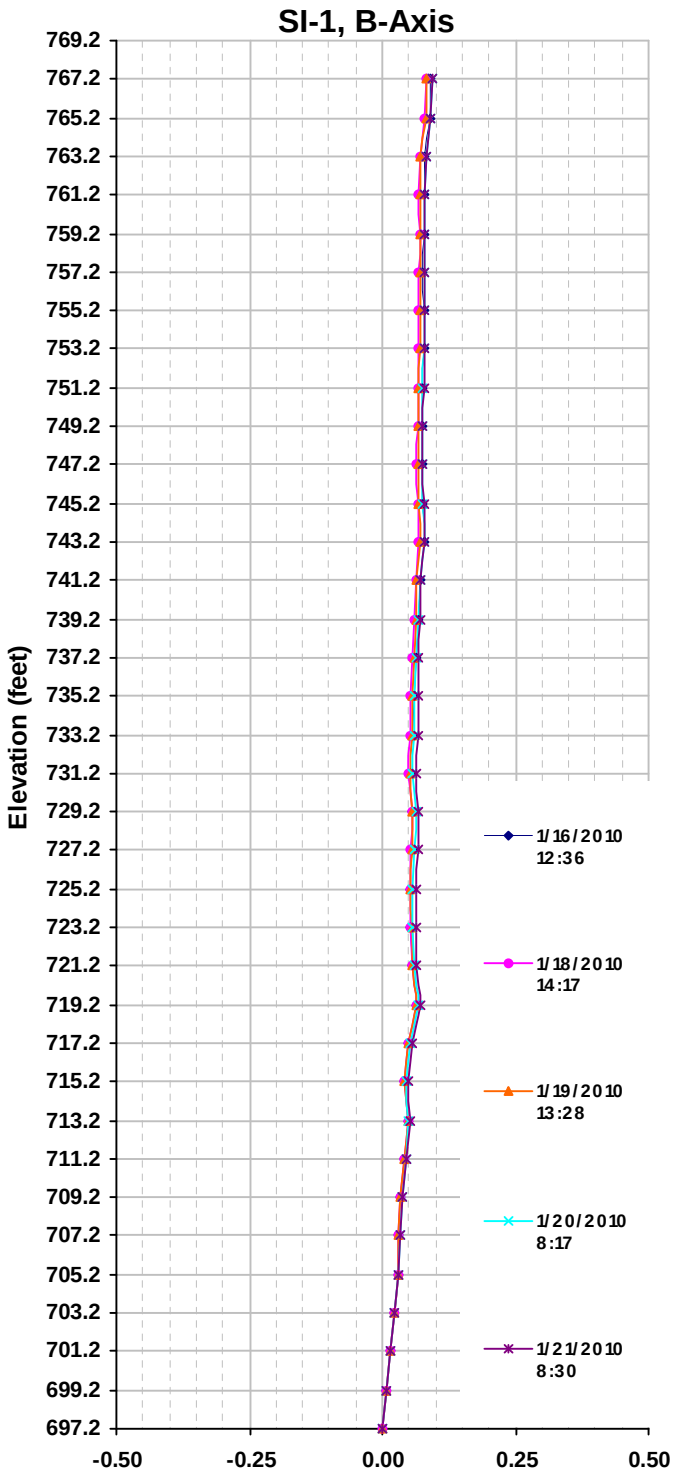




* DR (Displacement Ratio) = Max Horizontal Displacement / Vertical Displacement

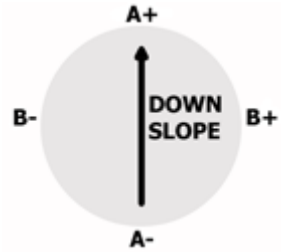


Maximum Displacement: .0840 (in)
Initial date: 8/8/2009

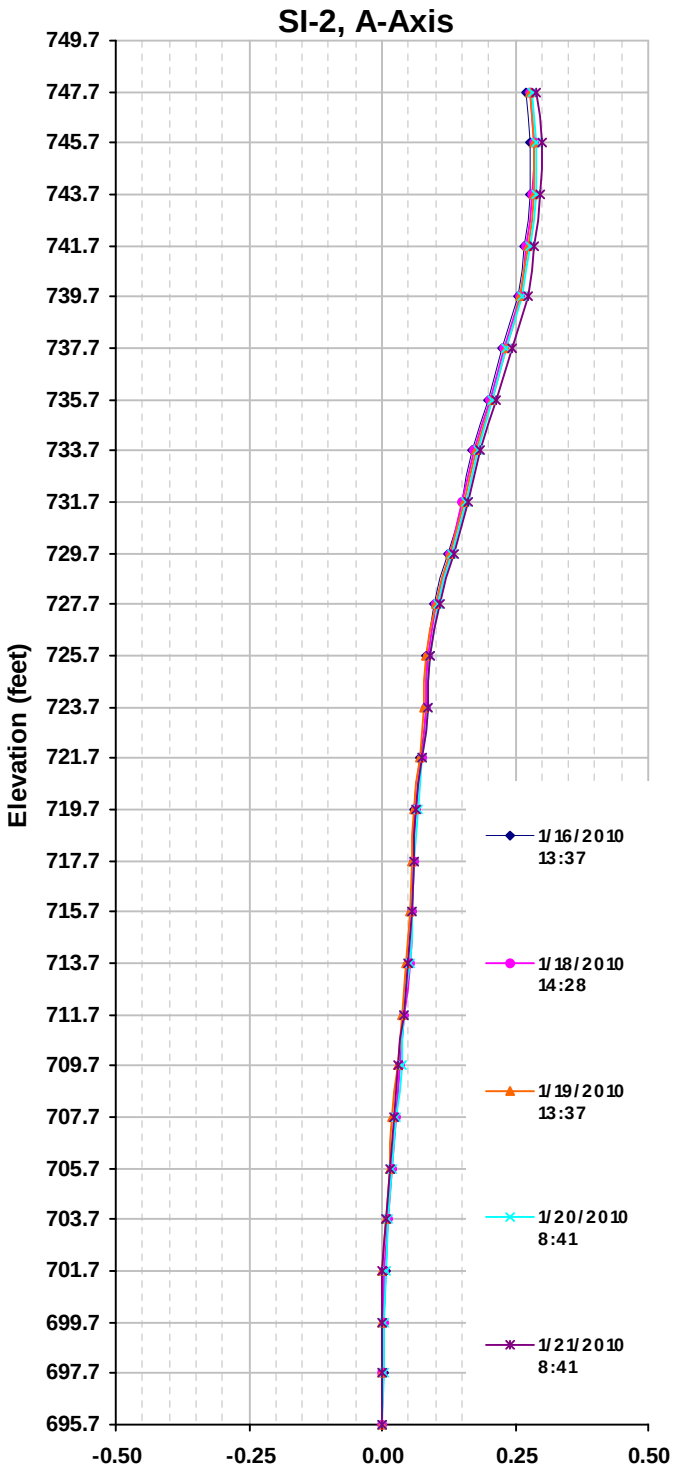


Maximum Displacement: .0936 (in)
Initial date: 8/8/2009

Stantec



Kingston Fossil Plant
Test Embankment
Kingston, TN
175669015
1/22/2010



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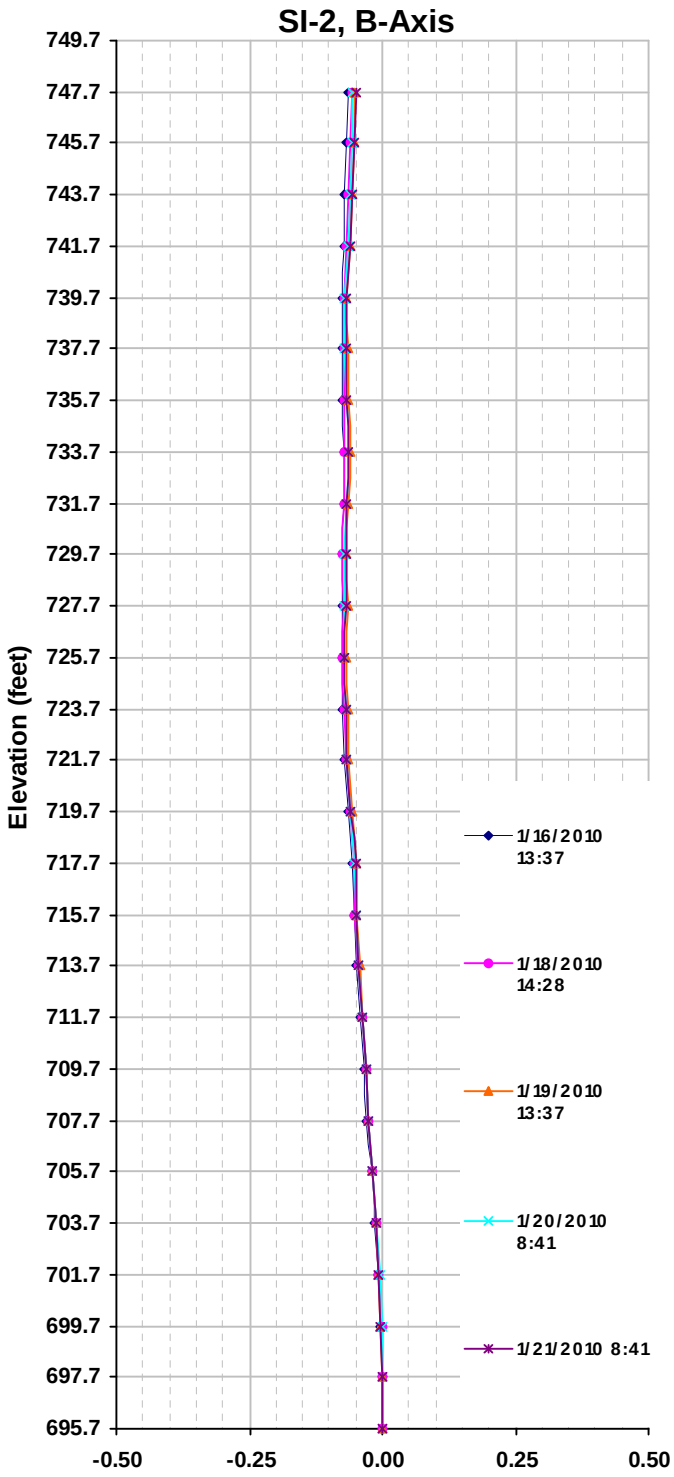
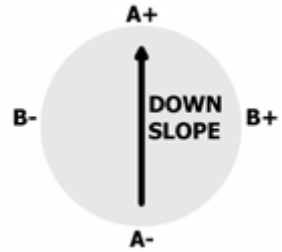
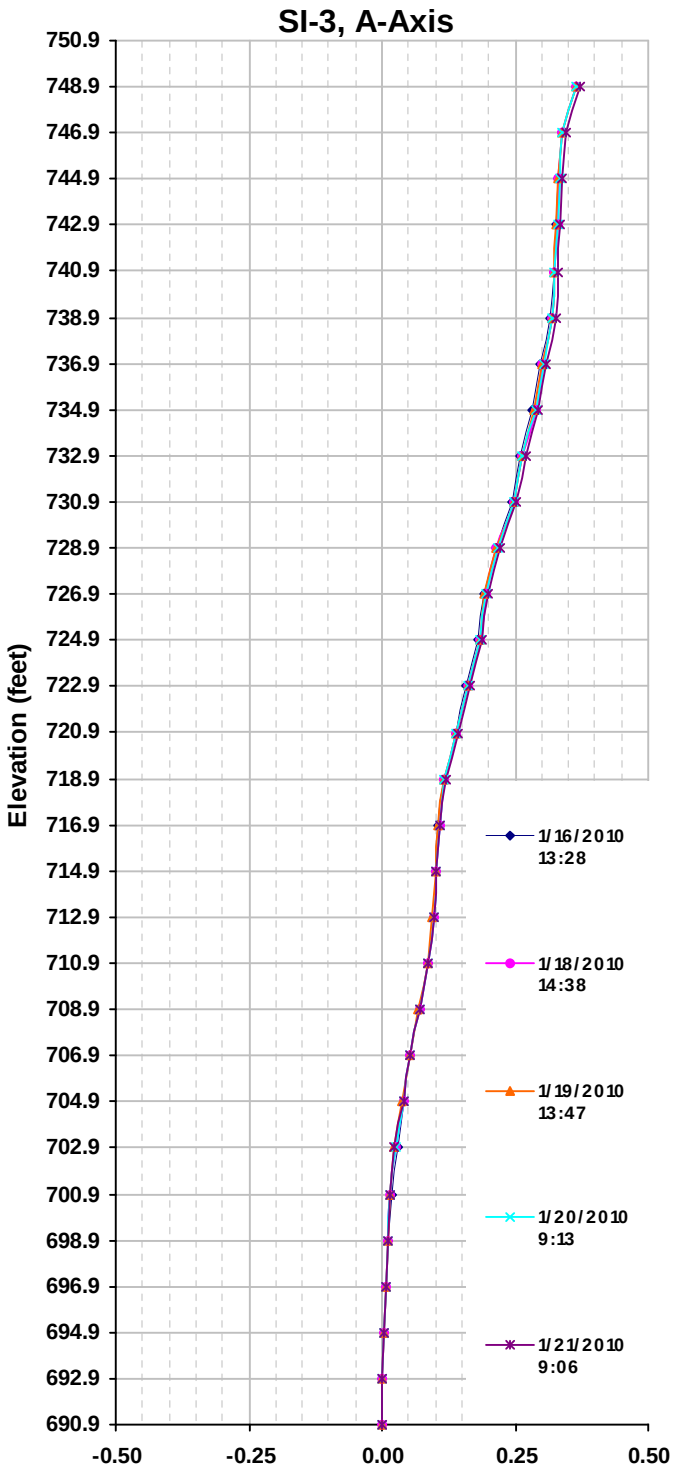


Maximum Displacement: .0768 (in)
Initial date: 8/8/2009

Stantec



Kingston Fossil Plant
Test Embankment
Kingston, TN
175669015
1/22/2010



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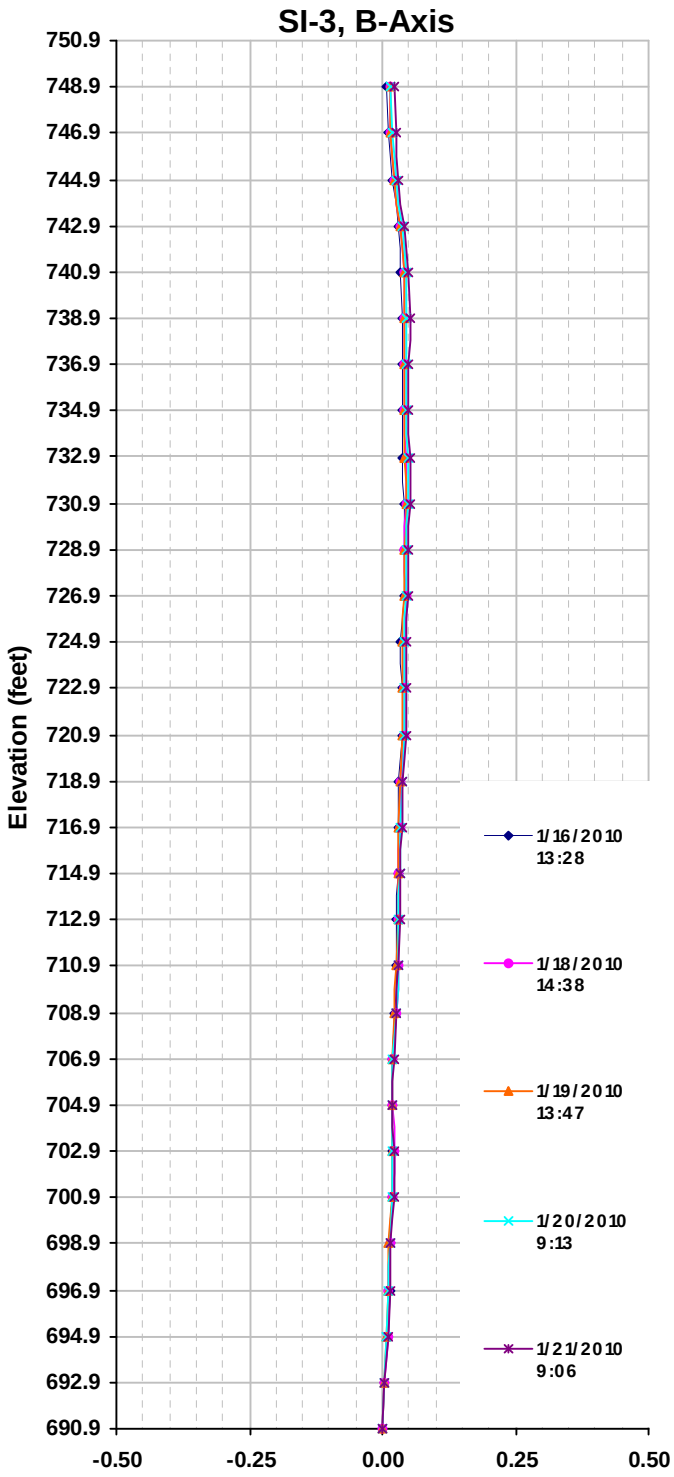
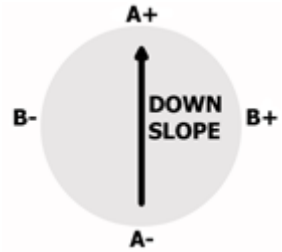
Initial date: 8/8/2009

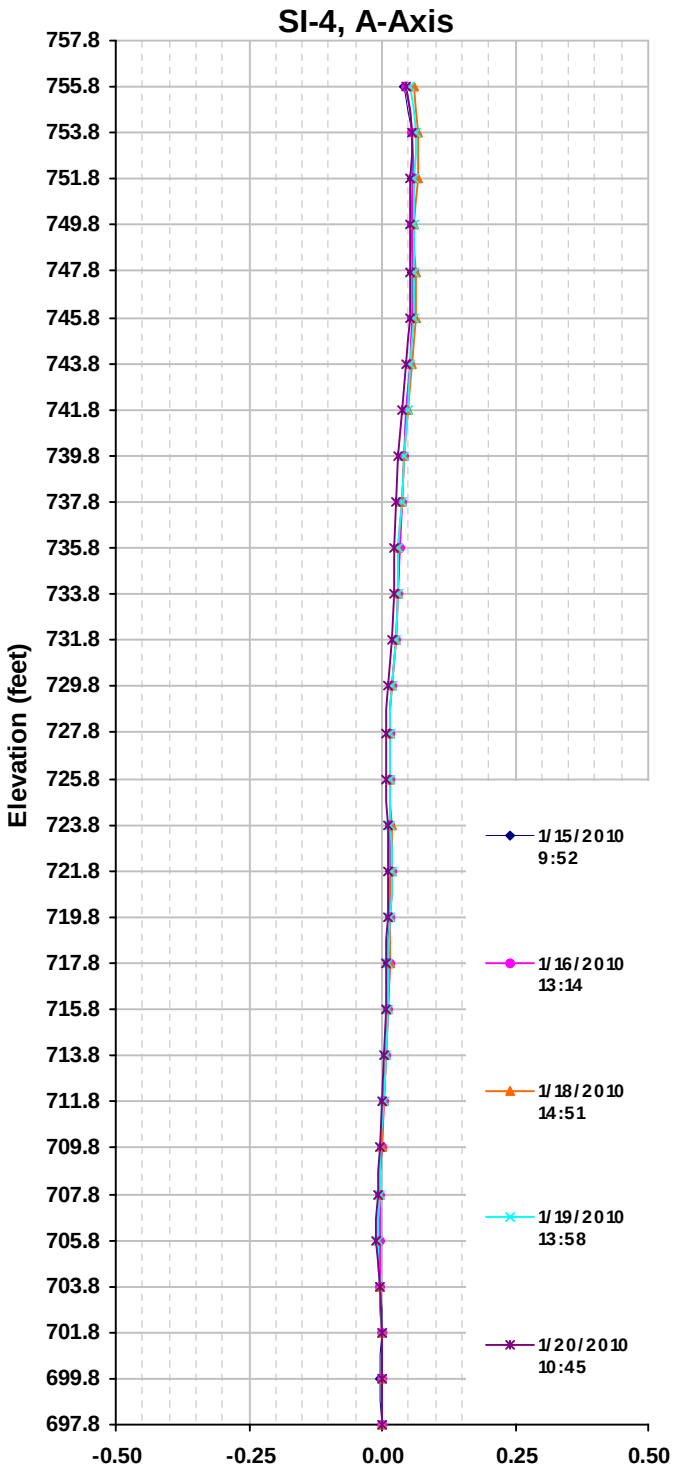


Maximum Displacement: .0522 (in)
Initial date: 8/8/2009

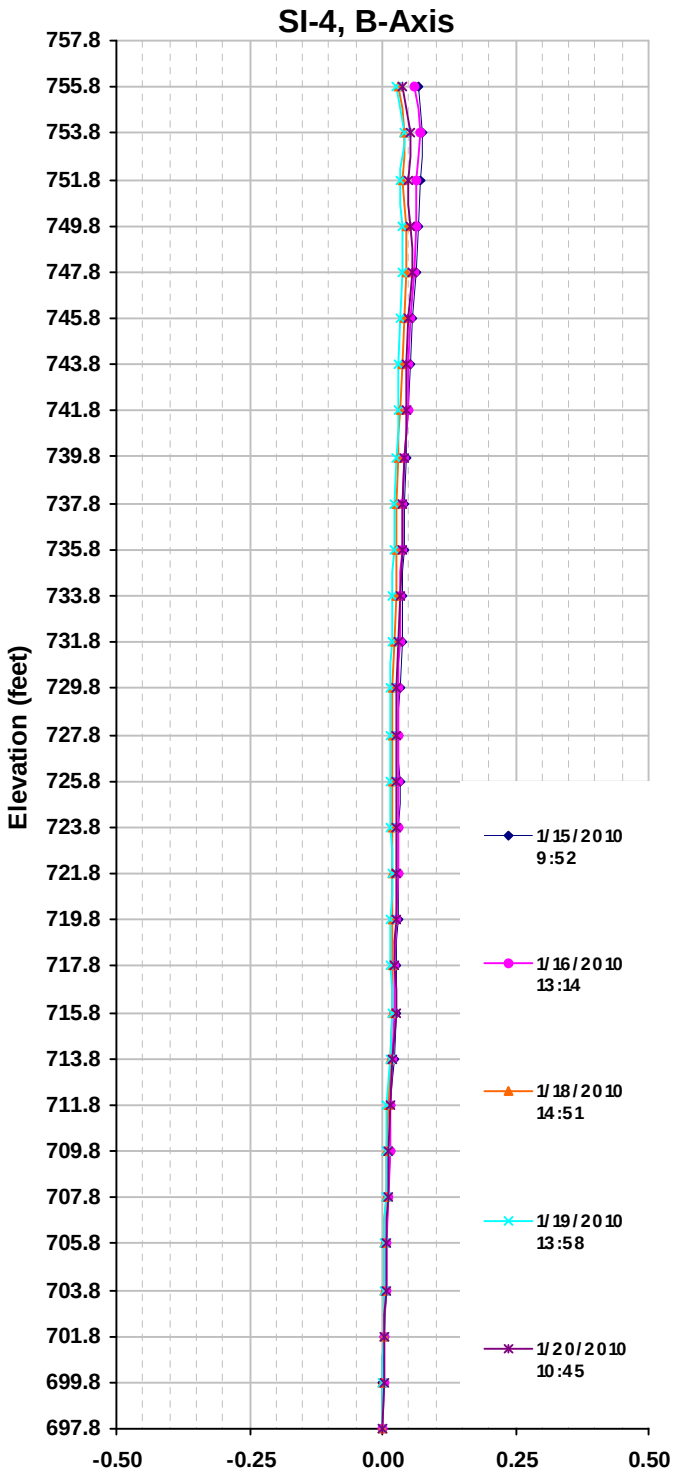
Stantec



Kingston Fossil Plant
Test Embankment
Kingston, TN
175669015
1/22/2010

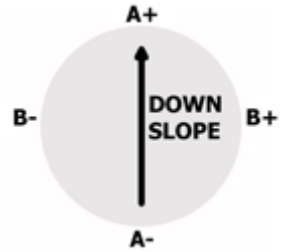


Maximum Displacement: .0684 (in)
Initial date: 10/9/2009

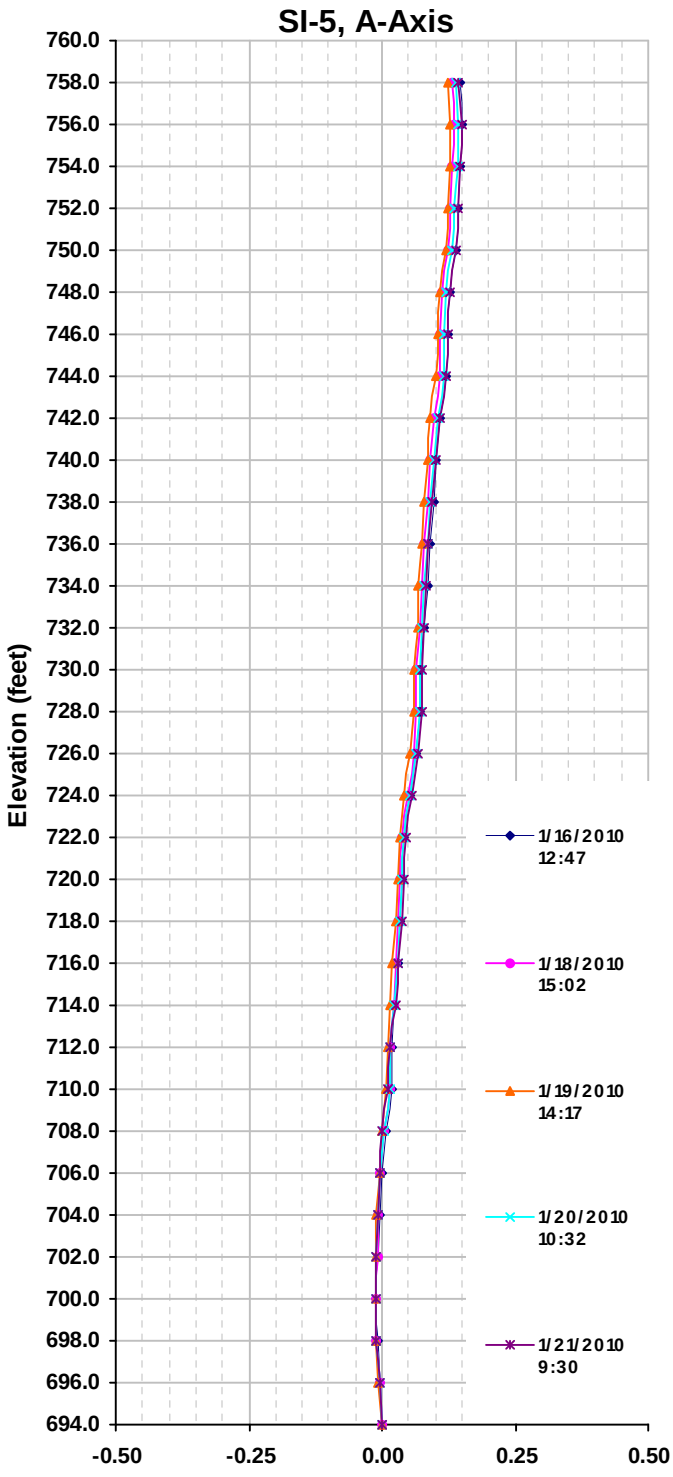


Maximum Displacement: .0768 (in)
Initial date: 10/9/2009

Stantec



Kingston Fossil Plant
Test Embankment
Kingston, TN
175669015
1/22/2010



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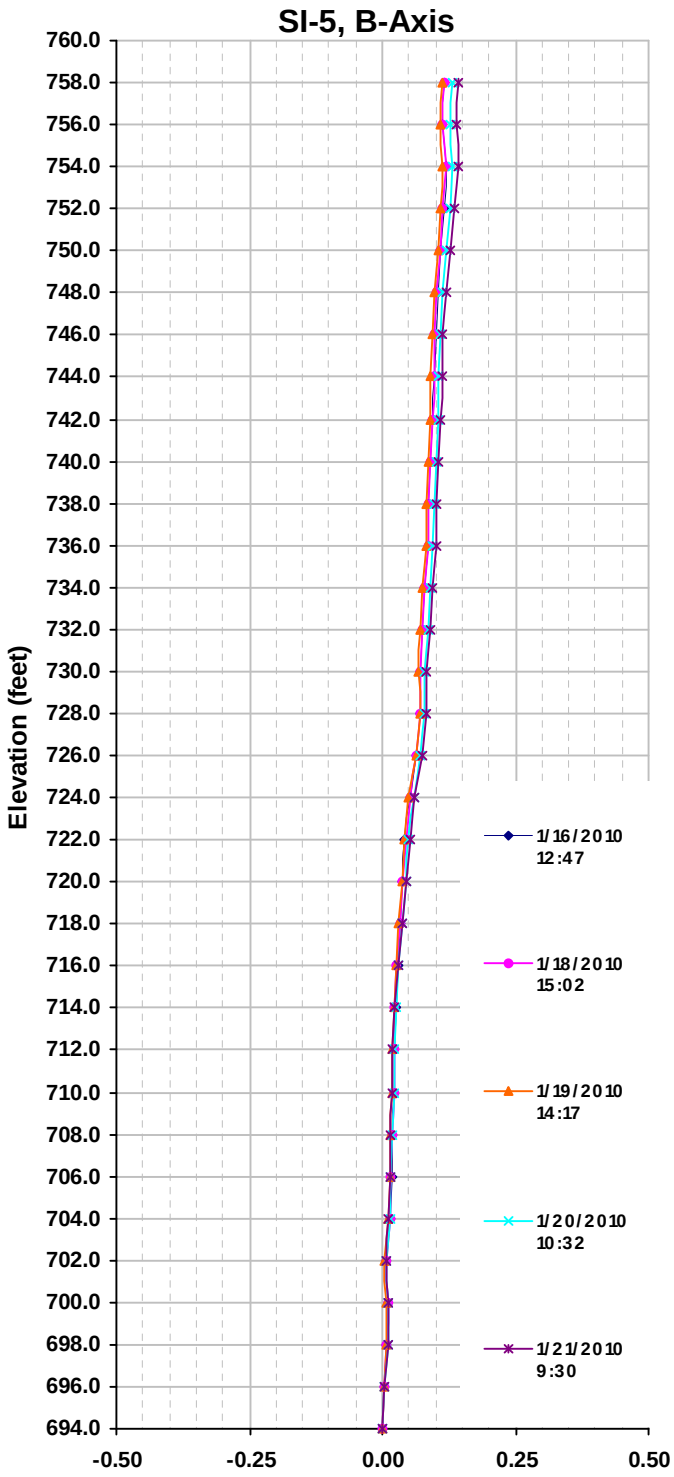
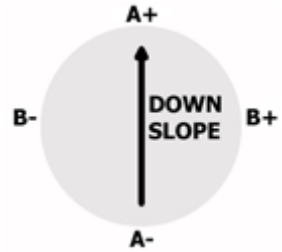
Initial date: 8/8/2009

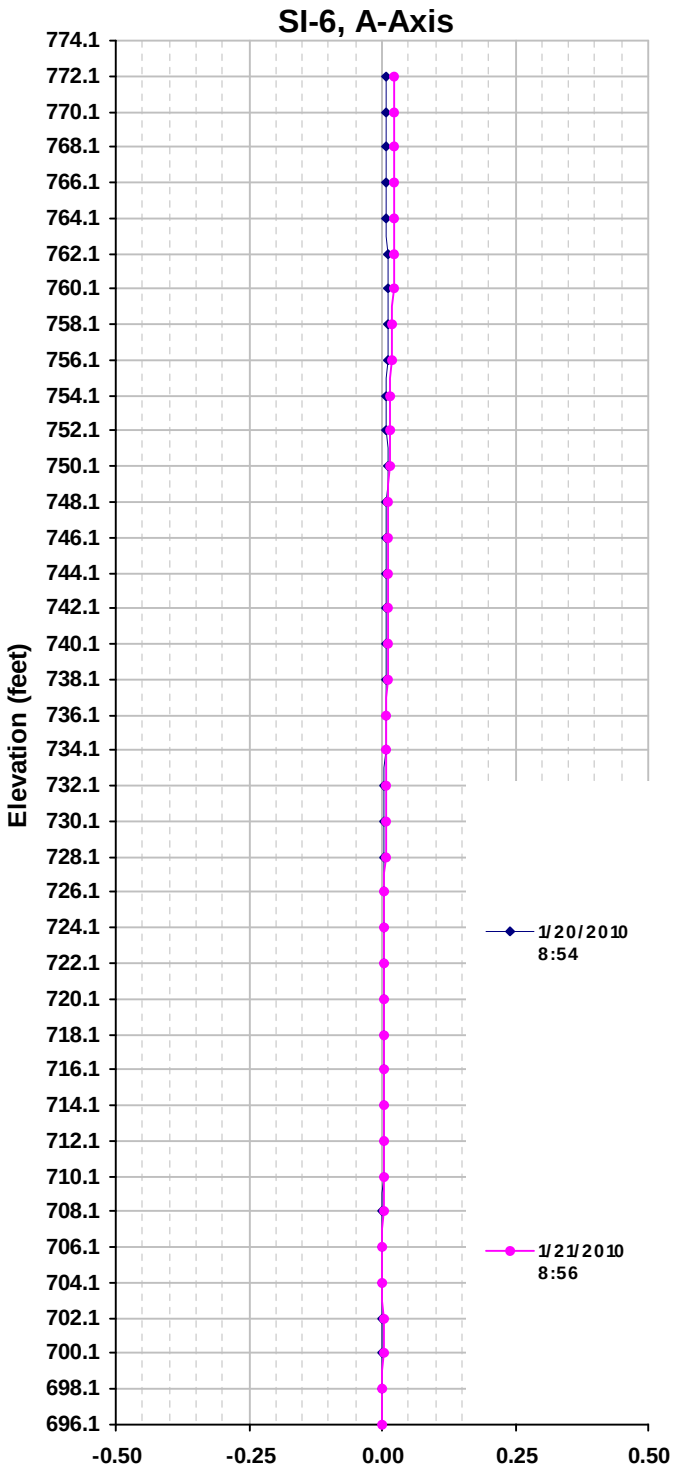


Maximum Displacement: .1428 (in)
Initial date: 8/8/2009

Stantec



Kingston Fossil Plant
Test Embankment
Kingston, TN
175669015
1/22/2010



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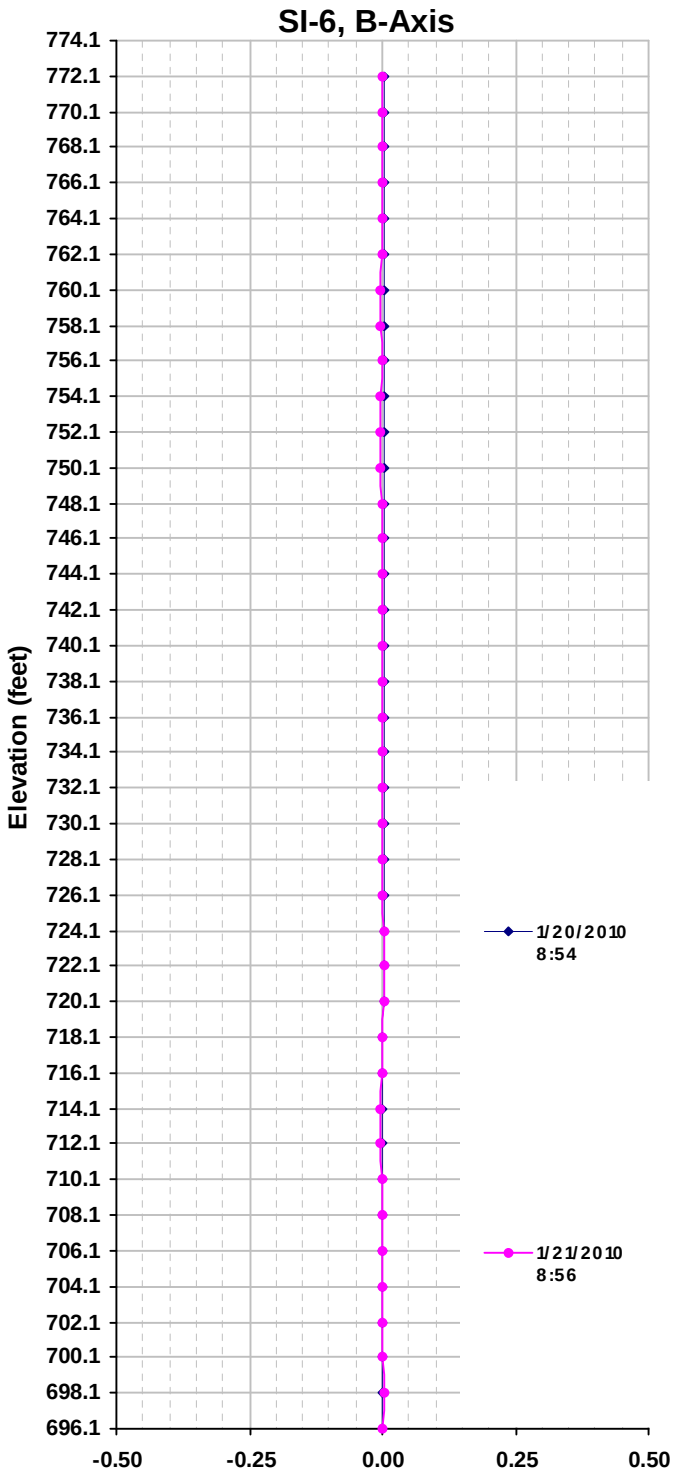
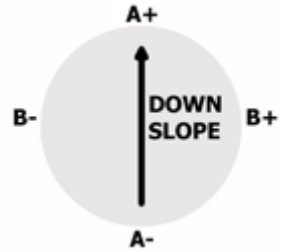
Initial date: 1/19/2010



Maximum Displacement: .0054 (in)
Initial date: 1/19/2010

Stantec



Kingston Fossil Plant
Test Embankment
Kingston, TN
175669015
1/22/2010

Exhibit C

Stability of Section 2 at Completion of Tier 2

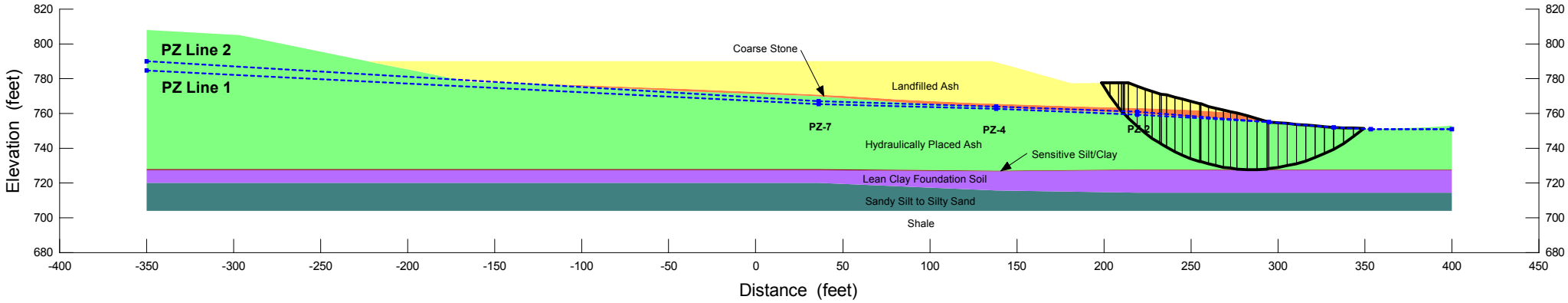
Slope Stability - Deep Failure Analysis
Test Embankment - Section 2
Trigger Event - Nov 1

Kingston Fossil Plant
Tennessee Valley Authority

Date Saved: 1/23/2010
Last Solved on 1/23/2010 at 4:14:04 PM
Method: Modified Spencer
File Name: Test Emb_Sec2_Slope_Stability (Existing).gsz

Material Type	Unit Weight		Cohesion	Friction Angle	PZ Line
	Saturated	Moist			
Landfilled Ash	111	109	0	30	2
Coarse Stone	128	115	0	38	2
Hydraulically Placed Ash	107	100	0	25	2
Sensitive Silt/Clay	107	100	0	28	2
Lean Clay Foundation Soil	130		0	32	1
Sandy Silt to Silty Sand	128		0	30	1
Shale	Impenetrable				1

Factor of Safety: 1.65
Center: (285, 827.5) ft
Radius: 99.8 ft
Minimum Slip Surface Depth: 1 ft

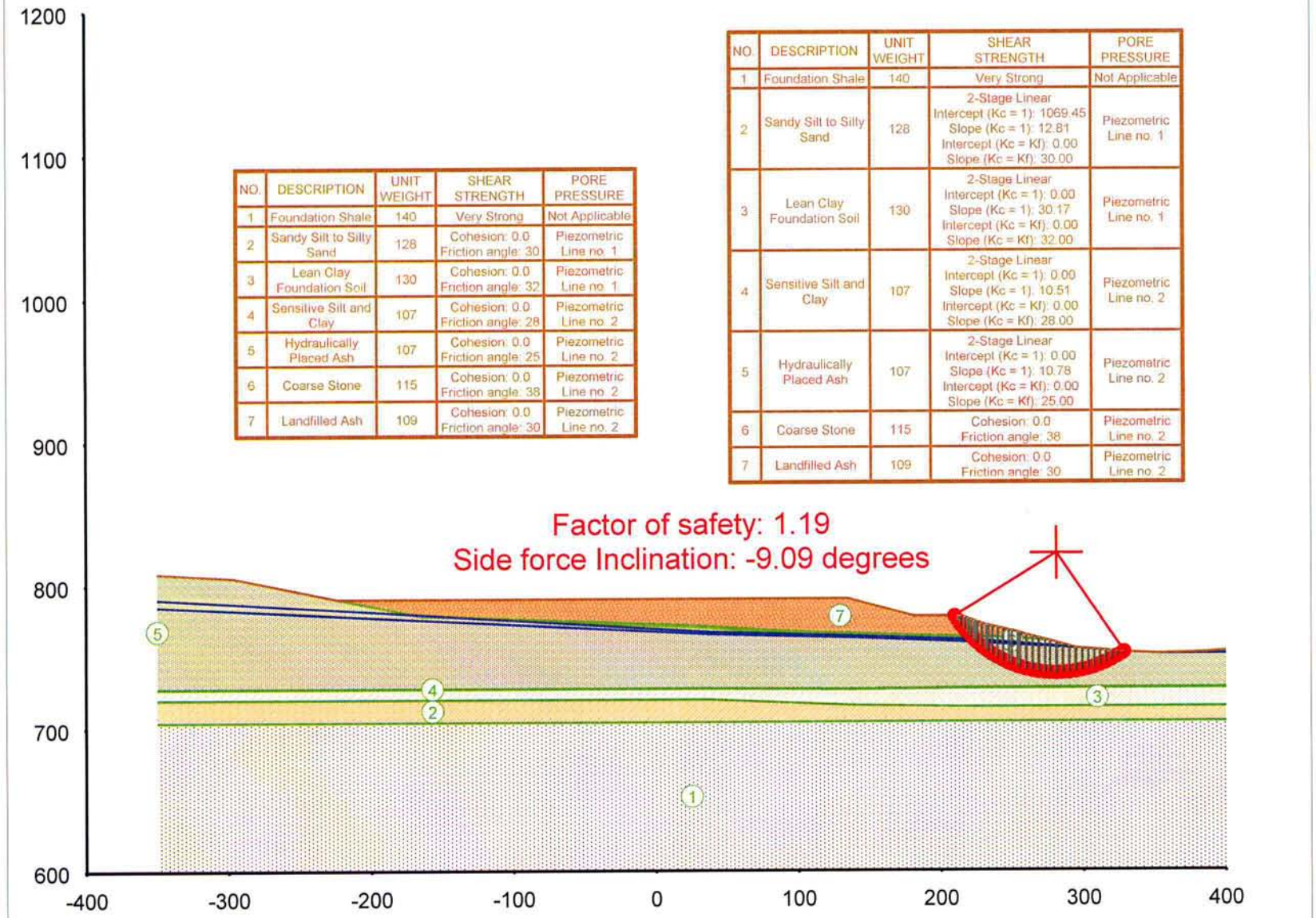


Directory: \\Us1243-f01\workgroup\1756\active\175669015\environmental\analysis\Trigger Event Nov 1\Test Embankment Stability Runs\2010-01-11\Test Emb_Sec2_Slope_Stability (Existing).gsz

TVA Kingston Test Embankment - Section 2

NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Foundation Shale	140	Very Strong	Not Applicable
2	Sandy Silt to Silty Sand	128	Cohesion: 0.0 Friction angle: 30	Piezometric Line no. 1
3	Lean Clay Foundation Soil	130	Cohesion: 0.0 Friction angle: 32	Piezometric Line no. 1
4	Sensitive Silt and Clay	107	Cohesion: 0.0 Friction angle: 28	Piezometric Line no. 2
5	Hydraulically Placed Ash	107	Cohesion: 0.0 Friction angle: 25	Piezometric Line no. 2
6	Coarse Stone	115	Cohesion: 0.0 Friction angle: 38	Piezometric Line no. 2
7	Landfilled Ash	109	Cohesion: 0.0 Friction angle: 30	Piezometric Line no. 2

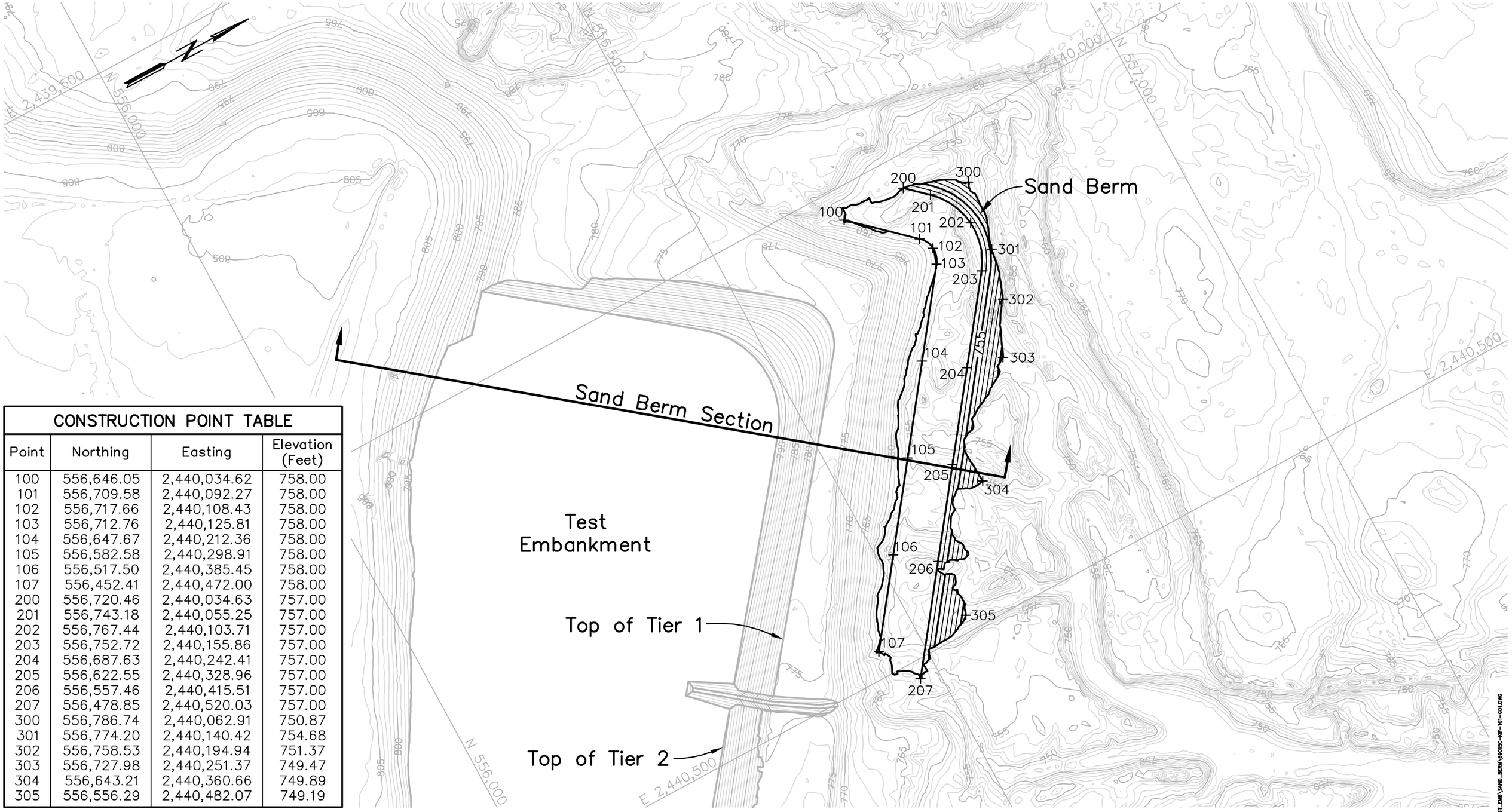
NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Foundation Shale	140	Very Strong	Not Applicable
2	Sandy Silt to Silty Sand	128	2-Stage Linear Intercept (Kc = 1): 1069.45 Slope (Kc = 1): 12.81 Intercept (Kc = Kf): 0.00 Slope (Kc = Kf): 30.00	Piezometric Line no. 1
3	Lean Clay Foundation Soil	130	2-Stage Linear Intercept (Kc = 1): 0.00 Slope (Kc = 1): 30.17 Intercept (Kc = Kf): 0.00 Slope (Kc = Kf): 32.00	Piezometric Line no. 1
4	Sensitive Silt and Clay	107	2-Stage Linear Intercept (Kc = 1): 0.00 Slope (Kc = 1): 10.51 Intercept (Kc = Kf): 0.00 Slope (Kc = Kf): 28.00	Piezometric Line no. 2
5	Hydraulically Placed Ash	107	2-Stage Linear Intercept (Kc = 1): 0.00 Slope (Kc = 1): 10.78 Intercept (Kc = Kf): 0.00 Slope (Kc = Kf): 25.00	Piezometric Line no. 2
6	Coarse Stone	115	Cohesion: 0.0 Friction angle: 38	Piezometric Line no. 2
7	Landfilled Ash	109	Cohesion: 0.0 Friction angle: 30	Piezometric Line no. 2



Date: Mon Jan 25 2010 Filename: Section 2 - Existing - Type 2 Search.UT4 Time: 15:31:55

Exhibit D

Recommended Sand Berm at the Western End of the Test Embankment



CONSTRUCTION POINT TABLE			
Point	Northing	Easting	Elevation (Feet)
100	556,646.05	2,440,034.62	758.00
101	556,709.58	2,440,092.27	758.00
102	556,717.66	2,440,108.43	758.00
103	556,712.76	2,440,125.81	758.00
104	556,647.67	2,440,212.36	758.00
105	556,582.58	2,440,298.91	758.00
106	556,517.50	2,440,385.45	758.00
107	556,452.41	2,440,472.00	758.00
200	556,720.46	2,440,034.63	757.00
201	556,743.18	2,440,055.25	757.00
202	556,767.44	2,440,103.71	757.00
203	556,752.72	2,440,155.86	757.00
204	556,687.63	2,440,242.41	757.00
205	556,622.55	2,440,328.96	757.00
206	556,557.46	2,440,415.51	757.00
207	556,478.85	2,440,520.03	757.00
300	556,786.74	2,440,062.91	750.87
301	556,774.20	2,440,140.42	754.68
302	556,758.53	2,440,194.94	751.37
303	556,727.98	2,440,251.37	749.47
304	556,643.21	2,440,360.66	749.89
305	556,556.29	2,440,482.07	749.19

NOTES:

1. Topographic mapping was generated from a LIDAR survey performed on Nov., 25, 2009 by Tuck Mapping Solutions, Inc.
2. Topographic information on the Test Embankment between Tier 1 and Tier 2 depicts the approximate surface configuration in mid–January, 2010.

PLAN – SAND BERM

SCALE: 1"=100'

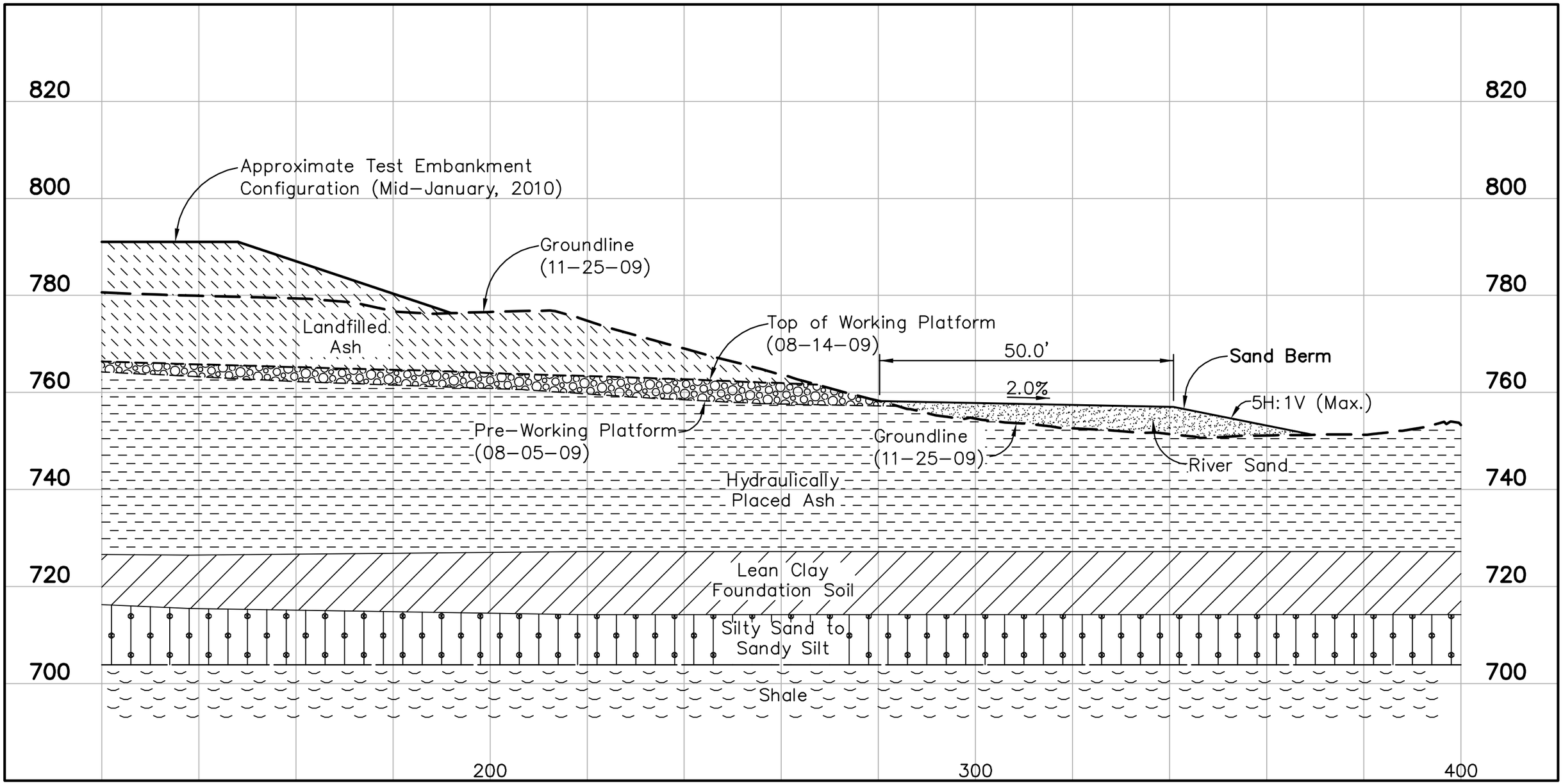
69015c–kif–101–gd1.DWG

50 0 100 200 FEET

GRAPHIC SCALE: 1" = 100'

PLOT DATE: 01/27/2010 USER: GRAHAM, DAVE
N:\7594\ACTIVE\75690015\ENVIRONMENTAL\DRAWING\TEST_EMB\SAND_BERM\69015c-KIF-101-001.DWG

D-1



SECTION – SAND BERM

SCALE: 1"=20'

68118c–kif–301–sa1.DWG

NOTE:

Place sand berm against the existing test embankment slope at the coordinates and elevations illustrated on the plan view.

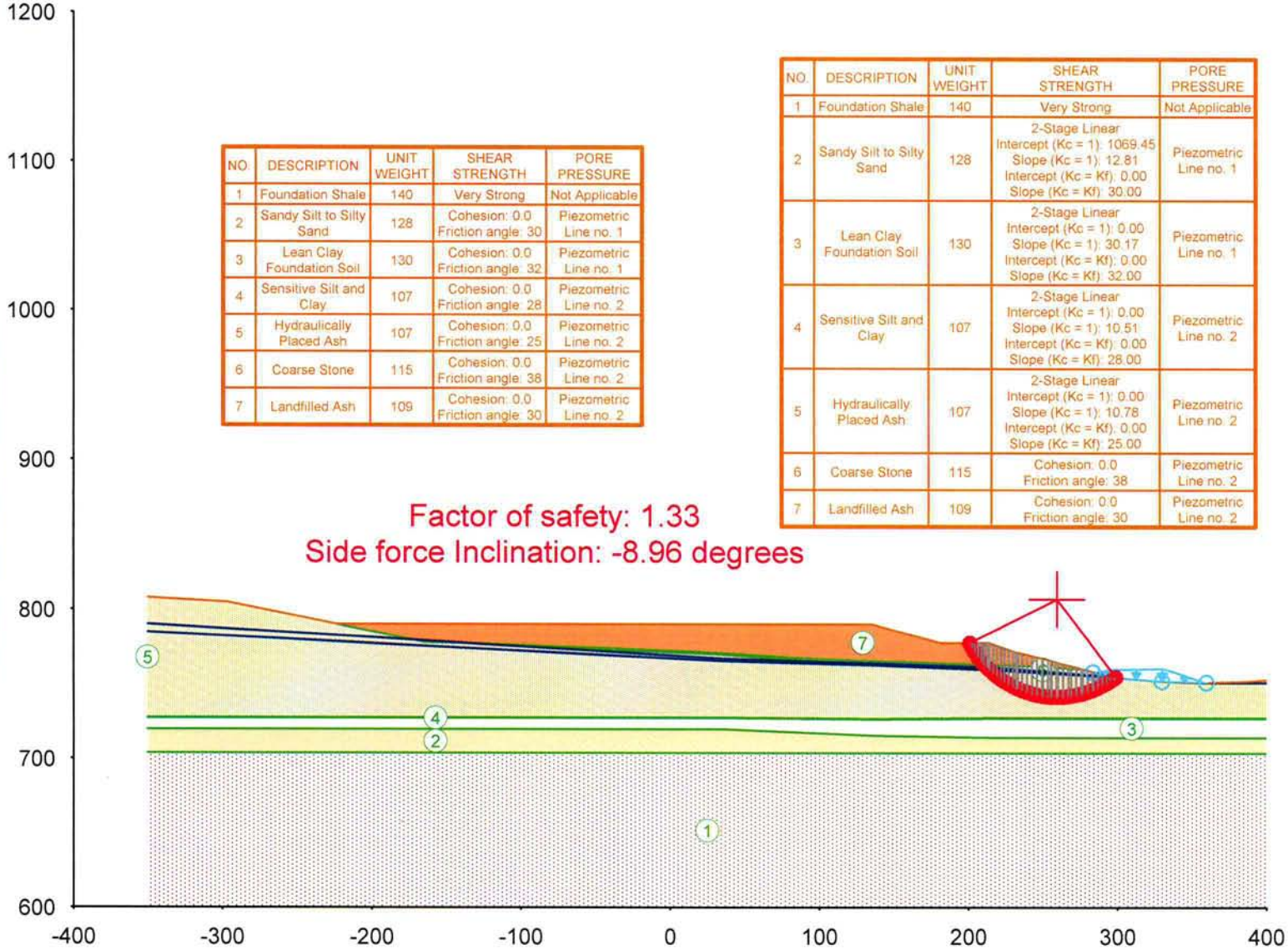
PLOT DATE: 01/27/2010 USER: GRAHAM, DAVE
V:\1756\ACTIVE\175669015\ENVIRONMENTAL\DRAWING\TEST_EMB\SAND_BERM\68118C–KIF–301–SA1.DWG

TVA Kingston Test Embankment - Section 2

NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Foundation Shale	140	Very Strong	Not Applicable
2	Sandy Silt to Silty Sand	128	Cohesion: 0.0 Friction angle: 30	Piezometric Line no. 1
3	Lean Clay Foundation Soil	130	Cohesion: 0.0 Friction angle: 32	Piezometric Line no. 1
4	Sensitive Silt and Clay	107	Cohesion: 0.0 Friction angle: 28	Piezometric Line no. 2
5	Hydraulically Placed Ash	107	Cohesion: 0.0 Friction angle: 25	Piezometric Line no. 2
6	Coarse Stone	115	Cohesion: 0.0 Friction angle: 38	Piezometric Line no. 2
7	Landfilled Ash	109	Cohesion: 0.0 Friction angle: 30	Piezometric Line no. 2

NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Foundation Shale	140	Very Strong	Not Applicable
2	Sandy Silt to Silty Sand	128	2-Stage Linear Intercept (Kc = 1): 1069.45 Slope (Kc = 1): 12.81 Intercept (Kc = Kf): 0.00 Slope (Kc = Kf): 30.00	Piezometric Line no. 1
3	Lean Clay Foundation Soil	130	2-Stage Linear Intercept (Kc = 1): 0.00 Slope (Kc = 1): 30.17 Intercept (Kc = Kf): 0.00 Slope (Kc = Kf): 32.00	Piezometric Line no. 1
4	Sensitive Silt and Clay	107	2-Stage Linear Intercept (Kc = 1): 0.00 Slope (Kc = 1): 10.51 Intercept (Kc = Kf): 0.00 Slope (Kc = Kf): 28.00	Piezometric Line no. 2
5	Hydraulically Placed Ash	107	2-Stage Linear Intercept (Kc = 1): 0.00 Slope (Kc = 1): 10.78 Intercept (Kc = Kf): 0.00 Slope (Kc = Kf): 25.00	Piezometric Line no. 2
6	Coarse Stone	115	Cohesion: 0.0 Friction angle: 38	Piezometric Line no. 2
7	Landfilled Ash	109	Cohesion: 0.0 Friction angle: 30	Piezometric Line no. 2

Factor of safety: 1.33
Side force Inclination: -8.96 degrees



Date: Mon Feb 01 2010 Filename: Section 2 - Existing - Sand Berm - Type 2 Search.UT4 Time: 09:52:38

Exhibit E

Drained Embankment Stability with Recommended Sand Berm upon Completion of Tier 2

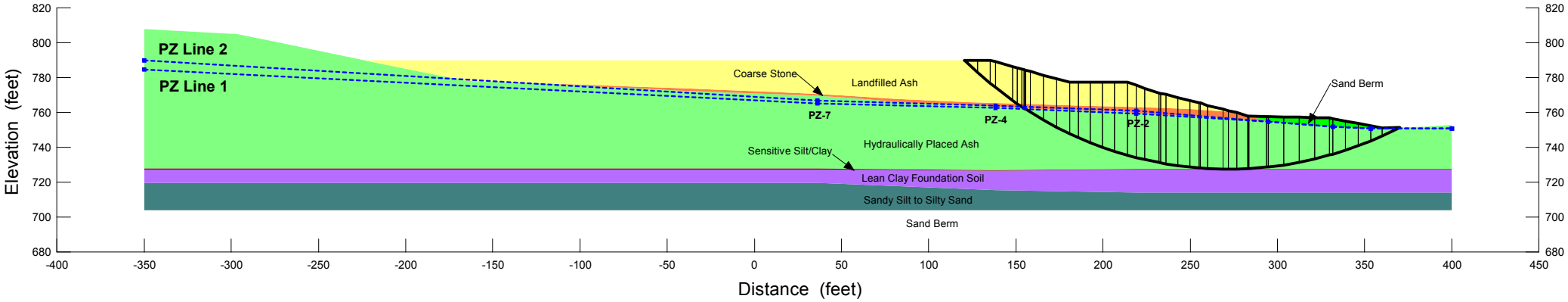
Slope Stability - Deep Failure Analysis
Test Embankment - Section 2
Trigger Event - Nov 1

Kingston Fossil Plant
Tennessee Valley Authority

Date Saved: 1/26/2010
Last Solved on 1/26/2010 at 11:06:24 AM
Method: Modified Spencer
File Name: Test Emb_Sec2_Slope_Stability (Existing with Berm3).gsz

Factor of Safety: 2.03
Center: (271.875, 942.375) ft
Radius: 214.675 ft
Minimum Slip Surface Depth: 1 ft

Material Type	Unit Weight		Cohesion	Friction Angle	PZ Line
	Saturated	Moist			
Landfilled Ash	111	109	0	30	2
Coarse Stone	128	115	0	38	2
Hydraulically Placed Ash	107	100	0	25	2
Sensitive Silt/Clay	107	100	0	28	2
Lean Clay Foundation Soil	130	130	0	32	1
Sandy Silt to Silty Sand	128	128	0	30	1
Sand Berm	110	110	0	30	1
Shale	Impenetrable				



Directory: \\Us1243-f01\workgroup\1756\active\175669015\environmental\analysis\Trigger Event Nov 1\Test Embankment Stability Runs\2010-01-11\Test Emb_Sec2_Slope_Stability (Existing with Berm3).gsz

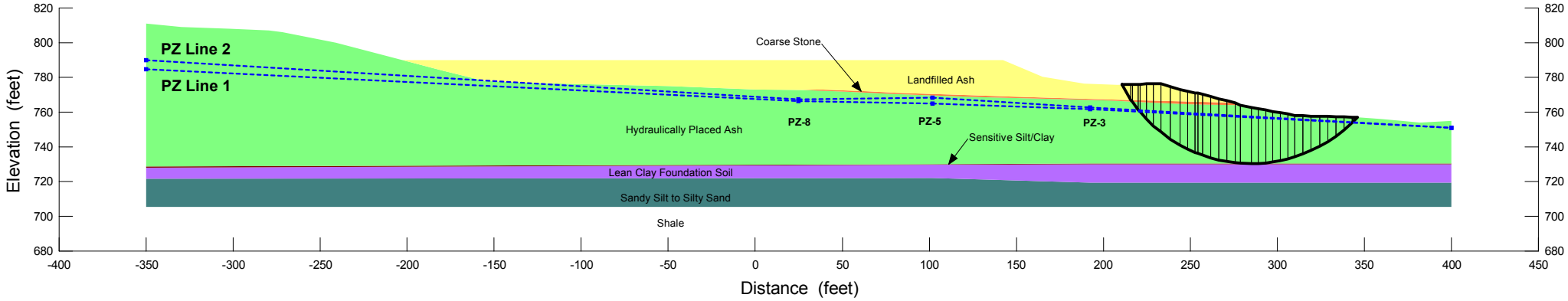
Slope Stability - Deep Failure Analysis
Test Embankment - Section 3
Trigger Event - Nov 1

Kingston Fossil Plant
Tennessee Valley Authority

Date Saved: 1/23/2010
Last Solved on 1/23/2010 at 4:23:09 PM
Method: Modified Spencer
File Name: Test Emb_Sec3_Slope_Stability (Existing).gsz

Material Type	Unit Weight		Cohesion	Friction Angle	PZ Line
	Saturated	Moist			
Landfilled Ash	111	109	0	30	2
Coarse Stone	128	115	0	38	2
Hydraulically Placed Ash	107	100	0	25	2
Sensitive Silt/Clay	107	100	0	28	2
Lean Clay Foundation Soil	130	130	0	32	1
Sandy Silt to Silty Sand	128	128	0	30	1
Shale	Impenetrable				1

Factor of Safety: 2.20
Center: (285, 813.75) ft
Radius: 83.45 ft
Minimum Slip Surface Depth: 1 ft



Directory: \\Us1243-f01\workgroup\1756\active\175669015\environmental\analysis\Trigger Event Nov 1\Test Embankment Stability Runs\2010-01-11\Test Emb_Sec3_Slope_Stability (Existing).gsz

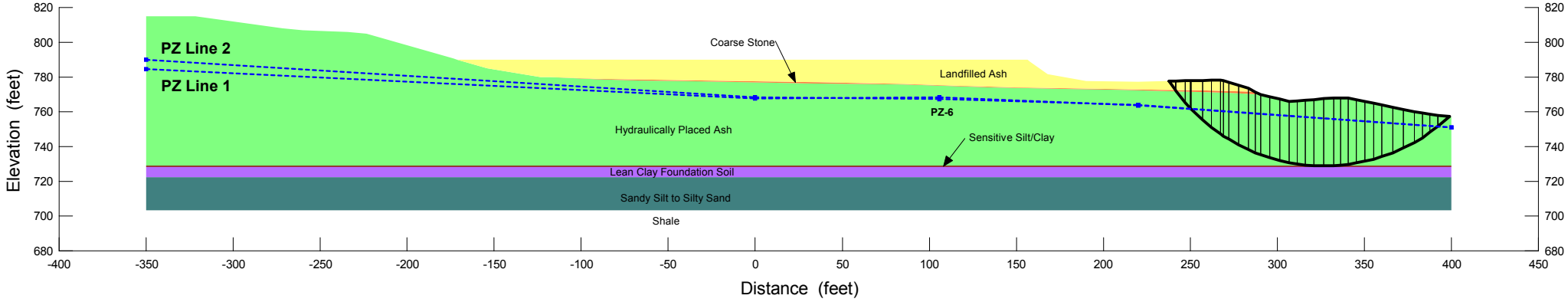
Slope Stability - Deep Failure Analysis
Test Embankment - Section 4
Trigger Event - Nov 1

Kingston Fossil Plant
Tennessee Valley Authority

Date Saved: 1/23/2010
Last Solved on 1/23/2010 at 5:10:15 PM
Method: Modified Spencer
File Name: Test Emb_Sec4_Slope_Stability (Existing).gsz

Material Type	Unit Weight		Cohesion	Friction Angle	PZ Line
	Saturated	Moist			
Landfilled Ash	111	109	0	30	2
Coarse Stone	128	115	0	38	2
Hydraulically Placed Ash	107	100	0	25	2
Sensitive Silt/Clay	107	100	0	28	2
Lean Clay Foundation Soil	130	130	0	32	1
Sandy Silt to Silty Sand	128	128	0	30	1
Shale	Impenetrable				1

Factor of Safety: 3.10
Center: (326.875, 834.375) ft
Radius: 105.575 ft
Minimum Slip Surface Depth: 1 ft



Directory: \\Us1243-f01\workgroup\1756\active\175669015\environmental\analysis\Trigger Event Nov 1\Test Embankment Stability Runs\2010-01-11\Test Emb_Sec4_Slope_Stability (Existing).gsz

Exhibit F

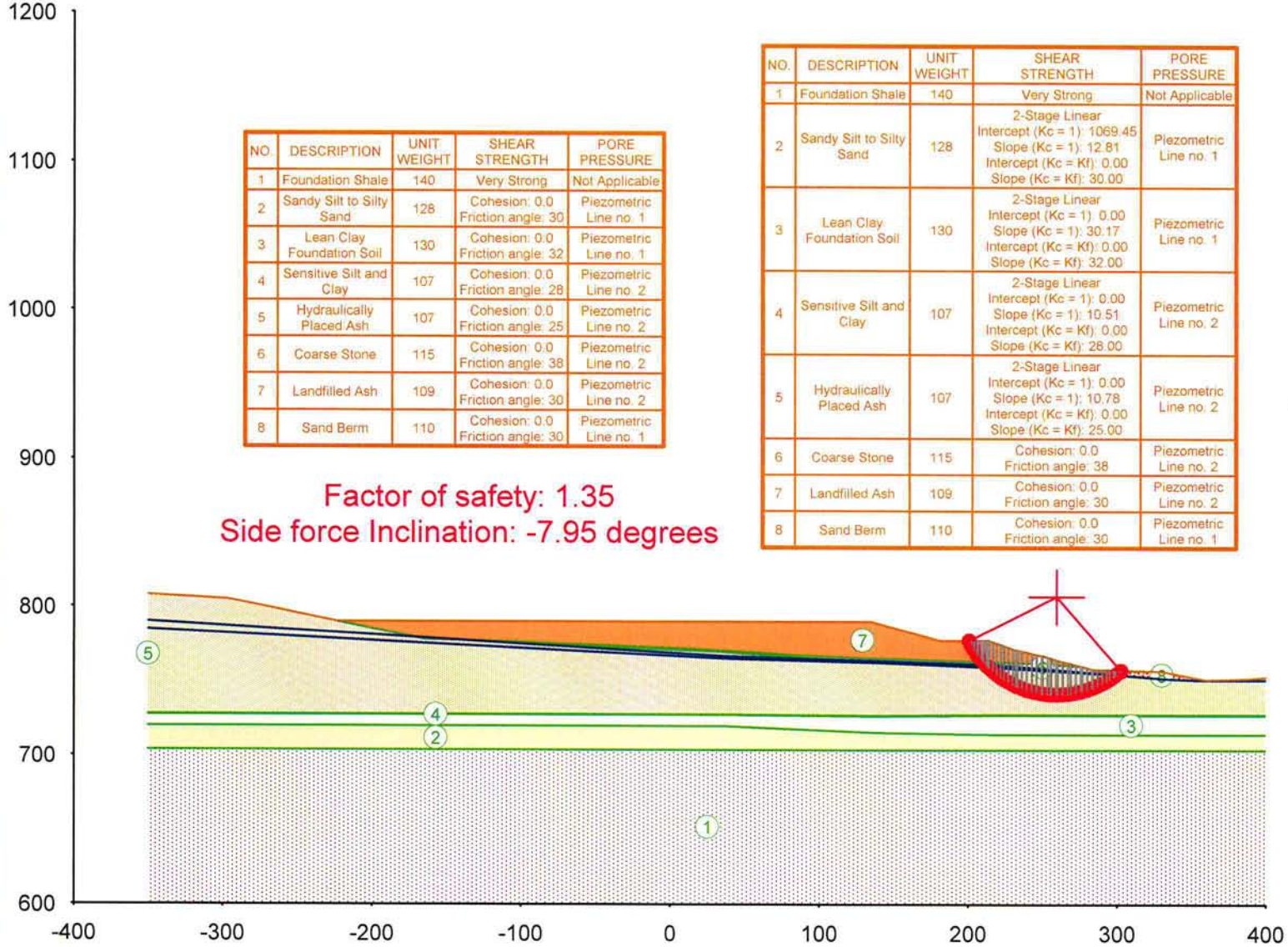
Undrained Embankment Stability with Recommended Sand Berm upon Completion of Tier 2

TVA Kingston Test Embankment - Section 2

NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Foundation Shale	140	Very Strong	Not Applicable
2	Sandy Silt to Silty Sand	128	Cohesion: 0.0 Friction angle: 30	Piezometric Line no. 1
3	Lean Clay Foundation Soil	130	Cohesion: 0.0 Friction angle: 32	Piezometric Line no. 1
4	Sensitive Silt and Clay	107	Cohesion: 0.0 Friction angle: 28	Piezometric Line no. 2
5	Hydraulically Placed Ash	107	Cohesion: 0.0 Friction angle: 25	Piezometric Line no. 2
6	Coarse Stone	115	Cohesion: 0.0 Friction angle: 38	Piezometric Line no. 2
7	Landfilled Ash	109	Cohesion: 0.0 Friction angle: 30	Piezometric Line no. 2
8	Sand Berm	110	Cohesion: 0.0 Friction angle: 30	Piezometric Line no. 1

NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Foundation Shale	140	Very Strong	Not Applicable
2	Sandy Silt to Silty Sand	128	2-Stage Linear Intercept (Kc = 1): 1069.45 Slope (Kc = 1): 12.81 Intercept (Kc = Kf): 0.00 Slope (Kc = Kf): 30.00	Piezometric Line no. 1
3	Lean Clay Foundation Soil	130	2-Stage Linear Intercept (Kc = 1): 0.00 Slope (Kc = 1): 30.17 Intercept (Kc = Kf): 0.00 Slope (Kc = Kf): 32.00	Piezometric Line no. 1
4	Sensitive Silt and Clay	107	2-Stage Linear Intercept (Kc = 1): 0.00 Slope (Kc = 1): 10.51 Intercept (Kc = Kf): 0.00 Slope (Kc = Kf): 28.00	Piezometric Line no. 2
5	Hydraulically Placed Ash	107	2-Stage Linear Intercept (Kc = 1): 0.00 Slope (Kc = 1): 10.78 Intercept (Kc = Kf): 0.00 Slope (Kc = Kf): 25.00	Piezometric Line no. 2
6	Coarse Stone	115	Cohesion: 0.0 Friction angle: 38	Piezometric Line no. 2
7	Landfilled Ash	109	Cohesion: 0.0 Friction angle: 30	Piezometric Line no. 2
8	Sand Berm	110	Cohesion: 0.0 Friction angle: 30	Piezometric Line no. 1

Factor of safety: 1.35
Side force Inclination: -7.95 degrees



Date: Fri Jan 29 2010 Filename: Section 2 - Existing w Berm - Type 2 Search.UT4 Time: 09:35:28

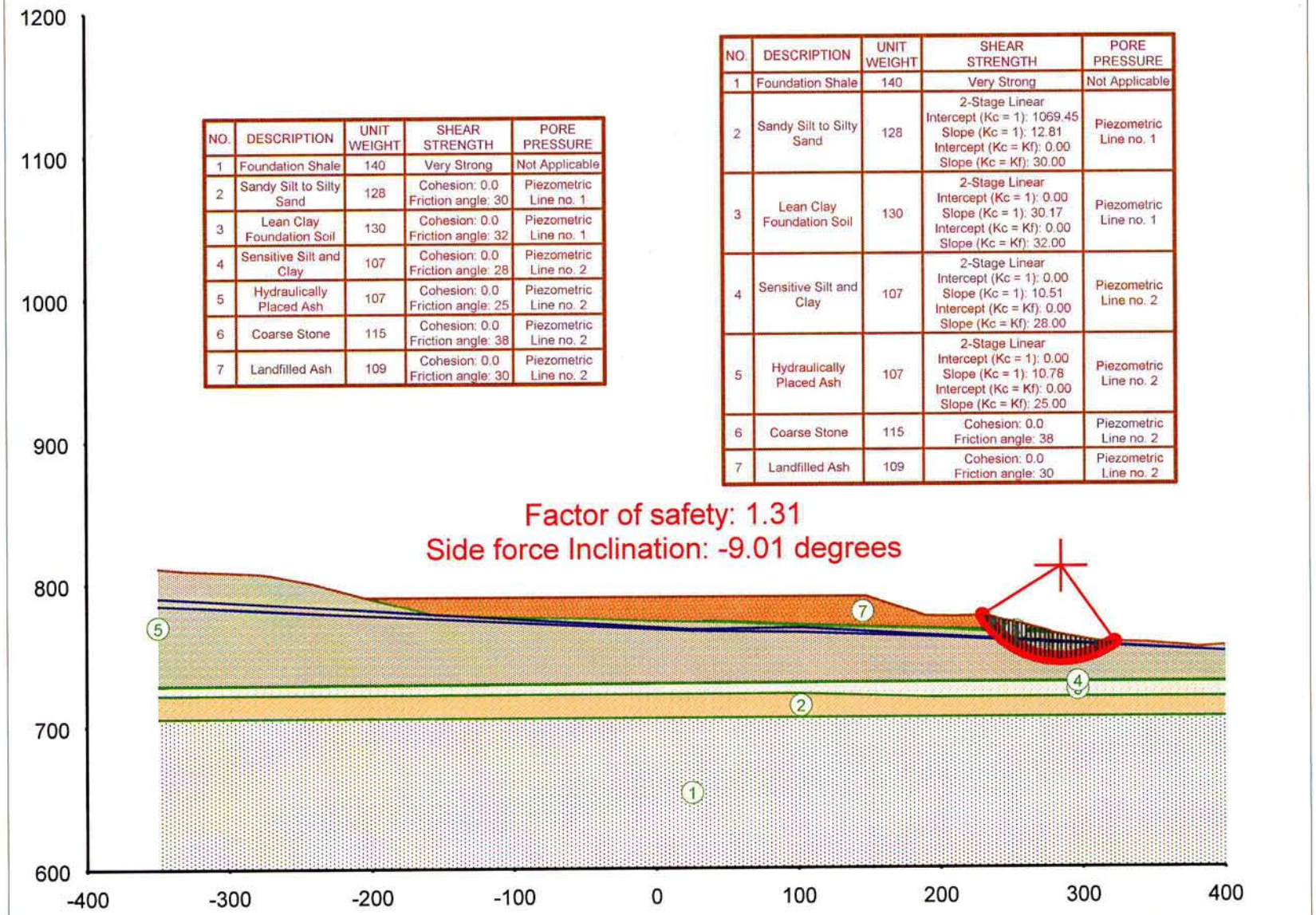
TVA Kingston Test Embankment - Section 3

NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Foundation Shale	140	Very Strong	Not Applicable
2	Sandy Silt to Silty Sand	128	Cohesion: 0.0 Friction angle: 30	Piezometric Line no. 1
3	Lean Clay Foundation Soil	130	Cohesion: 0.0 Friction angle: 32	Piezometric Line no. 1
4	Sensitive Silt and Clay	107	Cohesion: 0.0 Friction angle: 28	Piezometric Line no. 2
5	Hydraulically Placed Ash	107	Cohesion: 0.0 Friction angle: 25	Piezometric Line no. 2
6	Coarse Stone	115	Cohesion: 0.0 Friction angle: 38	Piezometric Line no. 2
7	Landfilled Ash	109	Cohesion: 0.0 Friction angle: 30	Piezometric Line no. 2

NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Foundation Shale	140	Very Strong	Not Applicable
2	Sandy Silt to Silty Sand	128	2-Stage Linear Intercept (Kc = 1): 1069.45 Slope (Kc = 1): 12.81 Intercept (Kc = Kf): 0.00 Slope (Kc = Kf): 30.00	Piezometric Line no. 1
3	Lean Clay Foundation Soil	130	2-Stage Linear Intercept (Kc = 1): 0.00 Slope (Kc = 1): 30.17 Intercept (Kc = Kf): 0.00 Slope (Kc = Kf): 32.00	Piezometric Line no. 1
4	Sensitive Silt and Clay	107	2-Stage Linear Intercept (Kc = 1): 0.00 Slope (Kc = 1): 10.51 Intercept (Kc = Kf): 0.00 Slope (Kc = Kf): 28.00	Piezometric Line no. 2
5	Hydraulically Placed Ash	107	2-Stage Linear Intercept (Kc = 1): 0.00 Slope (Kc = 1): 10.78 Intercept (Kc = Kf): 0.00 Slope (Kc = Kf): 25.00	Piezometric Line no. 2
6	Coarse Stone	115	Cohesion: 0.0 Friction angle: 38	Piezometric Line no. 2
7	Landfilled Ash	109	Cohesion: 0.0 Friction angle: 30	Piezometric Line no. 2

Factor of safety: 1.31

Side force Inclination: -9.01 degrees



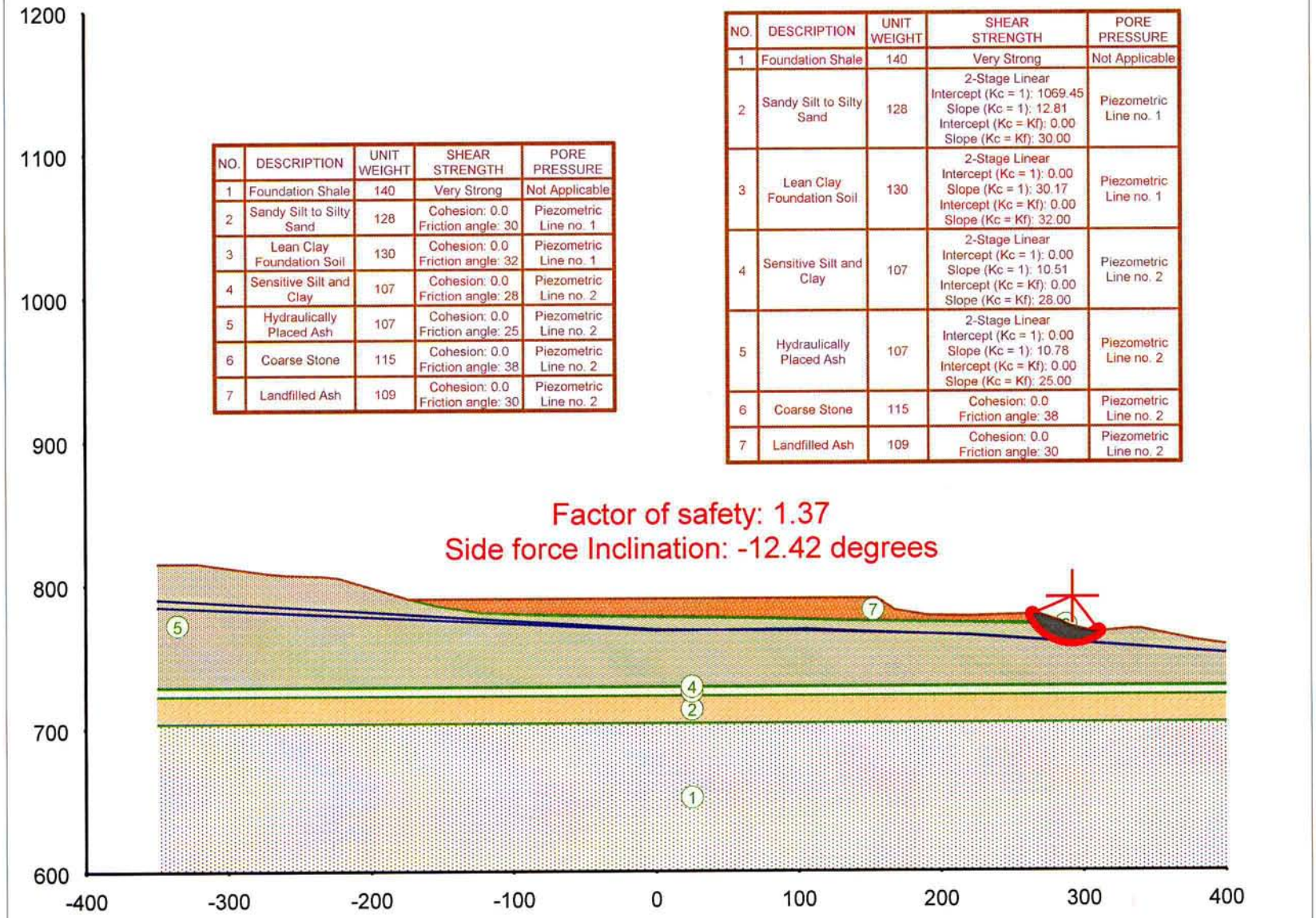
Date: Fri Jan 22 2010 Filename: Section 3 Type 2 Search.UT4 Time: 15:33:35

TVA Kingston Test Embankment - Section 4

NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Foundation Shale	140	Very Strong	Not Applicable
2	Sandy Silt to Silty Sand	128	Cohesion: 0.0 Friction angle: 30	Piezometric Line no. 1
3	Lean Clay Foundation Soil	130	Cohesion: 0.0 Friction angle: 32	Piezometric Line no. 1
4	Sensitive Silt and Clay	107	Cohesion: 0.0 Friction angle: 28	Piezometric Line no. 2
5	Hydraulically Placed Ash	107	Cohesion: 0.0 Friction angle: 25	Piezometric Line no. 2
6	Coarse Stone	115	Cohesion: 0.0 Friction angle: 38	Piezometric Line no. 2
7	Landfilled Ash	109	Cohesion: 0.0 Friction angle: 30	Piezometric Line no. 2

NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Foundation Shale	140	Very Strong	Not Applicable
2	Sandy Silt to Silty Sand	128	2-Stage Linear Intercept (Kc = 1): 1069.45 Slope (Kc = 1): 12.81 Intercept (Kc = Kf): 0.00 Slope (Kc = Kf): 30.00	Piezometric Line no. 1
3	Lean Clay Foundation Soil	130	2-Stage Linear Intercept (Kc = 1): 0.00 Slope (Kc = 1): 30.17 Intercept (Kc = Kf): 0.00 Slope (Kc = Kf): 32.00	Piezometric Line no. 1
4	Sensitive Silt and Clay	107	2-Stage Linear Intercept (Kc = 1): 0.00 Slope (Kc = 1): 10.51 Intercept (Kc = Kf): 0.00 Slope (Kc = Kf): 28.00	Piezometric Line no. 2
5	Hydraulically Placed Ash	107	2-Stage Linear Intercept (Kc = 1): 0.00 Slope (Kc = 1): 10.78 Intercept (Kc = Kf): 0.00 Slope (Kc = Kf): 25.00	Piezometric Line no. 2
6	Coarse Stone	115	Cohesion: 0.0 Friction angle: 38	Piezometric Line no. 2
7	Landfilled Ash	109	Cohesion: 0.0 Friction angle: 30	Piezometric Line no. 2

Factor of safety: 1.37
Side force Inclination: -12.42 degrees



Date: Fri Jan 22 2010 Filename: Section 4 Type 2 Search.UT4 Time: 15:44:28

Exhibit G

Undrained Embankment Stability with Recommended Sand Berm upon Completion of Tier 3

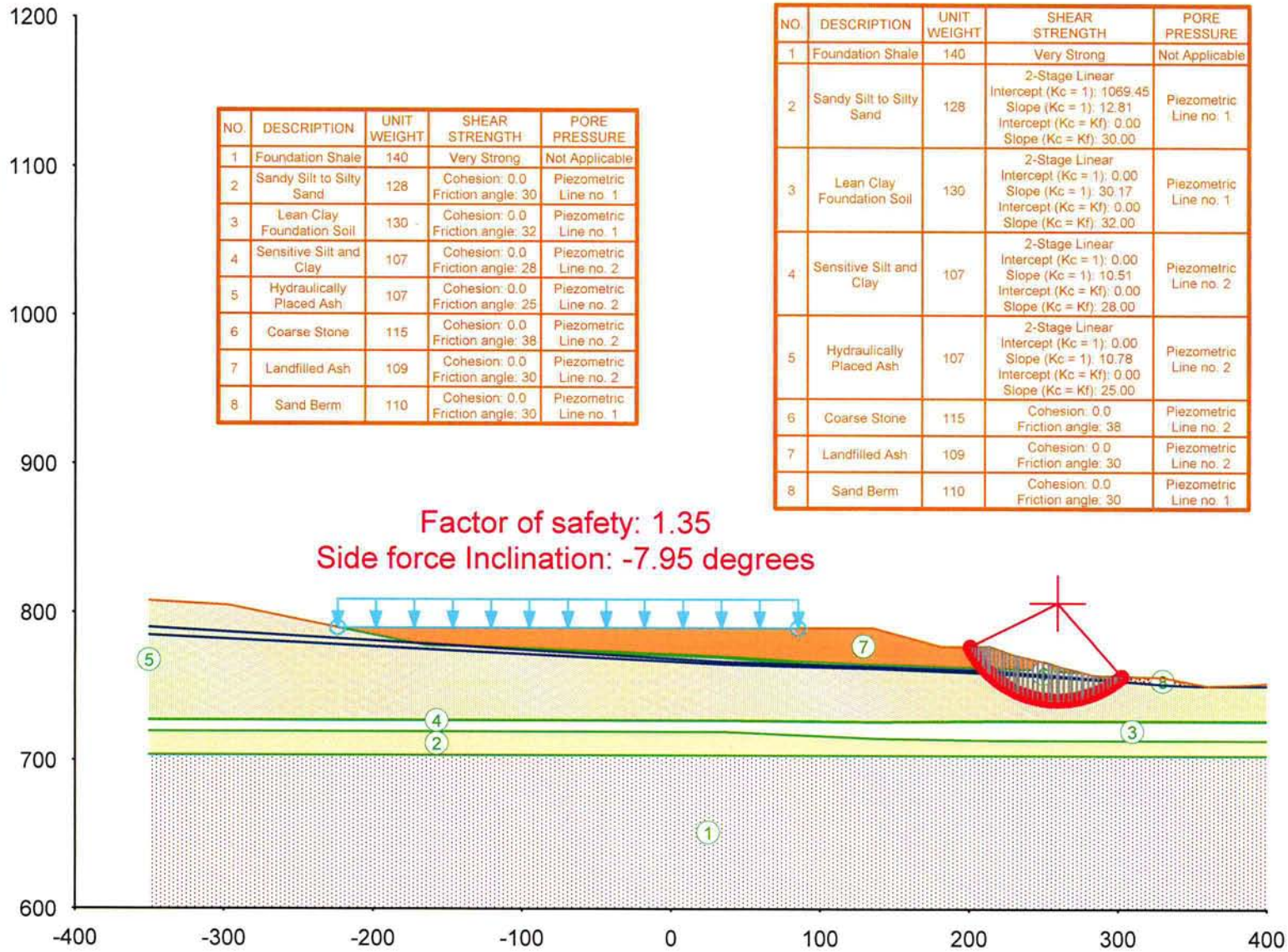
TVA Kingston Test Embankment - Section 2

NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Foundation Shale	140	Very Strong	Not Applicable
2	Sandy Silt to Silty Sand	128	Cohesion: 0.0 Friction angle: 30	Piezometric Line no. 1
3	Lean Clay Foundation Soil	130	Cohesion: 0.0 Friction angle: 32	Piezometric Line no. 1
4	Sensitive Silt and Clay	107	Cohesion: 0.0 Friction angle: 28	Piezometric Line no. 2
5	Hydraulically Placed Ash	107	Cohesion: 0.0 Friction angle: 25	Piezometric Line no. 2
6	Coarse Stone	115	Cohesion: 0.0 Friction angle: 38	Piezometric Line no. 2
7	Landfilled Ash	109	Cohesion: 0.0 Friction angle: 30	Piezometric Line no. 2
8	Sand Berm	110	Cohesion: 0.0 Friction angle: 30	Piezometric Line no. 1

NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Foundation Shale	140	Very Strong	Not Applicable
2	Sandy Silt to Silty Sand	128	2-Stage Linear Intercept (Kc = 1): 1069.45 Slope (Kc = 1): 12.81 Intercept (Kc = Kf): 0.00 Slope (Kc = Kf): 30.00	Piezometric Line no. 1
3	Lean Clay Foundation Soil	130	2-Stage Linear Intercept (Kc = 1): 0.00 Slope (Kc = 1): 30.17 Intercept (Kc = Kf): 0.00 Slope (Kc = Kf): 32.00	Piezometric Line no. 1
4	Sensitive Silt and Clay	107	2-Stage Linear Intercept (Kc = 1): 0.00 Slope (Kc = 1): 10.51 Intercept (Kc = Kf): 0.00 Slope (Kc = Kf): 28.00	Piezometric Line no. 2
5	Hydraulically Placed Ash	107	2-Stage Linear Intercept (Kc = 1): 0.00 Slope (Kc = 1): 10.78 Intercept (Kc = Kf): 0.00 Slope (Kc = Kf): 25.00	Piezometric Line no. 2
6	Coarse Stone	115	Cohesion: 0.0 Friction angle: 38	Piezometric Line no. 2
7	Landfilled Ash	109	Cohesion: 0.0 Friction angle: 30	Piezometric Line no. 2
8	Sand Berm	110	Cohesion: 0.0 Friction angle: 30	Piezometric Line no. 1

Factor of safety: 1.35

Side force Inclination: -7.95 degrees



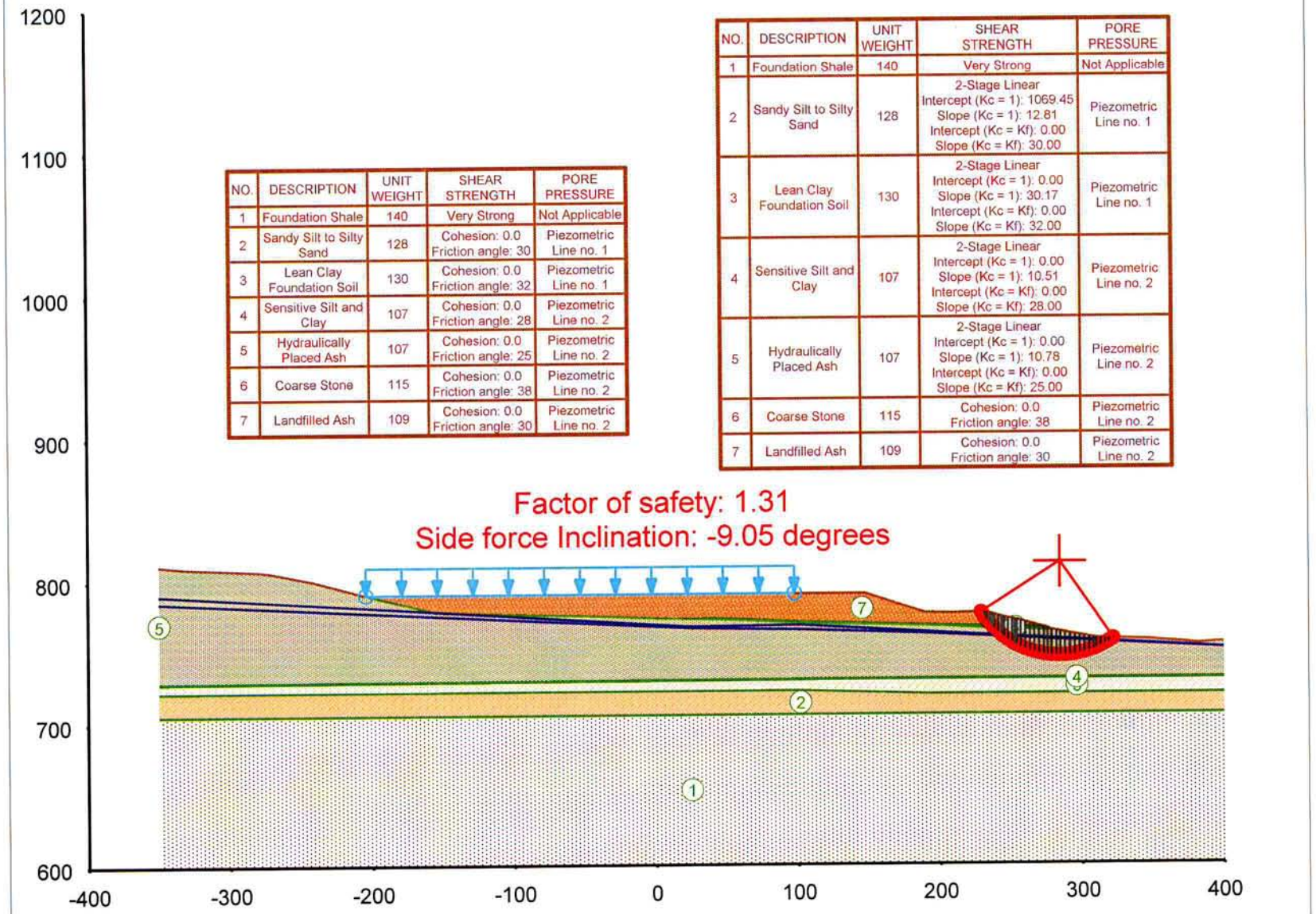
Date: Fri Jan 29 2010 Filename: Section 2 - Existing w Berm - 11 ft Load - Type 2 Search.UT4 Time: 09:44:28

TVA Kingston Test Embankment - Section 3

NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Foundation Shale	140	Very Strong	Not Applicable
2	Sandy Silt to Silty Sand	128	Cohesion: 0.0 Friction angle: 30	Piezometric Line no. 1
3	Lean Clay Foundation Soil	130	Cohesion: 0.0 Friction angle: 32	Piezometric Line no. 1
4	Sensitive Silt and Clay	107	Cohesion: 0.0 Friction angle: 28	Piezometric Line no. 2
5	Hydraulically Placed Ash	107	Cohesion: 0.0 Friction angle: 25	Piezometric Line no. 2
6	Coarse Stone	115	Cohesion: 0.0 Friction angle: 38	Piezometric Line no. 2
7	Landfilled Ash	109	Cohesion: 0.0 Friction angle: 30	Piezometric Line no. 2

NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Foundation Shale	140	Very Strong	Not Applicable
2	Sandy Silt to Silty Sand	128	2-Stage Linear Intercept (Kc = 1): 1069.45 Slope (Kc = 1): 12.81 Intercept (Kc = Kf): 0.00 Slope (Kc = Kf): 30.00	Piezometric Line no. 1
3	Lean Clay Foundation Soil	130	2-Stage Linear Intercept (Kc = 1): 0.00 Slope (Kc = 1): 30.17 Intercept (Kc = Kf): 0.00 Slope (Kc = Kf): 32.00	Piezometric Line no. 1
4	Sensitive Silt and Clay	107	2-Stage Linear Intercept (Kc = 1): 0.00 Slope (Kc = 1): 10.51 Intercept (Kc = Kf): 0.00 Slope (Kc = Kf): 28.00	Piezometric Line no. 2
5	Hydraulically Placed Ash	107	2-Stage Linear Intercept (Kc = 1): 0.00 Slope (Kc = 1): 10.78 Intercept (Kc = Kf): 0.00 Slope (Kc = Kf): 25.00	Piezometric Line no. 2
6	Coarse Stone	115	Cohesion: 0.0 Friction angle: 38	Piezometric Line no. 2
7	Landfilled Ash	109	Cohesion: 0.0 Friction angle: 30	Piezometric Line no. 2

Factor of safety: 1.31
Side force Inclination: -9.05 degrees

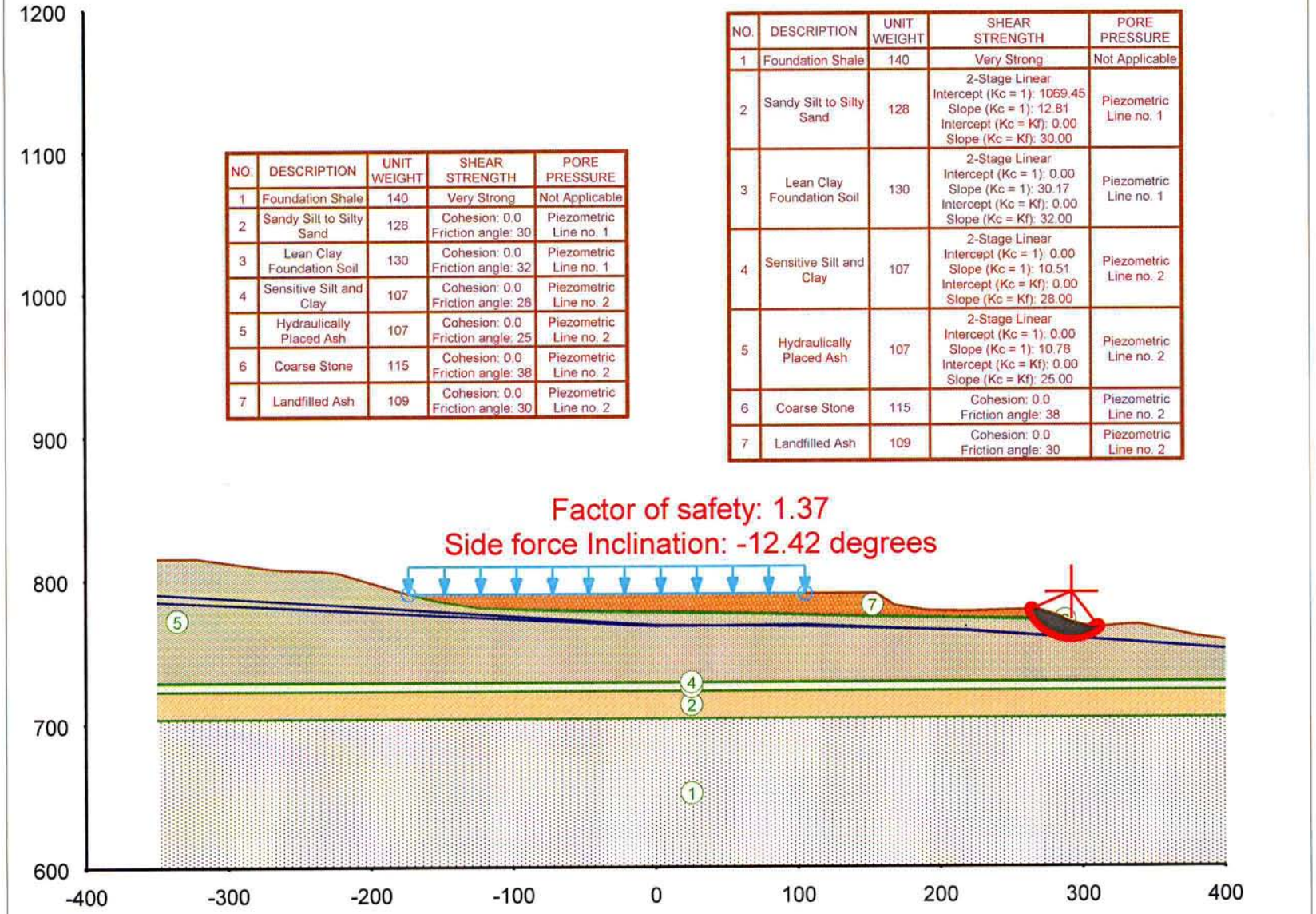


Date: Fri Jan 22 2010 Filename: Section 3 - 11 ft Load Type 2 Search.UT4 Time: 15:46:26

TVA Kingston Test Embankment - Section 4

NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Foundation Shale	140	Very Strong	Not Applicable
2	Sandy Silt to Silty Sand	128	Cohesion: 0.0 Friction angle: 30	Piezometric Line no. 1
3	Lean Clay Foundation Soil	130	Cohesion: 0.0 Friction angle: 32	Piezometric Line no. 1
4	Sensitive Silt and Clay	107	Cohesion: 0.0 Friction angle: 28	Piezometric Line no. 2
5	Hydraulically Placed Ash	107	Cohesion: 0.0 Friction angle: 25	Piezometric Line no. 2
6	Coarse Stone	115	Cohesion: 0.0 Friction angle: 38	Piezometric Line no. 2
7	Landfilled Ash	109	Cohesion: 0.0 Friction angle: 30	Piezometric Line no. 2

NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Foundation Shale	140	Very Strong	Not Applicable
2	Sandy Silt to Silty Sand	128	2-Stage Linear Intercept (Kc = 1): 1069.45 Slope (Kc = 1): 12.81 Intercept (Kc = Kf): 0.00 Slope (Kc = Kf): 30.00	Piezometric Line no. 1
3	Lean Clay Foundation Soil	130	2-Stage Linear Intercept (Kc = 1): 0.00 Slope (Kc = 1): 30.17 Intercept (Kc = Kf): 0.00 Slope (Kc = Kf): 32.00	Piezometric Line no. 1
4	Sensitive Silt and Clay	107	2-Stage Linear Intercept (Kc = 1): 0.00 Slope (Kc = 1): 10.51 Intercept (Kc = Kf): 0.00 Slope (Kc = Kf): 28.00	Piezometric Line no. 2
5	Hydraulically Placed Ash	107	2-Stage Linear Intercept (Kc = 1): 0.00 Slope (Kc = 1): 10.78 Intercept (Kc = Kf): 0.00 Slope (Kc = Kf): 25.00	Piezometric Line no. 2
6	Coarse Stone	115	Cohesion: 0.0 Friction angle: 38	Piezometric Line no. 2
7	Landfilled Ash	109	Cohesion: 0.0 Friction angle: 30	Piezometric Line no. 2



Date: Fri Jan 22 2010 Filename: Section 4 - 11 ft Load Type 2 Search.UT4 Time: 16:05:12