Prepared for

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SUPPLEMENTAL ASSESSMENT OF SEEPAGE AND SLOPE STABILITY

KINGSTON FOSSIL PLANT
EAST DIKE

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1. PURPOSE

The purpose of this Supplemental Assessment of Seepage and Slope Stability Report (Report) is to provide results and recommendations regarding seepage and slope stability in previously identified areas of concern along the East Dike and adjacent to the Intake Channel at the Tennessee Valley authority (TVA) Kingston Fossil Plant (KIF). The calculations presented in this package were prepared in consideration of newly acquired subsurface data. This Report is considered a supplement to the following three previous studies performed by Geosyntec Consultants (Geosyntec): (i) *Seepage and Stability Study for East Dike and Raised Dike* (Geosyntec, 2010a), hereinafter referred to as the "South End Study", ii) *Seepage and Stability Study for North End of East Dike* (Geosyntec, 2010b), hereinafter referred to as the "North End Study", and iii) *Re-assessment of East Dike Stability* presented orally to TVA in May 2011, hereinafter referred to as "Presentation".

2. BACKGROUND

The KIF is located on the Watts Bar Reservoir at the confluence of the Emory River and Clinch River in Harriman, Tennessee. The East Dike is on the far eastern edge of a portion of land that includes the Ball field Site, a former recreational facility constructed over deposited fly ash and the existing sluice channel. The area of the East Dike showing the Ball-field Site and the sluice channel is shown in Figure 1.

A narrow haul road constructed on the crest of the East Dike varies from approximate elevation (el) 755 feet mean sea level (MSL) at the southern end to el 748 feet at the northern end. Winter pool elevation of Watts Bar Lake is approximately el 737 feet; summer pool is approximately el 740 feet. Sideslope grades between the crest of the East Dike and Watts Bar Lake vary gradually along the length of the East Dike with the southern end being steeper (i.e., approximately 5 horizontal to 1 vertical (5H:1V)) than slopes on the northern end (i.e., approximately 10H:1V). An approximately el 2H:1V slope believed to be constructed primarily of fly ash extends approximately 10 feet above the haul road. Therefore, it appears that the East Dike separates Watts Bar Lake from the historic fly ash deposits that were deposited landside of the dike.

TVA reports that groundwater seeps have been observed in the southern section of East Dike along the face of the dike and along the face of the slope facing the haul road. The location and extent of these seeps were documented in the South End Study. In fact, the observed seepage along the East Dike was one of the reasons that TVA initiated both the South End Study and North End Study, although TVA reported that no seeps had previously been reported in the northern section.

The South End Study considered two cross sections along the East Dike, specifically referenced as cross section A and cross section B. The North End Study considered two additional cross sections referenced as cross section C and cross section D. To obtain subsurface information needed for these analyses, a total of ten standard Penetration Test (SPT) borings were advanced by MACTEC to auger refusal depths. The borings included continuous sampling to obtain either split-spoon or Shelby tube samples. A laboratory program was conducted consisting of moisture content, grain size distribution, soil classification, Atterberg limits, and consolidated undrained triaxial testing. Complete documentation of the SPT borings and laboratory testing for the South End Study and the North End Study are presented in the respective reports. The remainder of this section briefly describes the findings and recommendations presented in each of the three previously completed studies.

2.1 South End Study – Findings and Recommendations

Using subsurface information from previous investigations in the vicinity of the East Dike, seepage and slope stability analyses were performed by Geosyntec. Seepage analysis performed for the South End Study along cross section A indicated that a shallow phreatic surface exists within the East Dike. This shallow water surface was confirmed by the observation of seeps along the toe of the East Dike. Calculated factors of safety (FS) for a piping failure were approximately 3.0. This calculated FS value was considered appropriate by Geosyntec at the time given that the range of the calculated FS values recommended in the US Army Corps of Engineers Manual 1110-2-1901 (USACE 1986) was 1.5 to 15 and the fine-grained materials encountered within the East Dike that were believed to exhibit a low potential for piping.

Results of global stability analysis indicated that the calculated FS for the southern portion of the East Dike decreased from 1.53 to 1.20 when the pore pressure within the East Dike Lower Fill layer was increased from 0.5 ft to 2.0 ft above the ground surface. This increase in pore pressure was anticipated to capture the potential worst case conditions within the East Dike. Therefore, in the absence of additional information regarding the potential ranges of pore pressures and to improve the calculated stability of the East Dike at the potential worst case pore water pressure condition (i.e., elevated to 2 ft above the ground surface), Geosyntec recommended that TVA construct a rock blanket along the face of the East Dike adjacent to the intake channel.

2.2 North End Study – Findings and Recommendations

Using available subsurface information obtained previously in the vicinity of the East Dike, seepage and slope stability analyses were performed. Seepage analysis results performed along cross section D indicated that the calculated FS against a piping failure was approximately 1.96. The North End Study cited the *TVA Master Programmatic Document* in the selection of the minimum required FS against piping of 4.0, which was more restrictive that the previously reference USACE guidance. It was noted that the piezometer data contributing to this calculation were collected during the summer/fall season and combined with a winter pool elevation for the intake channel of 737 feet to model the worst case scenario. Geosyntec recommended regular monitoring of the piezometers in the East Dike during the winter season to determine the seasonal fluctuation of the groundwater table, if any. The additional piezometric data could then be used to determine if the seepage model accurately reflected the maximum measured hydraulic gradient.

Stability analysis results presented as part of the North End Study indicated that the East Dike is stable with respect to deep seated failures with a calculated factor of safety of 1.80, and meets the requirements for minimum factors of safety of 1.50.

2.3 Presentation – Findings and Recommendations

In May 2011, Geosyntec made an oral presentation to TVA personnel to report findings and interpretations regarding seepage and stability analysis results considering piezometer readings obtained along the East Dike throughout the winter/spring season. During the review of previous work, a discrepancy was discovered between laboratory measured strength values and observed in-situ test results. This discrepancy led Geosyntec to re-assess the stability of the East Dike in both the southern and northern portions.

Specifically, Geosyntec noted that stability analysis results based on the existing SPT in-situ data yielded results for global stability that were below the minimum required FS for a deep-seated failure, while stability analysis results based on laboratory measured strength values demonstrated that the calculated global stability FS values were greater than the minimum required values for deep seated failure. To address this discrepancy, Geosyntec recommended the execution of a supplemental field testing program consisting of a series of piezocone penetration test (CPTu) soundings to obtain a more reliable understanding of the subsurface conditions at the East Dike.

Additionally, Geosyntec recommended draining the red water pond and monitoring the response of a piezometer located at cross section D. However, this recommendation was not implemented at this time due to permitting restrictions.

3. SUPPLEMENTAL GEOTECHNICAL INVESTIGATION PROGRAM

A supplemental geotechnical investigation program was commissioned by TVA as a result of the recommendation in the Presentation to TVA. This program consisted of 12 total CPTu soundings that were performed by ConeTec along the length of the East Dike. CPTu soundings were performed at the location of the four previously identified cross sections A through D, as well as at intermediate locations between these cross section locations to aid in the assessment of conditions between the analyzed cross sections. Figure 1 shows the locations of the CPTu soundings along the East Dike. Several adjustments were made in the field to the CPTu sounding locations due to access restrictions and shallow penetration refusals. Figure 2 shows the final CPTu sounding locations.

The CPTu's were performed using an integrated electronic seismic piezocone that had a 15 cm^2 tip and a 225 cm^2 friction sleeve. The cone was advanced using a 20-ton track-mounted rig. Cone tip resistance, sleeve friction and pore pressures were recorded at approximately 2-inch vertical intervals throughout the depth of advancement. At each approximate 1-meter vertical interval, advancement was stopped so that an additional rod could be added. During the time period, the rate of pore water pressure dissipation was measure for a time interval of up to 5 minutes to aid in the assessment of in-situ hydraulic conductivity. Attachment A includes the report and sounding records developed by ConeTec.

4. TOPOGRAPHY, SUBSURFACE STRATIGRAPHY AND MATERIAL PROPERTIES

4.1 Topography

Recent topographic data were provided by TVA. These data were combined with bathymetric survey results to obtain a soil surface profile for the East Dike and adjacent portions of Watts Bar Lake. Figure 2 presents the topography and bathymetry in the vicinity of the East Dike including the CPTu and cross sections locations.

4.2 Subsurface Stratigraphy

Using information captured as the cone was advanced through the material of the East Dike, the soils were classified using ConeTec software that incorporated the normalized behavior type classification chart (Robertson 1986). This information was combined with data collected during the previous SPT field investigations performed at the site (Geosyntec 2010a, 2010b) to develop an updated interpretation of the subsurface stratigraphy. Figure 3 shows an interpreted subsurface profile at the CPTu locations. The stratigraphic delineations shown on Figure 3 represent Geosyntec's interpretation of the transition between different soil layers.

4.3 Shear Strength

The shear strength at each CPTu data point was assessed using correlations based on the measured values for tip resistance and sleeve friction. The following correlation to drained shear strength for frictional materials (i.e., sands and gravels) is given in the "Manual on Estimating Soil Properties for Foundation Design" by Kulhawy and Mayne (1990).

$$\phi' = \tan^{-1}\left(0.1 + 0.38\log\left(\frac{q_c}{\sigma'_{vo}}\right)\right) \qquad (1)$$

Where: φ' = effective stress friction angle, q_c = cone tip resistance and σ'_{vo} = vertical effective stress. This correlation was used for: (i) the Upper Dike Fill layer; (ii) the Alluvial Sandy Silt layer; and (iii) Sand & Gravel layer shown in Figure 3.

For the Lower Dike Fill layer shown in Figure 3, a ratio of undrained shear strength to vertical effective stress was plotted with depth as shown in Figures 4 and 5. These figures also indicate the value of the ratio selected for analysis. This strength model was used in lieu of a static undrained shear strength model so that the sensitivity of the slope stability model could be better assessed with changes in groundwater pore pressures.

The following correlation to shear strength for cohesive materials (i.e., silts and clays) is given by Robertson (1986), among others and was used to assess the undrained shear strength for: (i) the Alluvial Clayey Silt layer; and (ii) the Alluvial Silty Clay layer shown in Figure 3.

$$S_u = \frac{q_t - \sigma_v}{N_{kt}} \qquad (2)$$

Where: S_u = undrained shear strength, q_t = corrected tip resistance, σ_v = total vertical stress, N_{kt} = cone factor which generally ranges from 12 to 20. N_{kt} = 15 was selected for the calculation of the undrained shear strength based on comparison with the laboratory determined shear strength values that were obtained during the South and North End Study.

Equations 1 and 2 were incorporated in the ConeTec software and shear strengths were provided for each soil layer. Table 1 presents the range of values obtained from the CPTu soundings at cross sections A, B, C and D as well as the selected shear strength values to be used during analysis.

The results shown in Table 1 agree well with laboratory determined shear strength values that were obtained during the South and North End Study. The CPT results obtained as a part of this Report therefore support the opinion that the low blow counts obtained during the South and North End Study SPT drilling and sampling are the product of the saturated conditions of the subsurface combined with the disturbance to these materials caused during the act of drilling.

4.4 Hydraulic Conductivity

Two analysis methods based on CPTu data were used to assess hydraulic conductivity within the soil layers of the East Dike. One method is based on a correlation to soil behavior type and a second method is based on pore pressure dissipation test results.

The first method includes a correlation of hydraulic conductivity to the soil behavior type index, $I_{c,}$ as defined by Robertson (2009), where,

$$I_c = \left[(3.47 - \log Q_{tn})^2 + (\log F_r + 1.22)^2 \right]^{0.5} \quad (3)$$

Where: $Q_{tn} = \left(\frac{q_t - \sigma_v}{p_a} \right) \left(\frac{p_a}{\sigma'_{vo}} \right)^n$ and $F_r = \left(\frac{f_s}{q_t - \sigma_{vo}} \right) * 100\%$

In these equations, q_t = CPT corrected total cone resistance, f_s = CPT sleeve friction, σ_{vo} = in-situ total vertical stress, σ'_{vo} = in-situ effective vertical stress, $(qt - \sigma_v)/p_a$ = dimensionless net cone resistance, $(p_a/\sigma'_{vo})^n$ = stress normalization factor, n = stress exponent that varies with soil behavior type, and p_a = atmospheric pressure.

From the calculated quantity I_c, an approximate value for hydraulic conductivity can be calculated by the following equations.

$k = 10^{(0.952 - 3.04 * I_c)} (m/s)$ when $1.0 < I_c < 3.27$, and (4)

$k = 10^{(-4.52 - 1.37 * I_c)} (m/s)$ when $I_c > 3.27$. (5)

While this method is approximate, it provides calculation results at each approximate 2-inch depth interval. Therefore this method is useful for a point-by-point comparison, as well as for overall assessment of each layer.

The second method used to assess in-situ hydraulic conductivity is based on a correlation to the coefficient of consolidation in the horizontal direction (c_h) that is calculated using pore pressure dissipation test results obtained at each rod break during the field investigation program. In a balance of production rate and cost, dissipation tests were limited to 5 minutes in duration. If the time required for dissipation of 50 percent of the excess pore pressure buildup (t_{50}) due to cone penetration is captured within the 5 minute window, the calculation of a hydraulic conductivity is possible based on the following general equation:

$$k_h = (c_h * \gamma_w)/M \quad (6)$$

Where: k_h = horizontal hydraulic conductivity, c_h = coefficient of consolidation in the horizontal direction, γ_w = unit weight of water and M = 1-D constrained modulus. The value of c_h is calculated using t_{50} as documented in Robertson (1992).

$$c_h = (1.67 * 10^{-6})(10^{1-\log t_{50}})(m^2/s) \quad (7)$$

Robertson (2009) correlates the 1-D constraint modulus also using CPTu results as defined below.

$$M = \alpha_m (q_t - \sigma_{vo}) \quad (8)$$

Where: α_m = Q_{tn} (normalized cone tip resistance) when $Q_{tn} \leq 14$, and α_m = 14 when $Q_{tn} > 14$.

Using equations 3 through 8, the hydraulic conductivity of the subsurface layers below the East Dike was calculated along each of the four cross sections. Table 2 summarizes the subsurface layers and calculated hydraulic conductivities from the two referenced methods. The selected hydraulic conductivity used for analysis for each layer is given in Table 3. Figures 6 through 9 show the relationship of the calculated hydraulic conductivities with depth for the CPTu tests performed at cross section A, B, C, and D. The ratio of the coefficient of hydraulic conductivity in the horizontal and vertical directions was 10 based on information presented in Geosyntec (2010a, b). Sensitivity analyses were performed and demonstrated that calculations results are relatively insensitive to adjustments in the ratio.

4.5 <u>Water Levels</u>

The North End Study included a recommendation that additional piezometer data be collected in recognition of the fact that the results of the previous seepage analyses were obtained using water level data recorded during the summer/fall season combined with the approximate winter pool elevation of 737 feet at the intake channel. Figure 10 displays the piezometer data collected throughout the fall of 2010 and into the winter of

2011. Approximately half of the piezometers recorded a slight drop in the water level during the month of December. However, since the readings did not decrease uniformly across all piezometers, the maximum water levels were used in the analysis to provide a conservative assessment of seepage and stability.

Other known water levels that are used in the seepage and stability analysis include those at: (i) the sluice channel water surface (765 feet), (ii) the red water pond (748.7 feet); and (iii) the anoxic drain prior to discharge. The water level in the anoxic drain is assumed to be approximately halfway between the ground surface and the bottom of the drain. The anoxic drain closure report indicates that the depth of the drain varies from approximately 5 feet to 9 feet from the ground surface. The closure report is included in Attachment B.

4.6 Analyzed Cross Sections – Seepage Analysis

For seepage analysis, cross sections A, B, C and D have been analyzed in recognition of the fact that relatively high gradients were calculated during the South and North End Studies (ranging from 0.30 to 0.46). Analysis of all four cross sections will allow more detailed characterization of seepage conditions along the length of the East Dike.

4.7 Analyzed Cross Sections – Slope Stability

For the purposes the analysis of slope stability, cross sections A and D have been considered since these two cross sections appear to represent potentially the most critical conditions. Cross section A includes the locations of groundwater seeps that have been observed in that area, relatively thick fill layers, and the alignment of the former Old Swan Pond Creek (See Figure 11). Cross section D includes the highest measured groundwater pressures (i.e., water levels in piezometer D1B). These slope stability computations utilize the pore pressures that are calculated from the seepage analysis rather than defining the location of the groundwater surface directly within the slope stability model.

5. SEEPAGE ANALYSIS

Based on the interpreted subsurface beneath the East Dike a seepage model for cross sections A, B, C and D were developed using the computer program SLIDE, version 5.044 (Rocscience, 2010). SLIDE can be used to perform steady-state saturated and unsaturated groundwater seepage analysis using finite element methodology. The program uses site specific geometry and material parameter inputs to calculate pore water pressures, location of the phreatic surface and discharge quantities when boundary conditions are known. The geometry and boundary conditions used for the analyses correspond with the information given in the previous section of this report. Figures 12 through 15 show the four cross sections with their respective boundary conditions.

The seepage model output includes a calculation of the groundwater exit gradient at the toe of the East Dike. Under sufficiently high gradients, piping can occur and threaten the stability of the Dike. TVA's Master Programmatic Document (URS 2009) indicates that a reasonable selection of the factor of safety against piping failure is 4.0, when defined using equation 9:

$$FS_{eg} = \frac{i_c}{i_{eg}} \qquad (9)$$

Where: i_{eg} = calculated vertical exit gradient and i_c = the critical exit gradient. The critical exit gradient is defined using equation 10,

$$i_c = \frac{\gamma - \gamma_w}{\gamma_w} \qquad (10)$$

Where: γ = bulk unit weight of the soil and γ_w = unit weight of water. If the bulk unit weight of the soil is approximately 120 pounds per cubic foot, and the unit weight of water is 62.4 pounds per cubic foot, i_c is approximately 0.92. Therefore a critical hydraulic gradient of 0.90 will be used to evaluate the factor of safety.

5.1 Seepage Analysis – Existing Conditions

The results of the finite element computations considering existing conditions for cross sections A through D are shown in Figures 16 through 19. For each cross section, two runs were executed to demonstrate the sensitivity of the results to changes in hydraulic conductivity. The first run for each cross section considered hydraulic conductivities as determined by correlation to soil behavior index, I_c as outlined in the previous section. The second run for each cross section substituted hydraulic conductivity values as determined by pore pressure dissipation (PPD) for layers where valid tests were conducted. The remaining layers without a valid PPD test were modeled with the same hydraulic conductivity as the first run, based on I_c. Table 3 summarizes the values used for each run and cross section.

In general, the seepage results appear provide a good representation of the location of the phreatic surface as measured by the piezometers that were installed across the East Dike. Furthermore, the results for cross section A indicate that the phreatic surface intersects the ground surface near the toe of the Dike in agreement with the report of observed seeps in this area as documented in Geosyntec 2010a. In addition, the observed seeps have low flow velocities and rates. Channelized seepage flow has not been observed.

However, the results of the model at cross section D did not indicate a pressure head elevated beyond hydrostatic pressure as indicated in piezometer PZ-D1B. This peizometer is located at a depth of approximately 40 feet and suggests that the Alluvial Silty Clay layer identified only at cross section D may act as a localized low permeability confining layer for the more freely draining Alluvial Sandy Silt layer beneath it as indicated in Figure 19. However, because of the depth of the elevated pressures, impacts to calculated exit gradients and factor of safety for slope stability are minimal.

The parameter that appeared to cause the largest difference in calculated vertical gradient is the horizontal hydraulic conductivity. Comparison between runs one and two for each cross section (e.g., see Figure 16a and 16b) show a relatively large increase in maximum exit gradients when hydraulic conductivity values determined from PPD tests are used in the analysis in lieu of the values calculated using correlations to the soil index behavior. This is consistent with the inverse relationship between hydraulic gradient and conductivity.

Because the calculated factor of safety is relatively sensitive to hydraulic conductivity, calculations were conducted using the two different assumed hydraulic conductivity profiles. Results are presented in Table 4. Additionally, remedial measures are also considered using the two potential hydraulic conductivity profiles.

It is noted that the hydraulic conductivity based on the soil index, I_c, is determined by correlation to the soil behavior characterization, while the hydraulic conductivity values based on PPD tests are a direct measurement of the rate of water dissipation at a specific location. Furthermore, Geosyntec notes that the values of hydraulic conductivity developed from the PPD tests are in fairly good agreement with hydraulic conductivity values given in previous Geosyntec (2010a, b) reports based on laboratory tests. In general, these direct measurements are inherently "more accurate" than correlations, but as can be seen in the graphical summary, they do not capture the anticipated variability in the hydraulic conductivity based on even relatively subtle distinctions in soil behavior characterization. In addition, there is a limitation with using the PPD tests as a sole basis for interpretation simply given the number of discrete measurements. The data set for hydraulic conductivity values based on I_c is more extensive and is therefore useful for purposes of comparison of results with respect to stratigraphy.

In summary, the results of the seepage analysis are sensitive to the selection of the value for hydraulic conductivity. Arguments can be made in favor of using either of the methods presented for the determination of hydraulic conductivity; however calculation results indicate that a more conservative interpretation is given using results based on PPD tests for the Lower Dike Fill and Alluvial Clayey Silt.

5.2 Seepage Analysis without the Anoxic Drain and Red Water Pond

Based on discussion with TVA, Geosyntec understands that it is likely that the anoxic drain and red water pond located near the crest of the East Dike will be de-commissioned. Since the drain and pond were modeled as known boundary conditions, they were removed as a known quantity to assess the effects of their removal. Figures 20 through 23 show the seepage analysis results and calculated escape gradients for cross sections A through D under this condition. In general, the removal of the drain and red water pond from the model causes a slight decrease in calculated escape gradients. These results are summarized in Table 5.

5.3 Seepage Analysis with an Assumed Depressed Water Table

The final set of seepage analyses performed considers the de-commissioning of the anoxic drain and a depressed water table that may be accomplished through the use of a toe drain and/or pumping system. Figures 24 through 27 show the seepage analysis results and calculated escape gradients for cross sections A through D under this scenario. This remedial measure had the greatest effect on the decrease of the calculated maximum escape gradient. These results are also summarized in Table 5.

6. SLOPE STABILITY ANALYSIS

Slope stability analyses were performed using SLIDE, version 5.044 (Rocscience, 2010). Both Bishop's method and Spencer's method were used to calculate the lowest FS for a circular slip surface passing through the East Dike at cross sections A and D. The lowest calculated FS is displayed in the graphical output for each respective analysis.

The minimum required FS for slope stability is given in TVA's report "Facility Design and Construction Requirements, Volume 2, Rev. 1.0". The document indicates that a calculated FS of 1.5 is acceptable for long-term conditions.

Figures 28 and 29 show that the results of the slope stability analysis demonstrate that the minimum calculated FS for slope stability for cross section A is 1.94, and for cross section D, 2.28. These results were obtained using the strength parameters indicated in Table 1. Furthermore, the slope stability analyses were performed using pore pressures calculated during the seepage finite element analysis (FEA). Table 6 summarizes the results of the slope stability analysis.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 Slope Stability Analysis

As referenced in the Presentation and in the Background section of this report, the previously advanced in-situ SPT N-value results seemed to be anomalously low and inconsistent with the laboratory test results. The CPTu investigation that was executed for this phase of the assessment provided shear strength results that support the laboratory test results and, therefore, alleviate concerns regarding the potentially soft soil conditions within the Lower Dike Fill and Alluvial Silty Clay layers. Using published correlations between CPTu data and shear strength, revised parameters were obtained and used in the analyses. The results of the slope stability analyses indicate that the East Dike is stable with regard to a deep-seated failure mechanism and meets the minimum required FS of 1.5 at cross sections A and D.

7.2 Seepage Analysis

Based on revisions to the stratigraphy and hydraulic conductivity as revealed by the CPTu investigation, revised seepage analyses were performed and maximum vertical hydraulic gradients were calculated for cross sections A through D for: (i) existing conditions, (ii) removing of the anoxic drain and Red Water Pond, and (iii) using an assumed depressed water table. For each of these three conditions, two sets of analyses were performed to assess the sensitivity of the results to changes in the hydraulic conductivity of the Lower Dike Fill and Alluvial Clayey Silt layers. The seepage analysis results were compared to the requirement of FS_{min} = 4.0 as outlined in TVA's Master Programmatic Document; however it is noted the U.S. Army Corp of Engineers recommend a range of FS values from 1.5 to 15 (USACE 1986).

Under existing conditions and using hydraulic conductivities based solely on the soil index, I_c, cross section A has a FS of less than 4.0 while cross sections B, C and D have a FS greater than 4.0. When considering hydraulic conductivity based on PPD tests for the Lower Dike Fill and Alluvial Clayey Silt layers, the FS values range from 1.25 to 3.21.

When the analysis considered the removal of the anoxic drain and red water pond, FS values ranged from 3.31 at cross section A to 7.20 at cross section D using hydraulic conductivity values based on soil index. When the conductivity for the Lower Dike Fill and Alluvial Clayey Silt were based on PPD tests, the FS values ranged from 1.26 at cross section B to 7.2 at cross section D.

The final scenario considered an assumed depressed water table representing the presence of some type of dewatering system. Calculated FS values ranged from 6.97 at cross section B to 20.45 at cross section D using hydraulic conductivity values based on soil index. When the conductivity for the Lower Dike Fill and Alluvial Clayey Silt were based on PPD tests, the FS values ranged from 1.45 at cross section B to 12.30 at cross section D.

The seepage analysis results discussed above are summarized in Tables 4 and 5. It is noted that with the exception of cross section B under the more conservative hydraulic conductivity profile, the analyzed scenarios exceeded the FS_{min} = 1.5 given in USACE (1986). In addition, the soil typess found in the Lower Dike Fill at the toe of the East Dike are not generally susceptible to piping. Therefore, Geosyntec believes that the potential for a seepage failure due to piping is minimal under the current conditions, even though the exit gradient in many cases exceeds the value necessary to achieve the target FS values for exit gradient acknowledged in TVA's Master Programmatic Document. Given the low FS values calculated for exit gradient at cross section B using the more conservative hydraulic conductivity profile and the sensitivity of the results to this change, Geosyntec recommends that the anoxic drain be modified (and potentially de-commissioned) to reduce the potential for seepage at the face of the East Dike. The modification should recognize that a relatively shallow water level exists in the subsurface in the vicinity of the East Dike and that it would be beneficial for this water level to be lowered. This can be accomplished by installation of a deep French drain, a local sump, or a conveyance system directly from the anoxic drain to the Intake Channel. Independent tests performed recently by TVA indicate that the "environmental benefits" of the anoxic drain are minimal, as the quality of the water at the influent and effluent ends of the drain are similar.

Because the calculated high exit gradients have not shown any indication for initiating piping and have not had an adverse impact on global slope stability, Geosyntec further recommends that the East Dike be monitored for water levels, seepage, and movement, particularly in the area of cross section B. The monitoring of the East Dike should be in accordance with the Site Monitoring Plan for East Dike (SMP-ED) provided as Attachment C to this Report. The SMP-ED provides: (i) a summary of the results and conclusions discussed in this Report; (ii) monitoring stations; (iii) monitoring frequency; (iv) discussion and results of sensitivity analyses; (v) identification of trigger levels; and (vi) appropriate responses in the event that a trigger level is exceeded.

This recommendation importantly considers the fact that ongoing and future operations in this area are anticipated to result in a decrease in surface water infiltration, which will have the benefit of reducing water levels in the shallow subsurface resulting in improvements to the estimated factors of safety.

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TABLES

Table 1. Summary of Material Strength Properties

Material Layer	Moist Unit Weight (pcf)	Drained Shear Strength φ' (deg.)	Undrained Shear Strength (psf)	Used for Analysis[1]
Fly Ash[2]	75	25	$s_u/\sigma_v' = 0.8$, $s_{umin} = 200$ psf	$s_u/\sigma_v' = 0.8$, $s_{umin} = 200$ psf
Dense Bottom Ash[2]	100	30	$s_u/\sigma_v' = 0.8$, $s_{umin} = 200$ psf	$s_u/\sigma_v' = 0.8$, $s_{umin} = 200$ psf
Rock Embankment[2]	135	35	--	φ' = 35 deg.
Upper Dike Fill	125	33.3 - 41.2[3]	--	φ' = 32 deg.
Lower Dike Fill	120	--	$s_u/\sigma_v' = 0.55$, $s_{umin} = 300$ psf[3]	$s_u/\sigma_v' = 0.55$, $s_{umin} = 300$ psf
Alluvial Clayey Silt	125	--	1875 – 2784[3,4]	$s_u = 1800$ psf
Alluvial Silty Clay	125	--	602[3,4,5]	$s_u = 600$ psf
Alluvial Sandy Silt	125	35.2 - 35.9[3]	--	φ' = 30 deg.
Sand & Gravel	120	36.7 - 42.5[3]	--	φ' = 35 deg.

(1) The undrained condition is the most critical, however for free draining layers the drained shear strength is used in the analysis.
(2) Data provided in previous reports, [Geosyntec 2010a, Geosyntec 2010b].
(3) Data provided from CPT soundings. See Figures 4 and 5 for plots of s_u/σ_v' for the Lower Dike Fill Layer.
(4) Undrained shear strength determined using a cone factor, $N_k = 15$.
(5) The alluvial silty clay layer seemed to be located only near cross section D.

Table 2. Summary of hydraulic conductivity data obtained by CPTu

Material Layer		Hydraulic Conductivity, k_h (ft/s)			
		Cross Section A	**Cross Section B**	**Cross Section C**	**Cross Section D**
Upper Dike Fill	Based on I_c	1.60 e-5	7.05 e-5	3.99 e-4	3.99 e-4
	Based on PPD [(2)]	No Valid Tests	No Valid Tests	No Valid Tests	No Valid Tests
Lower Dike Fill	Based on I_c	1.93 e-7	1.19 e-6	2.15 e-6	1.56 e-6
	Based on PPD	No Valid Tests	No Valid Tests	1.25 e-7	1.08 e-7
Alluvial Clayey Silt	Based on I_c	7.80 e-8	1.03 e-8	2.4 e-7	2.66 e-7
	Based on PPD	9.50 e-9	9.34 e-8	1.75 e-7	1.01 e-7
Alluvial Silty Clay	Based on I_c	Not Present	Not Present	Not Present	1.05 e-8
	Based on PPD				No Valid Tests
Alluvial Sandy Silt	Based on I_c	Present in section, not at CPT location	1.04 e-5	7.19 e-6	1.06 e-5
	Based on PPD		No Valid Tests	6.18 e-8	No Valid Tests
Sand & Gravel	Based on I_c	3.37 e-5	7.69 e-5	2.42 e-4	4.58 e-6
	Based on PPD	No Valid Tests	No Valid Tests	No Valid Tests	No Valid Tests

1. Values shown are averages taken in the layer at each cross section A, B, C and D. Table 3 shows a value that is averaged from the data given in Table 2.
2. Pore Pressure Dissipation Tests (PPD).

Table 3. Summary of Material Properties Used in Analysis

Material Layer	Hydraulic Conductivity, k_h (ft/s)	
	$[k(I_c)]$ (2)	$[k(I_c\&PPD)]$(3)
Upper Dike Fill	4.33×10^{-5}	
Lower Dike Fill	1.27×10^{-6}	1.17×10^{-7}
Alluvial Clayey Silt	1.49×10^{-7}	9.47×10^{-8}
Alluvial Silty Clay(4)	1.05×10^{-8}	
Alluvial Sandy Silt	9.43×10^{-6}	
Sand & Gravel	8.93×10^{-5}	

1. Values of hydraulic conductivity for Fly Ash and Dense Bottom of 9.84×10^{-6} ft/s will be used for analysis based on data from Geosyntec (2010a, 2010b)
2. Based on correlation with soil index, I_c (parameter obtained with CPT Testing). Correlation from Robertson (2009). Values shown are averages taken throughout the layer.
3. Based on pore pressure dissipation tests during CPT testing. The calculation for k_h is: $k_h = (c_h * g_w)/M$, where c_h = coefficient of consolidation in the horizontal direction, g_w = unit weight of water, and M = the one-dimensional constraint modulus. Relationships for c_h, and M are given in Robertson (1992) and Robertson *et al.*(2009), respectively. Values shown are averages taken throughout the layer.
4. The alluvial silty clay layer seemed to be located only near cross section D

Table 4. Summary of Seepage Analyses Results – Existing Conditions

Cross Section	Maximum Hydraulic Gradient		$FS_{eg} = i_c/i_{max}$, using $i_c = 0.9$	
	i_{max} $[k(I_c)]$(2)	i_{max} $[k(I_c\&PPD)]$(3)	$[k(I_c)]$	$[k(I_c\&PPD)]$
A	0.284	0.408(4)	3.17	2.21
B	0.206	0.720(4,5)	4.36	1.25
C	0.180	0.451(4)	5.00	1.99
D	0.152	0.280(4)	5.92	3.21

1. TVA Programmatic Document specifies a minimum target FS of 4.0, however target FS values given in USACE (1986) range from 1.5 to 15.
2. Values for maximum hydraulic gradient listed in this column are based on the calculations using hydraulic conductivity (k) correlated to soil index, I_c for Upper Dike Fill, Lower Dike Fill, Alluvial Clayey Silt, Alluvial Silty Clay, Alluvial Sandy Silt and Sand & Gravel.
3. Values for maximum hydraulic gradient listed in this column are based on calculations using hydraulic conductivity (k) as determined from pore pressure dissipation (PPD) tests for the layers where valid tests exist (i.e., Lower Dike Fill and Alluvial Clayey Silt). Remaining layers were assigned hydraulic conductivities derived from the soil index, I_c.
4. The general trend of hydraulic gradients increasing is consistent with this series of analysis using the hydraulic conductivities determined from PPD tests which were lower than the hydraulic conductivity determined by correlation with I_c for the Lower Dike Fill and Alluvial Clayey Silt layers. See Tables 2 and 3 for hydraulic conductivity values.
5. The abnormally high increase in maximum hydraulic gradient seen in the analysis for cross section B can be explained by the fact that at this location the layers having low permeability (i.e., the Lower Dike Fill and Alluvial Clayey Silt) have a minimum thickness (See Figure 17b).

Table 5. Summary of Results for Possible Remedies for Seepage (Target $i_{max} = 0.225$ if $FS_{req} = 4.0$)

Remedial Option	Cross Section A		Cross Section B		Cross Section C		Cross Section D	
	i_{max} [k(I_c)]	i_{max} [k(I_c&PPD)]	i_{max} [k(I_c)]	i_{max} [k(I_c&PPD)]	i_{max} [k(I_c)]	i_{max} [k(I_c&PPD)]	i_{max} [k(I_c)]	i_{max} [k(I_c&PPD)]
Existing Conditions[(1)]	0.284 (FS = 3.17)	0.408 (FS = 2.21)	0.206 (FS = 4.36)	0.720 (FS = 1.25)[2]	0.180 (FS = 5.00)	0.451 (FS = 1.99)[2]	0.152 (FS = 5.92)	0.280 (FS = 3.21)
Removal of Anoxic Drain and Red Water Pond	0.272 (FS = 3.31)	0.378 (FS = 2.38)	0.183 (FS = 4.92)	0.713 (FS = 1.26)[2]	0.174 (FS = 5.17)	0.420 (FS = 2.14)	0.125 (FS = 7.2)	0.125 (FS = 7.2)
Water Table Drawdown	0.069 (FS = 13.04)	0.280 (FS = 3.21)	0.129 (FS = 6.97)	0.619 (FS = 1.45)[2]	0.083 (FS = 10.84)	0.416 (FS = 2.16)[2]	0.044 (FS = 20.45)	0.073 (FS = 12.33)

1. Values shown for existing conditions are also shown in Table 4.
2. See note 5 on Table 4 for discussion of this value.

Table 6. Summary of Slope Stability Analyses – Existing Conditions

Cross Section	Calculated Minimum Factor of Safety	FS_{min}	Result OK?	Results shown in Figure
A	1.94	1.5	Yes	28
D	2.28	1.5	Yes	29

FIGURES

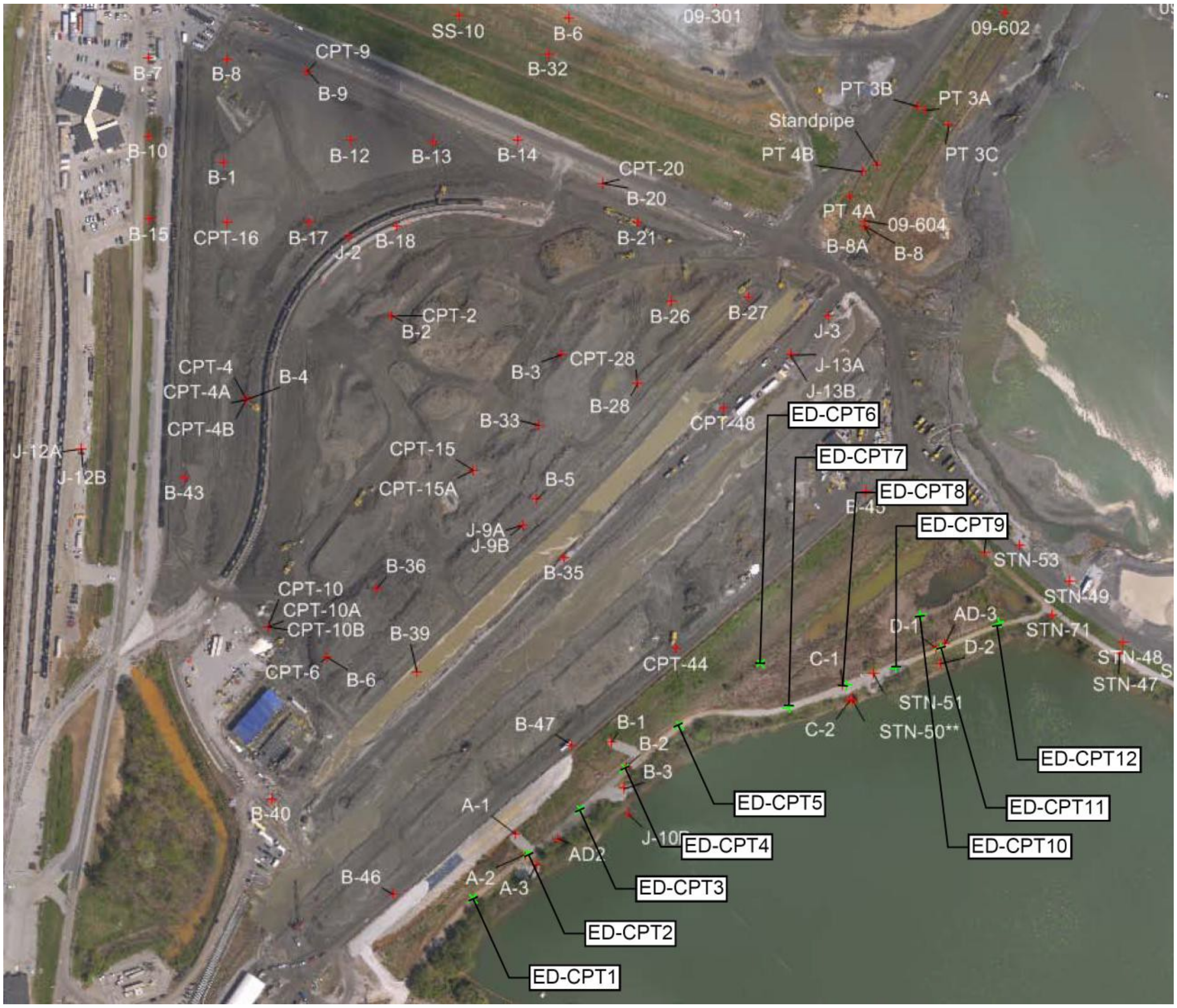


Figure 1. Proposed CPTu Locations. See Figure 2 for final locations.

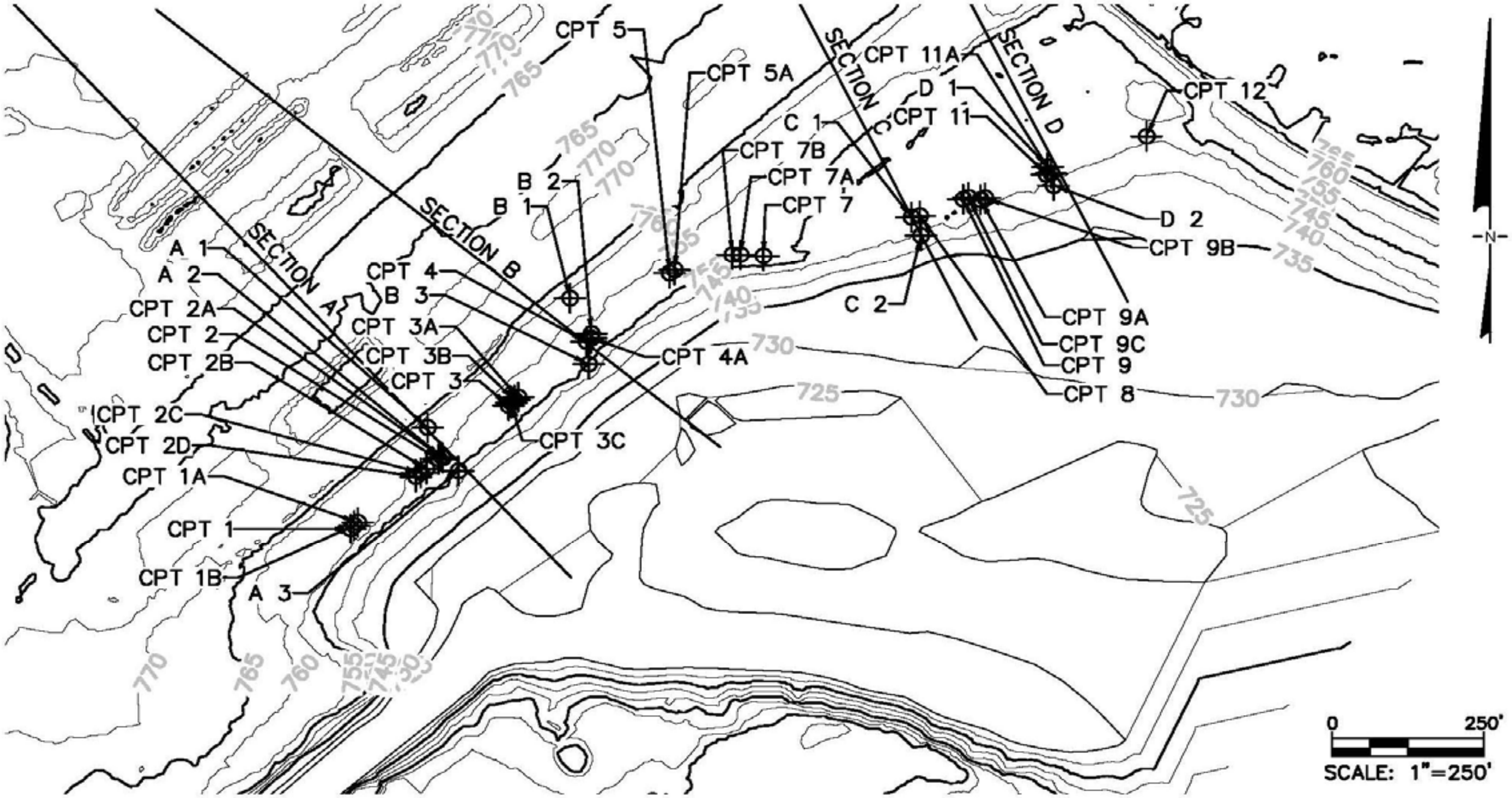


Figure 2. Final locations of CPTu sounding combined with recent topography of the East Dike and bathymetry of the intake channel.

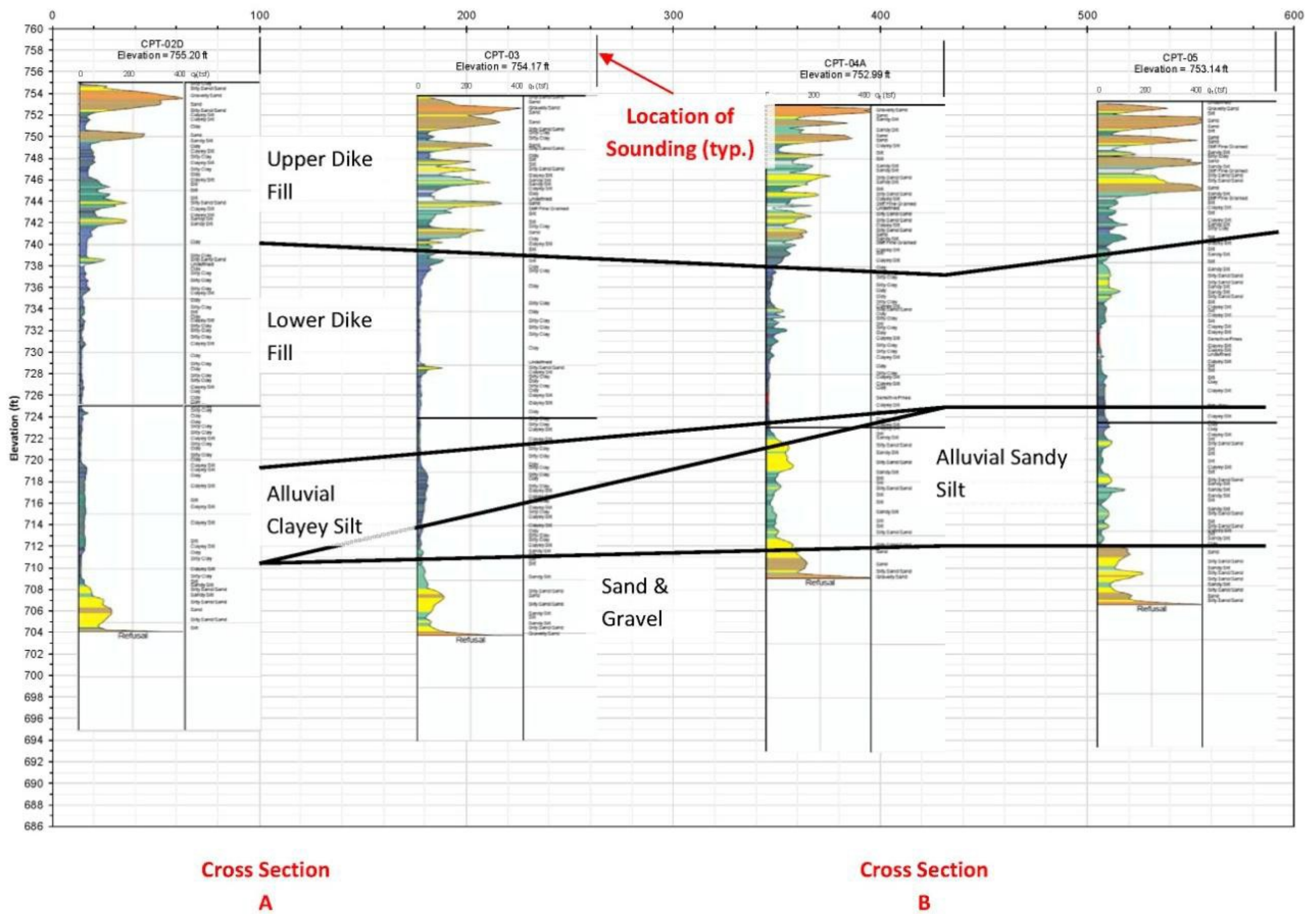


Figure 3a. Stratigraphy interpreted by CPTu soundings

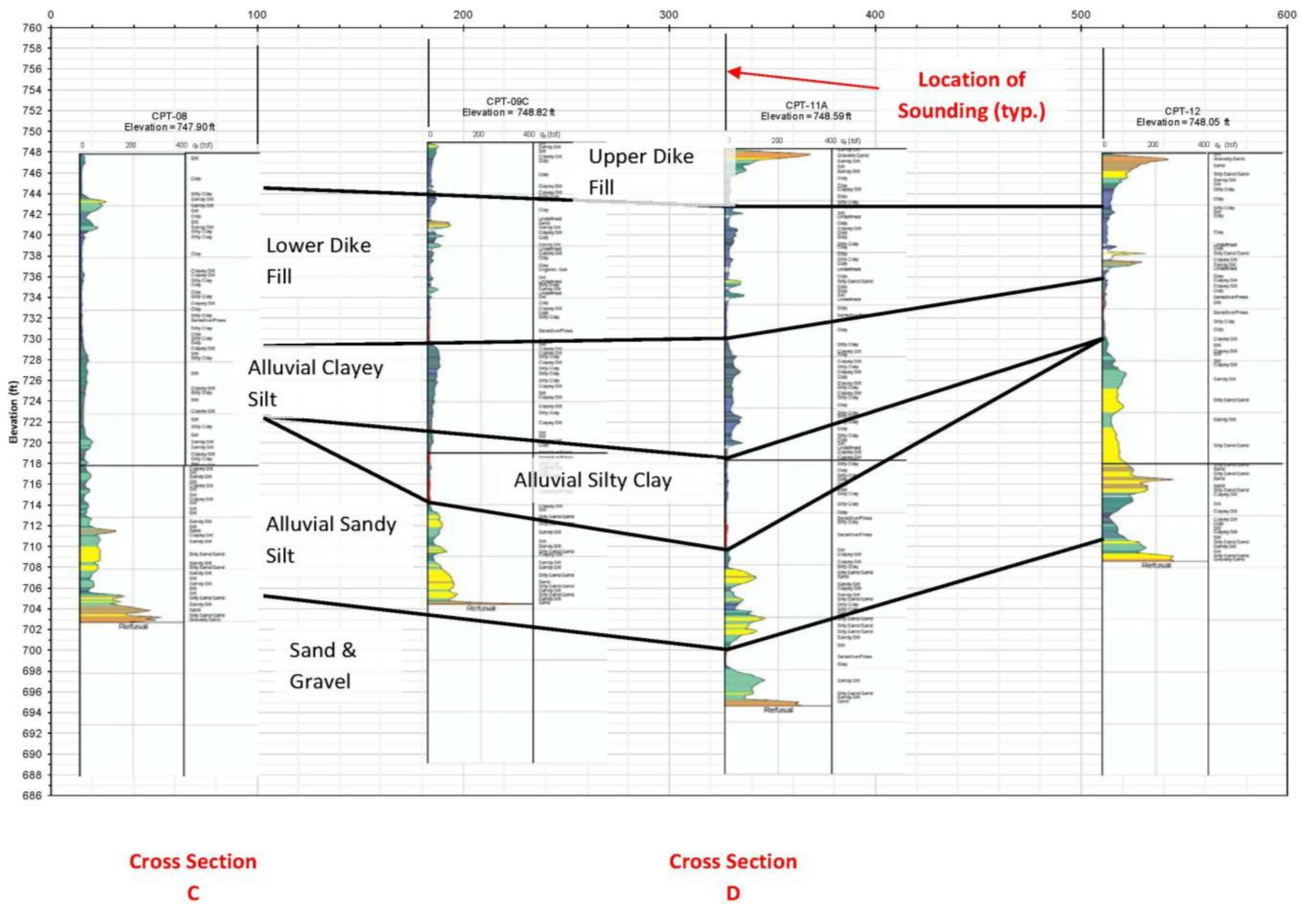


Figure 3b. Stratigraphy interpreted by CPTu soundings

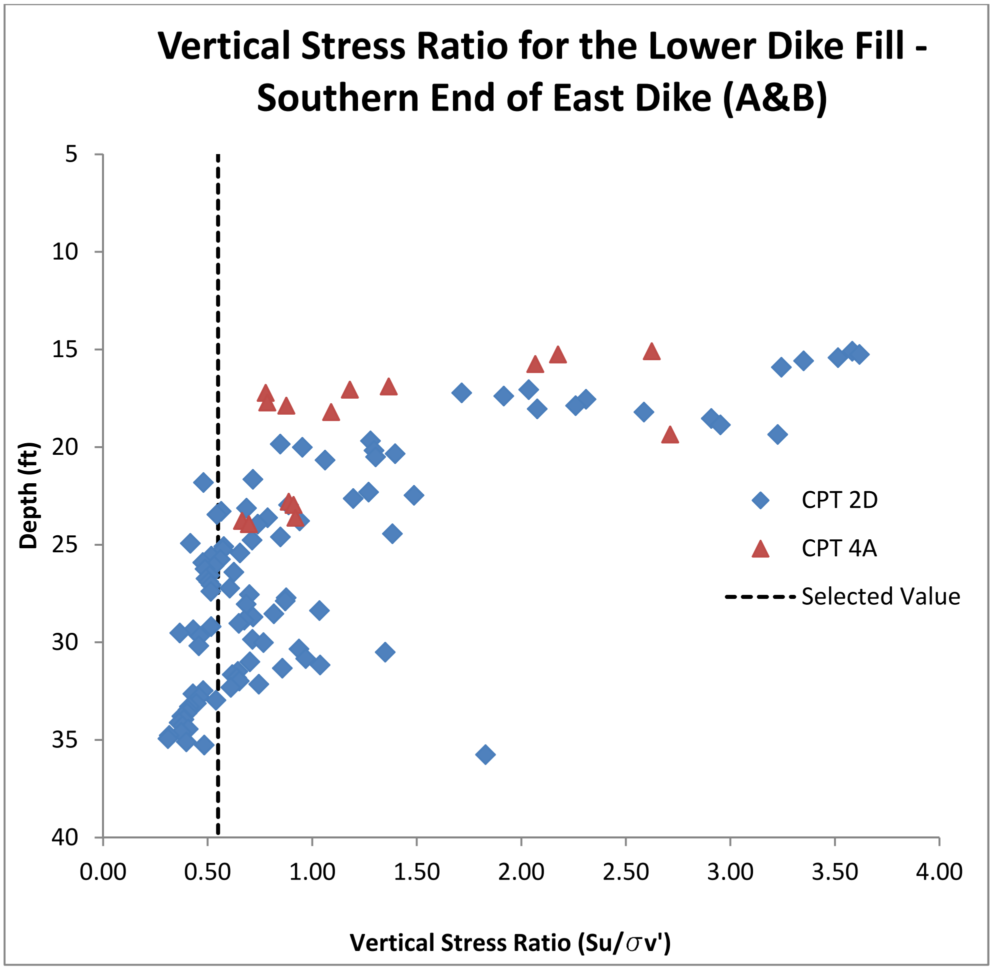


Figure 4. Vertical Stress Ratio in Lower Dike Fill for cross sections A&B.

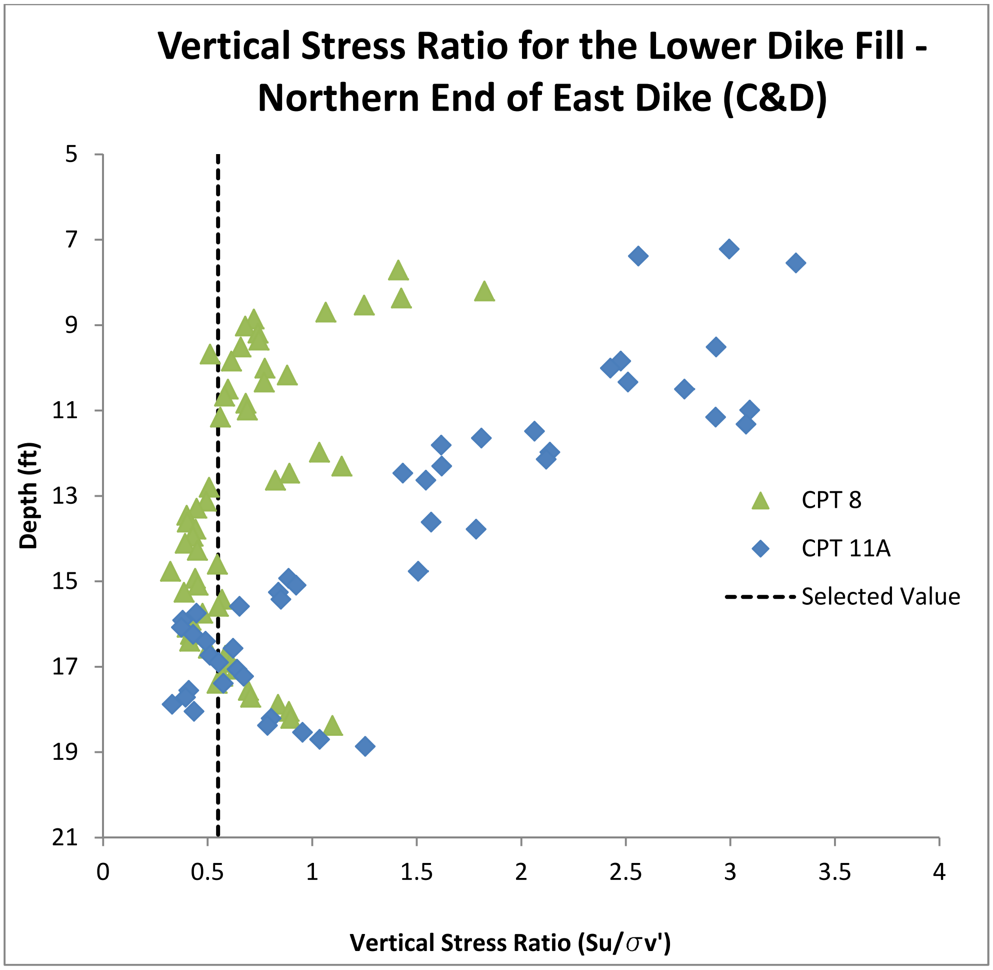


Figure 5. Vertical Stress Ratio in Lower Dike Fill for cross sections C&D.

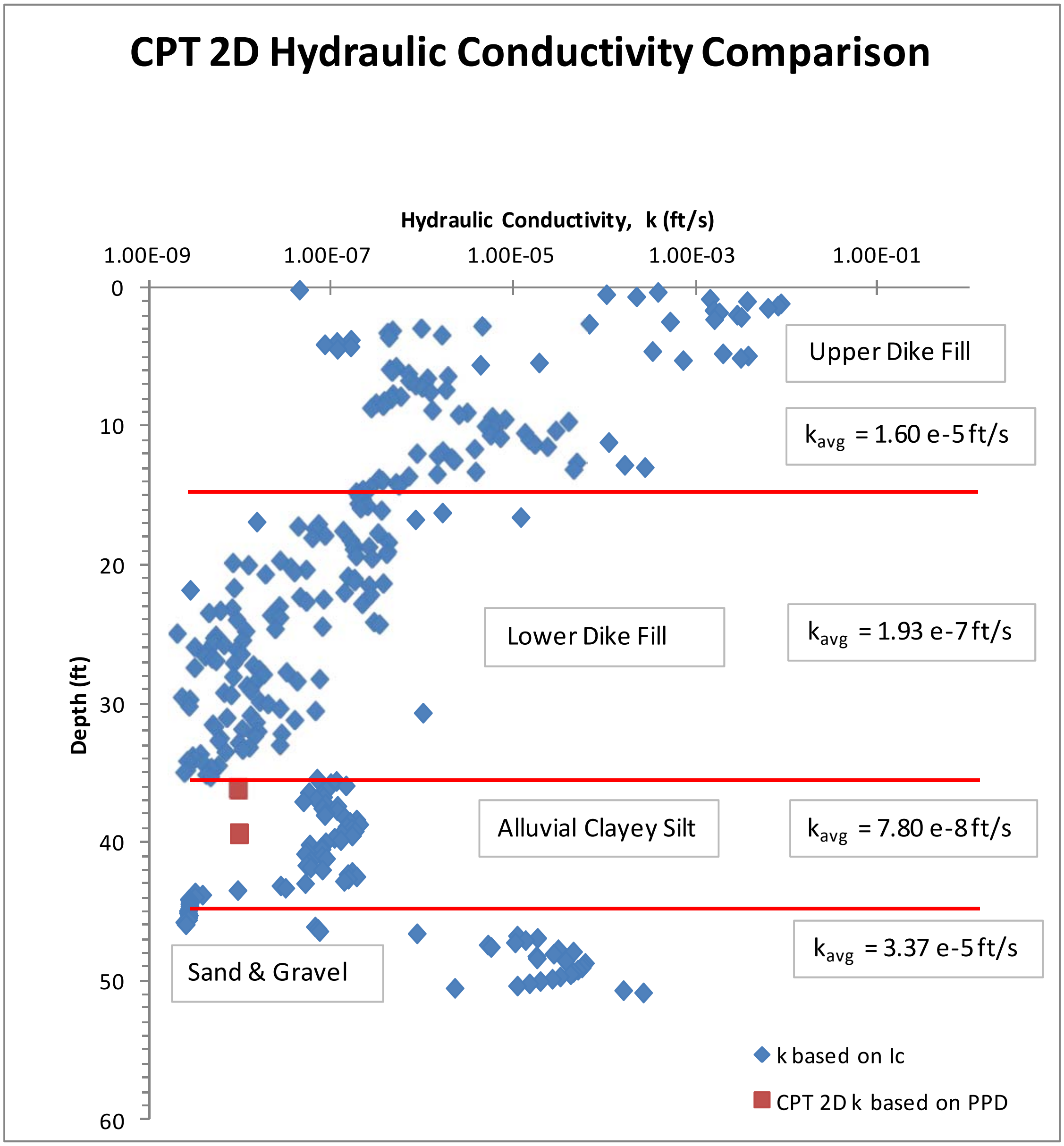


Figure 6. Assessment of hydraulic conductivity for CPTu located at cross section A. Average values shown are for the k values based on I_c.

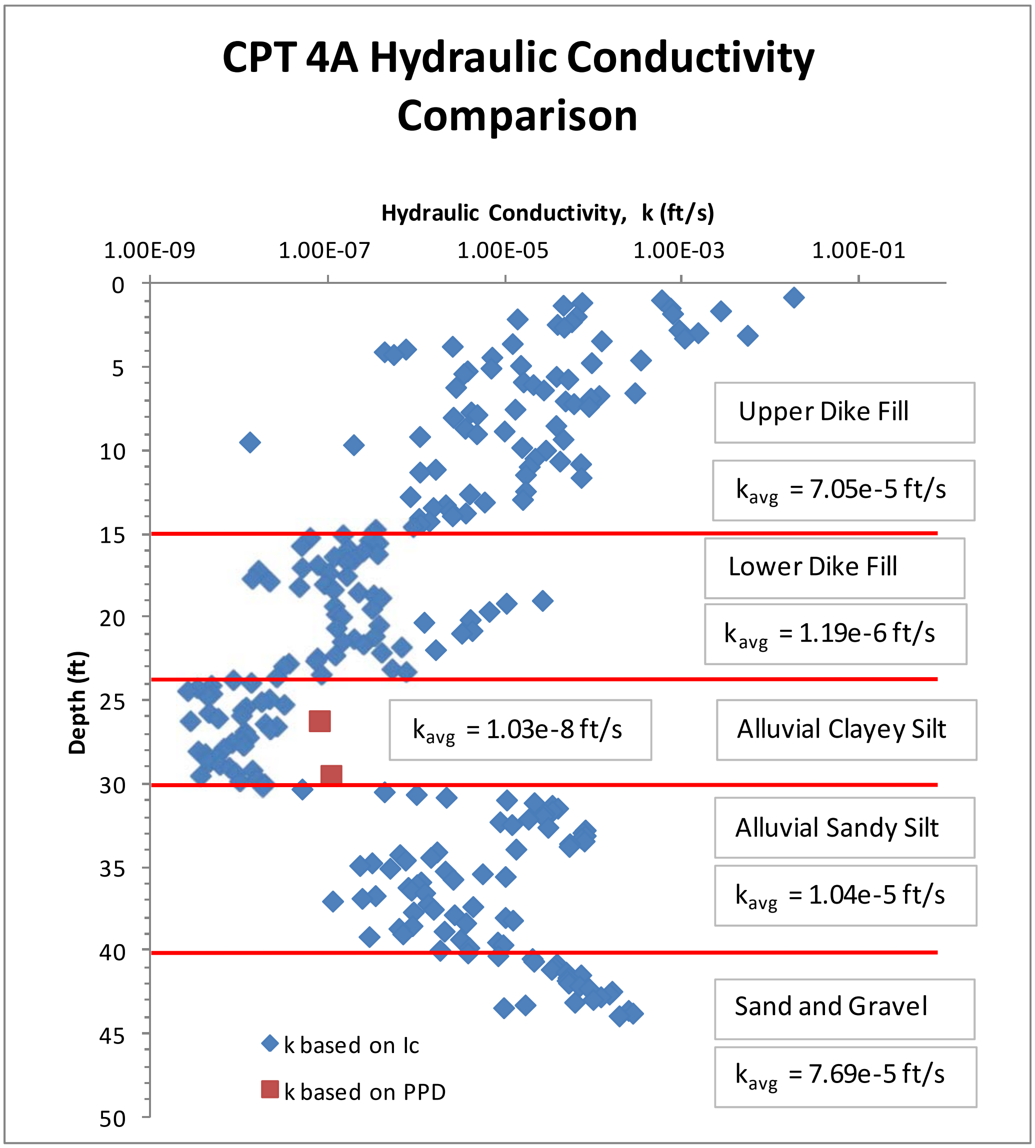


Figure 7. Assessment of hydraulic conductivity for CPTu located at cross section B. Average values shown are for the k values based on I_c.

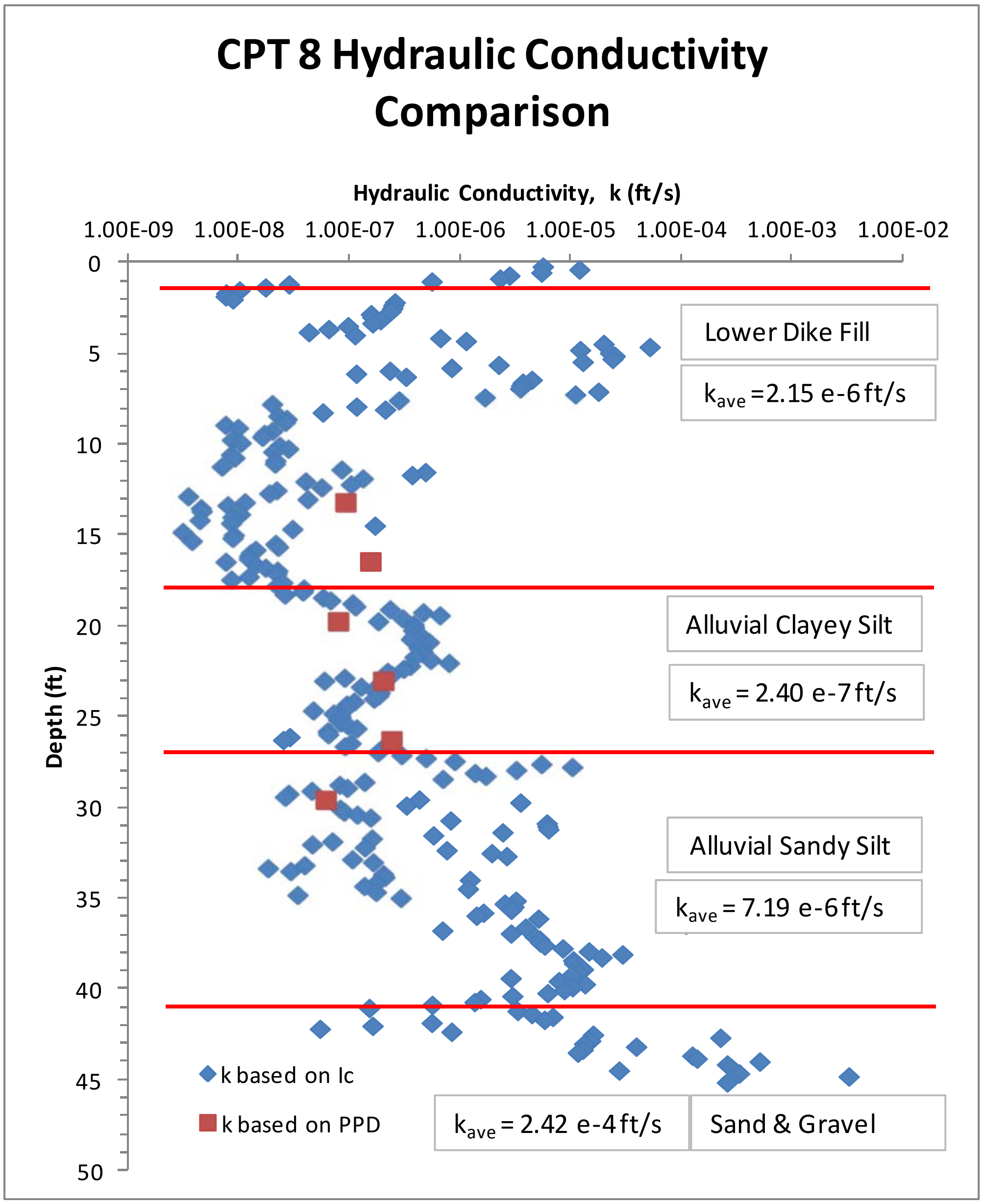


Figure 8. Assessment of hydraulic conductivity for CPTu located at cross section C. Average values shown are for the k values based on I_c.

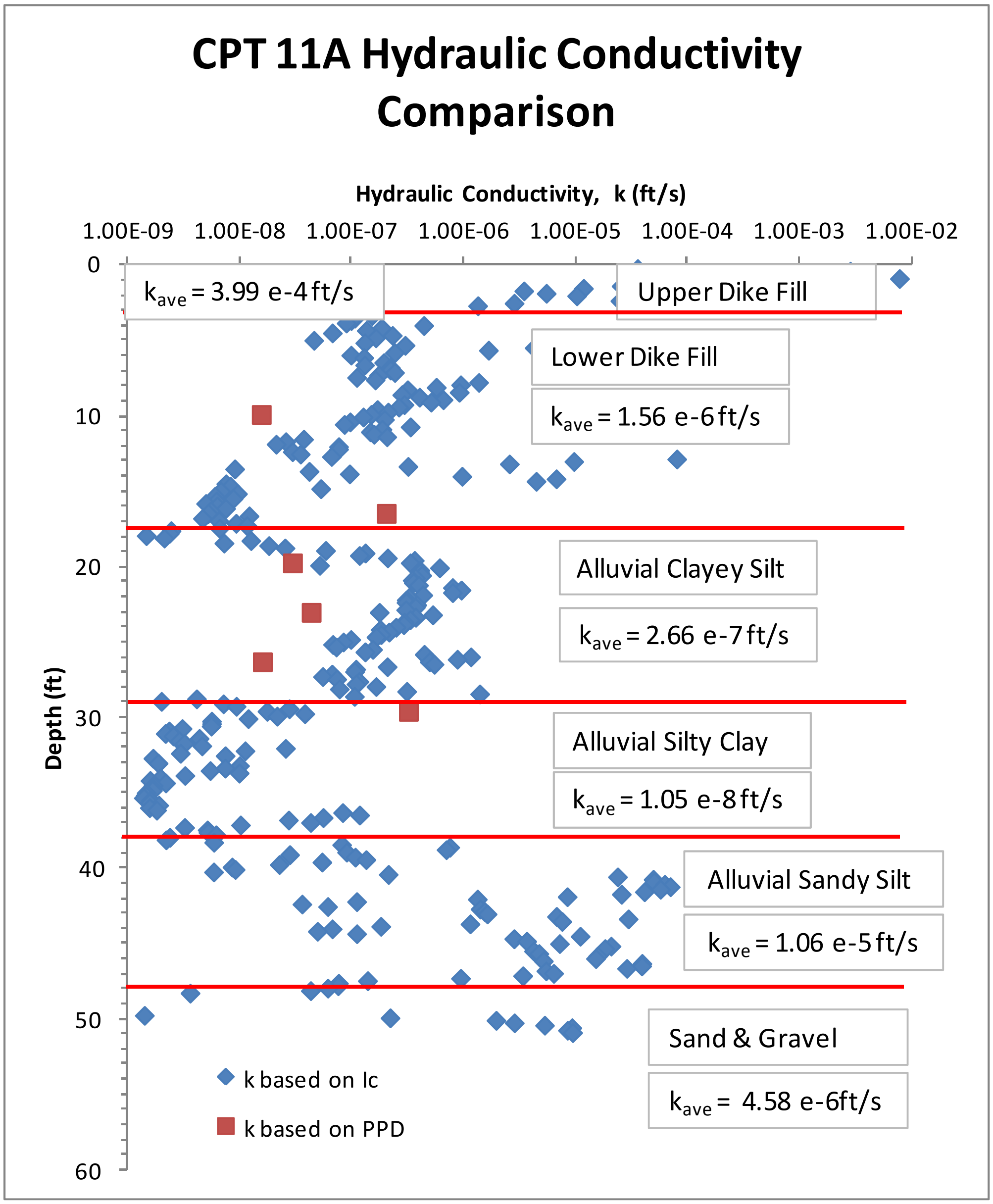


Figure 9. Assessment of hydraulic conductivity for the CPTu located at cross section D. Average values shown are for the k values based on I_c.

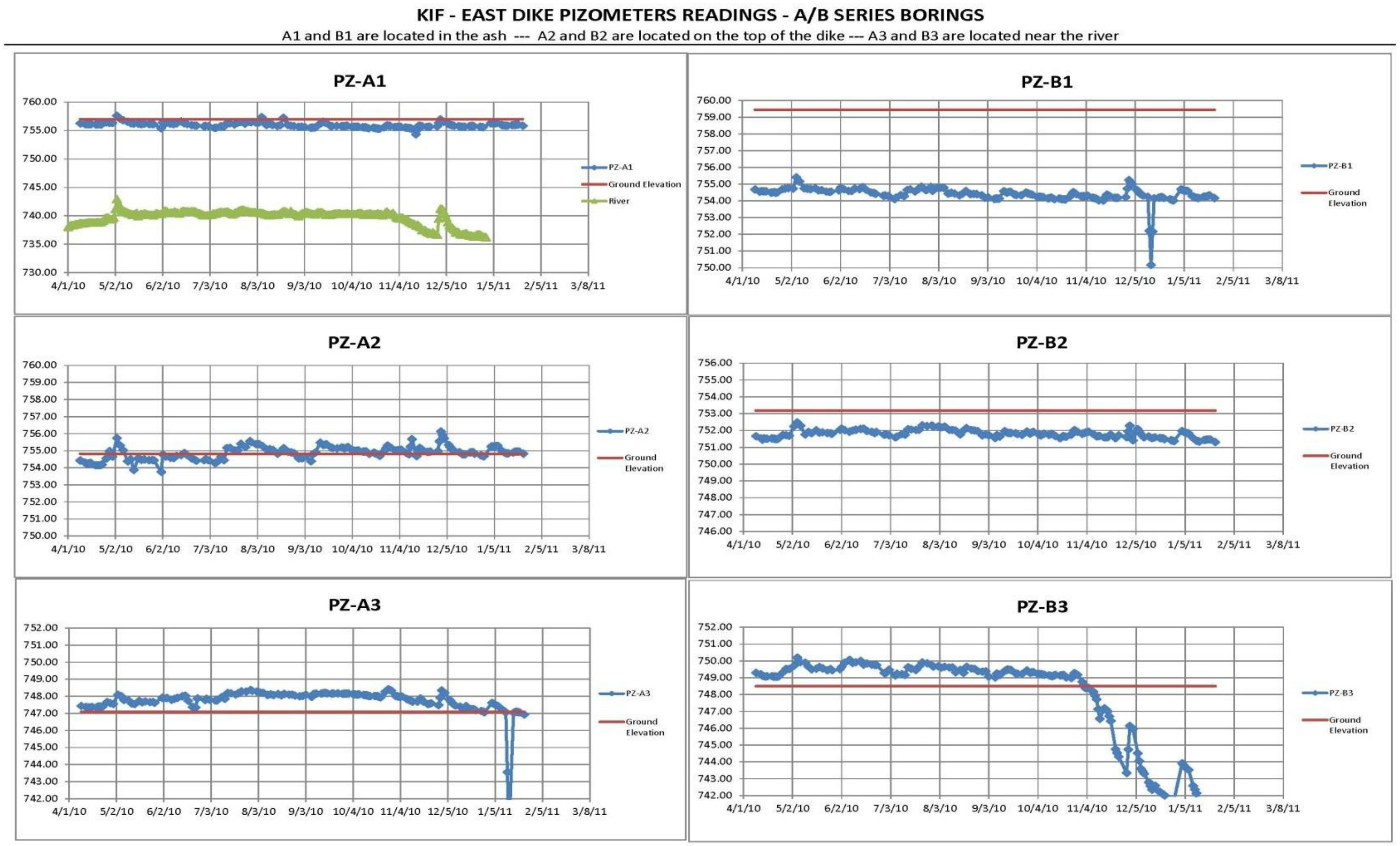


Figure 10a. Recorded piezometric data at cross sections A and B (refer to Figure 2 for boring locations)

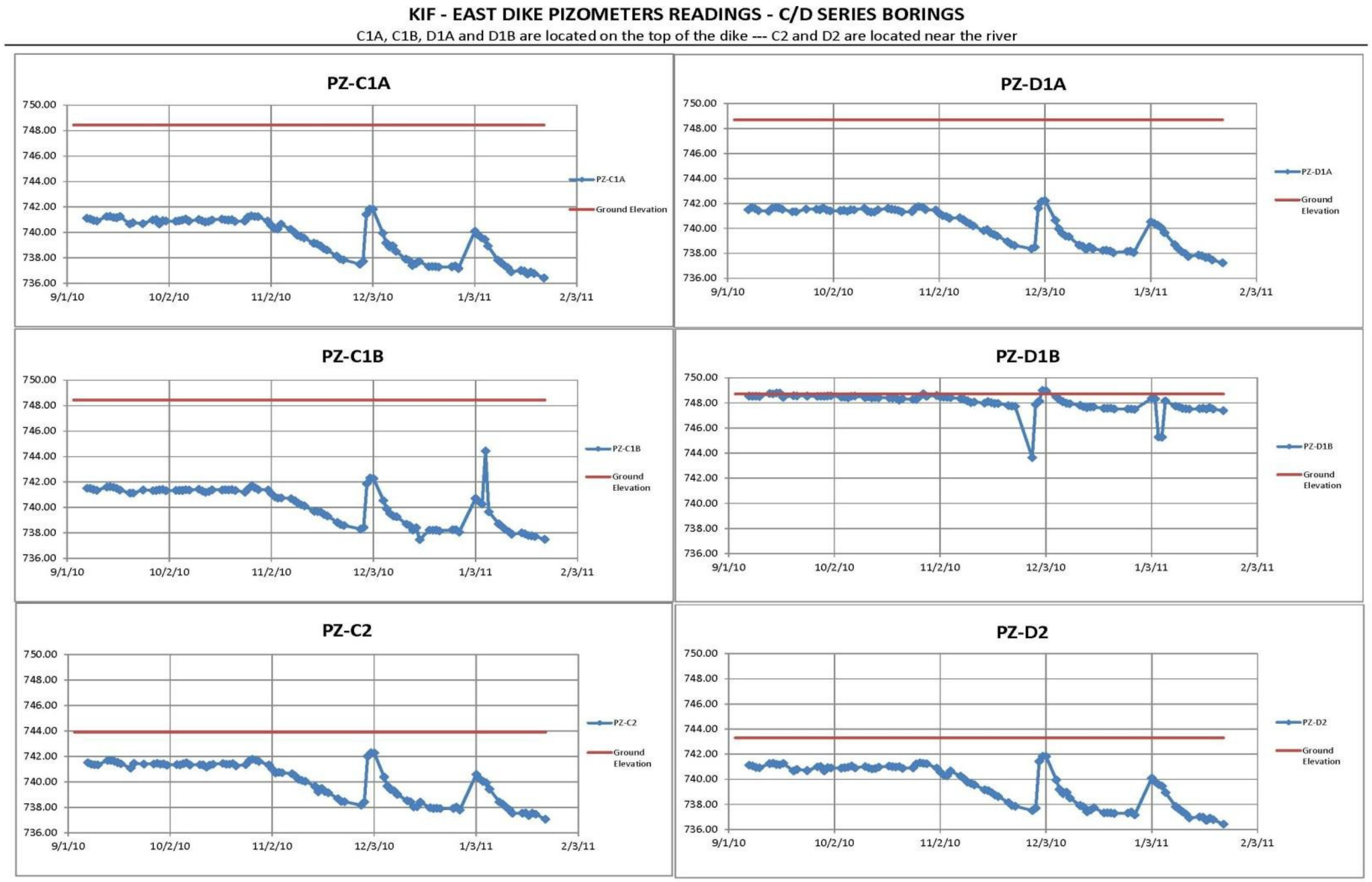


Figure 10b. Recorded piezometric data at cross sections C and D (refer to Figure 2 for boring locations)

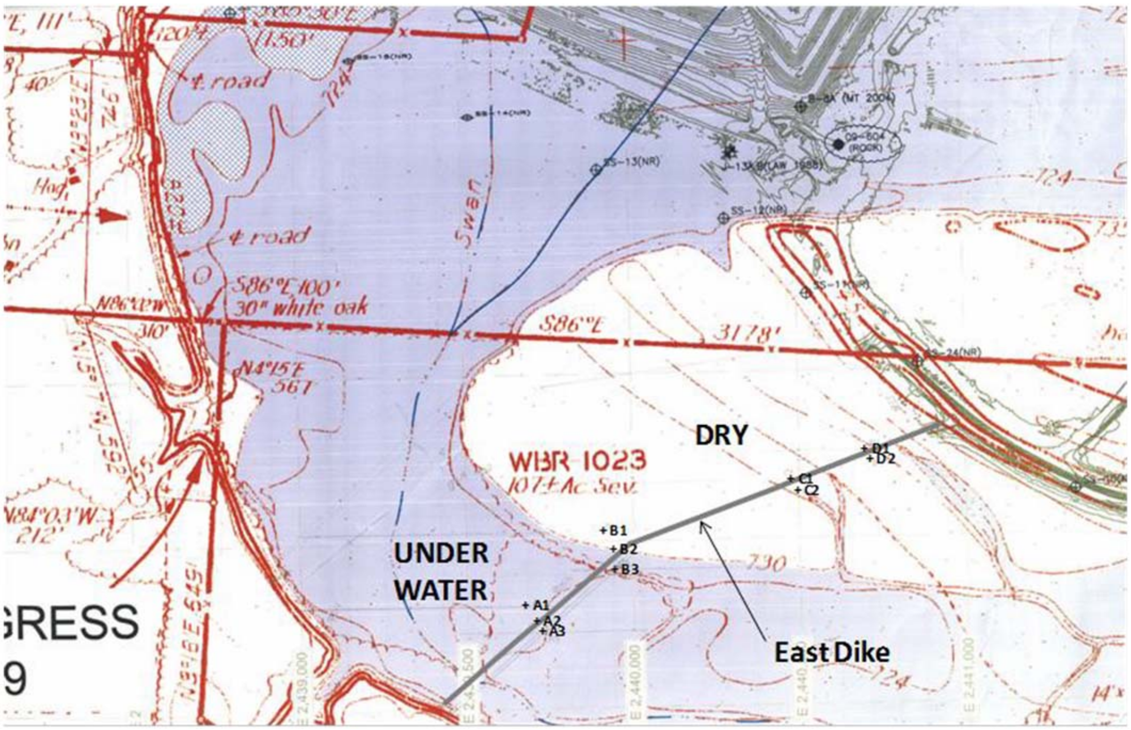


Figure 11. Location of East Dike in relation to the old Swan Pond Creek

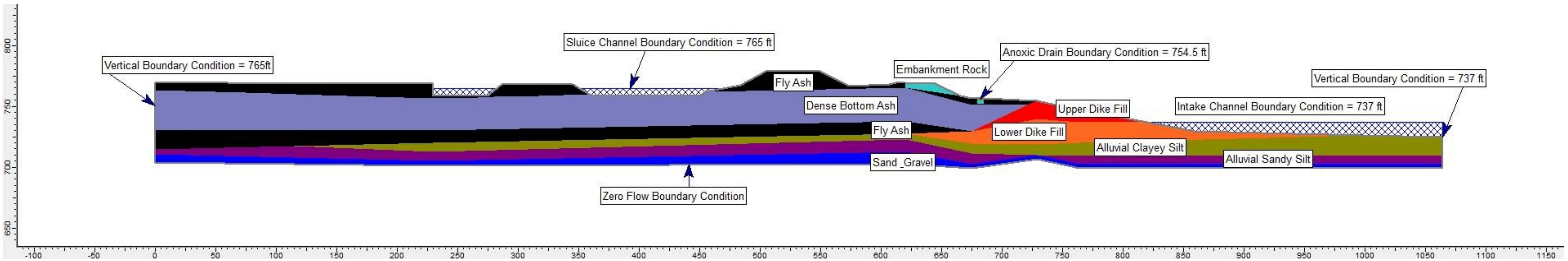


Figure 12. Cross section A boundary conditions for seepage analysis (scale in feet)

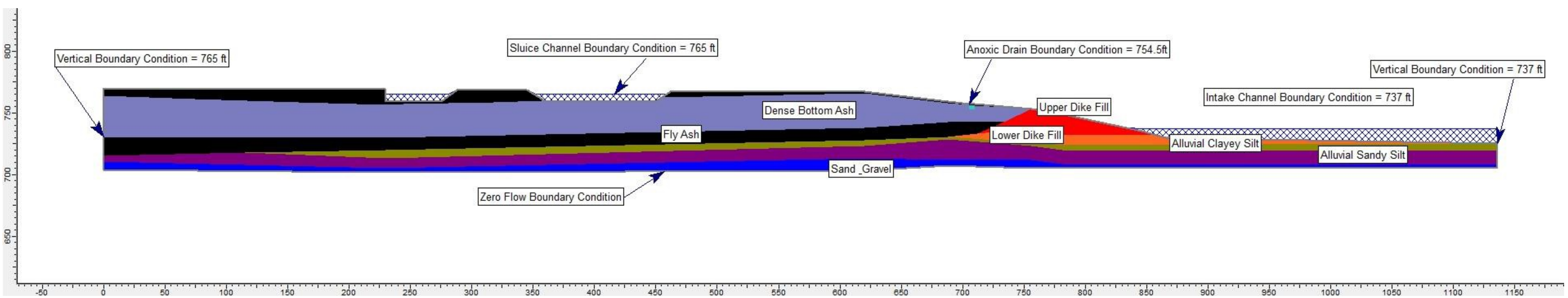


Figure 13. Cross section B boundary conditions for seepage analysis (scale in feet)

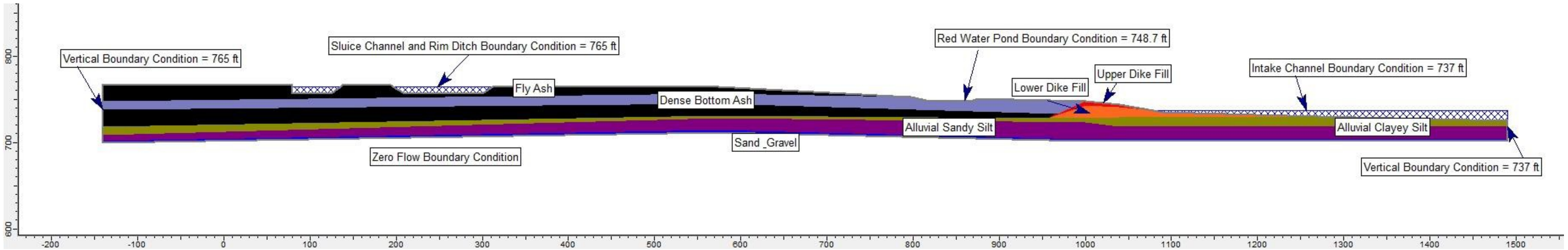


Figure 14. Cross section C boundary conditions for seepage analysis (scale in feet)

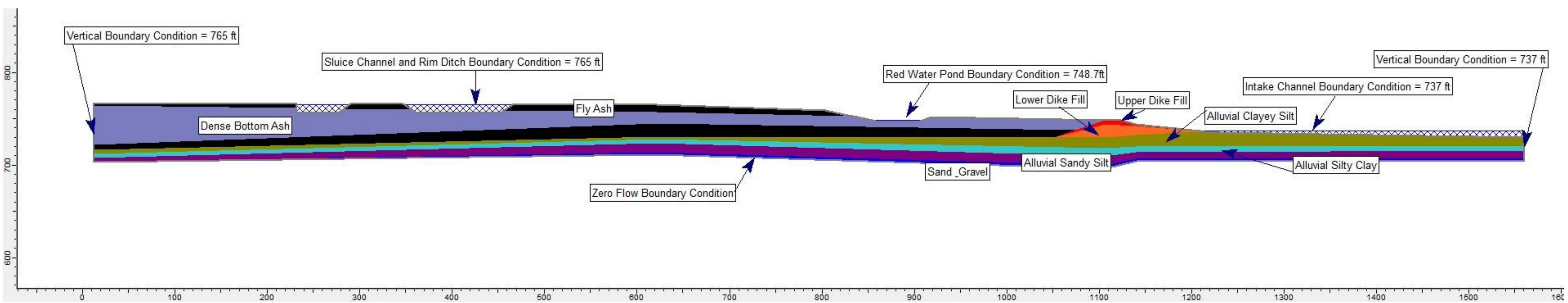


Figure 15. Cross section D boundary conditions for seepage analysis (scale in feet)

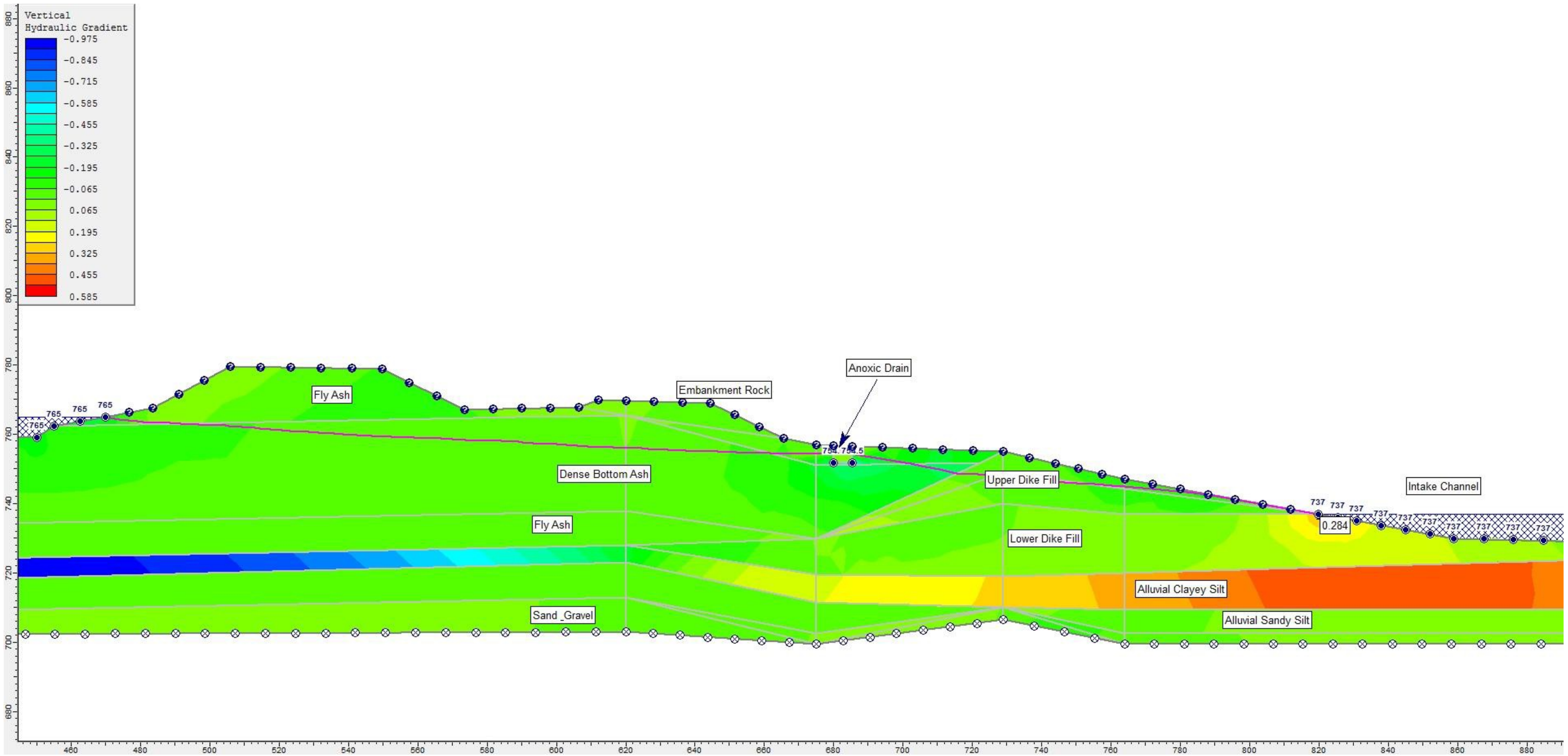


Figure 16a. Results of seepage analysis for cross section A – existing conditions using hydraulic conductivity (k_h) based on soil index, I_c. Maximum hydraulic gradient shown = 0.284. Vertical and horizontal scale is in units of feet.

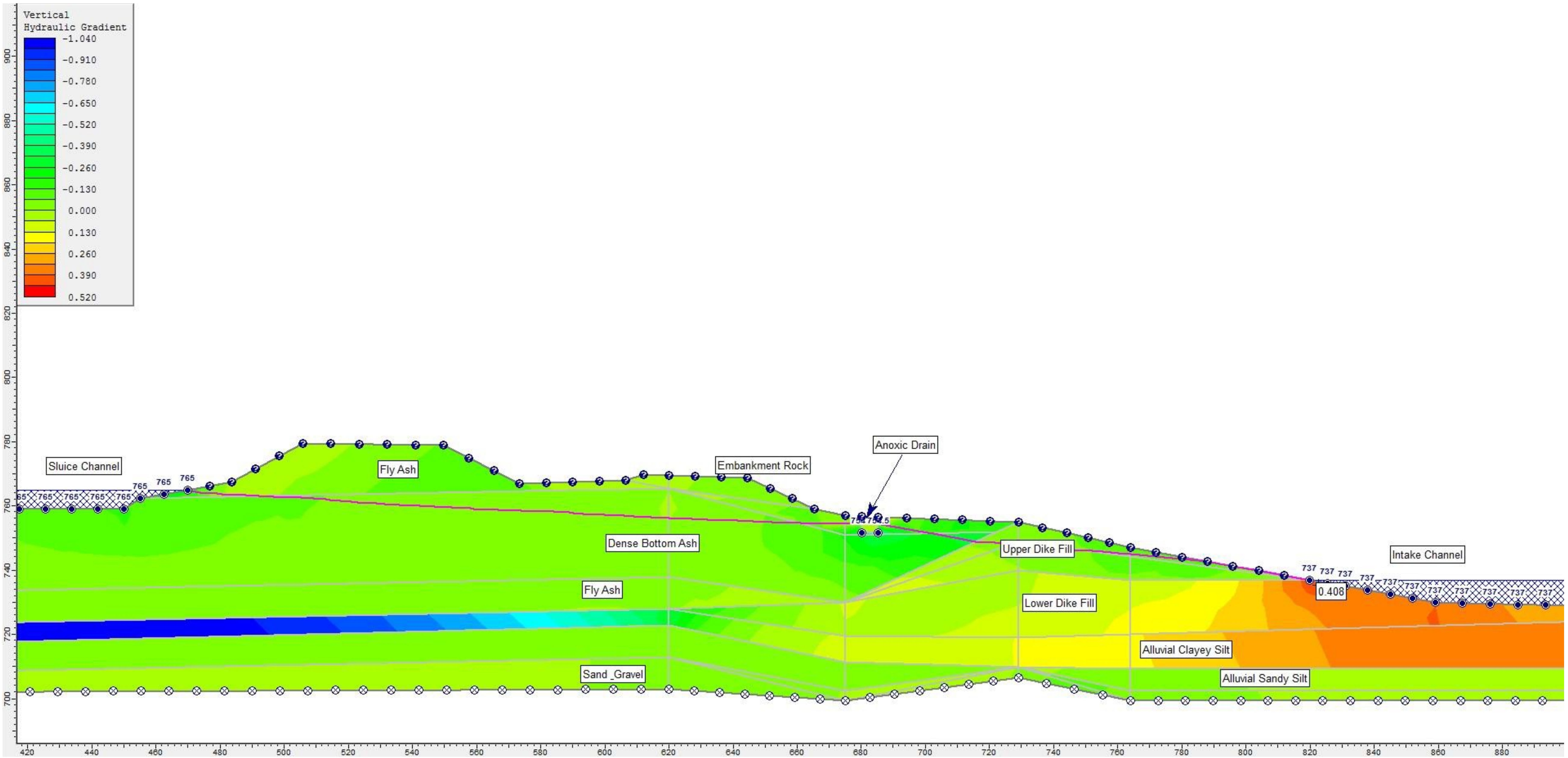


Figure 16b. Results of seepage analysis for cross section A – existing conditions using hydraulic conductivity (k_h) based on soil index, I_c for Upper Dike Fill, Alluvial Sandy Silt, and Sand & Gravel. Hydraulic conductivity for Lower Dike Fill and Alluvial Clayey Silt is based on pore pressure dissipation tests. Maximum hydraulic gradient shown = 0.408. Vertical and horizontal scale is in units of feet.

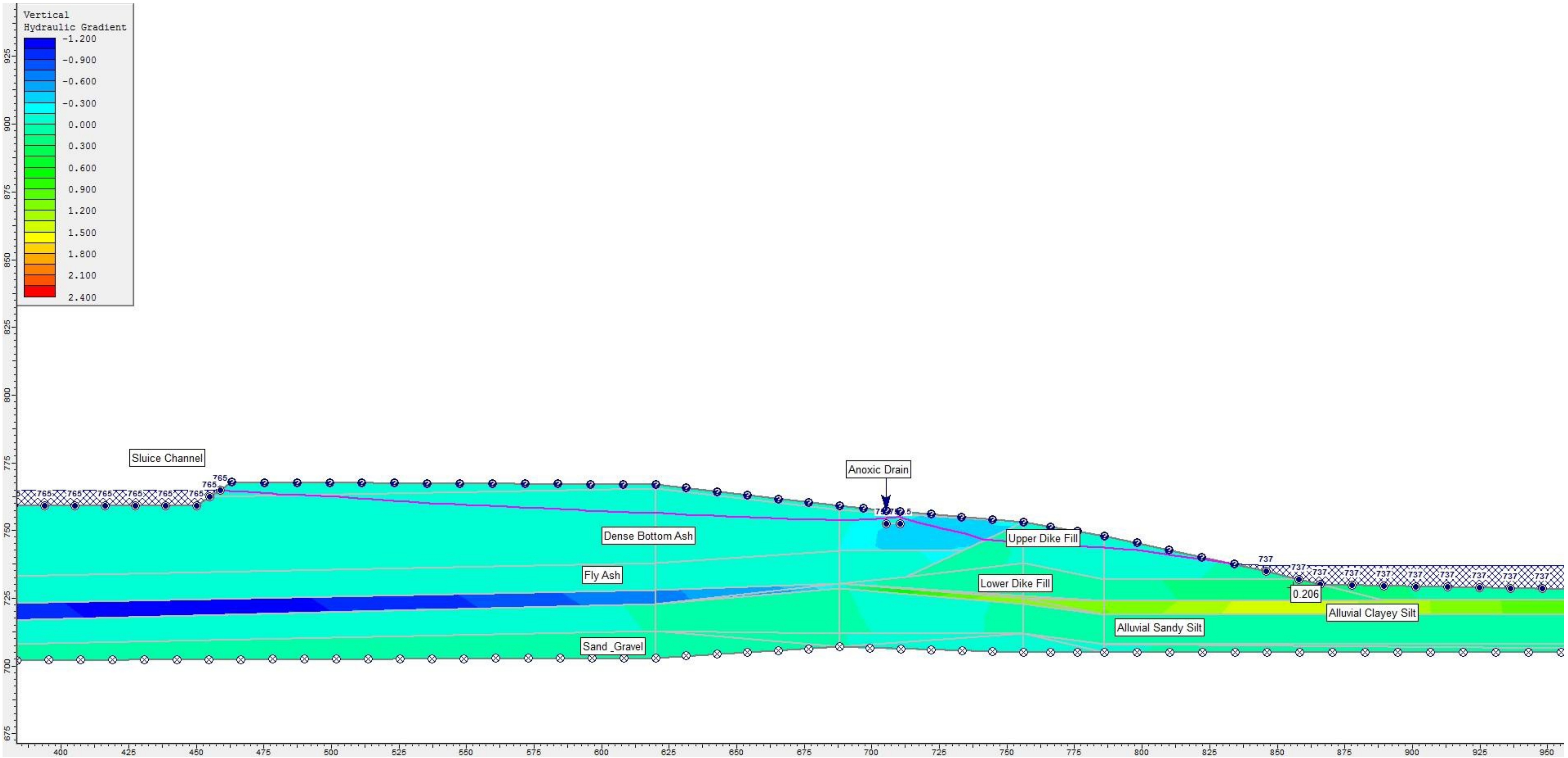


Figure 17a. Results of seepage analysis for cross section B – existing conditions using hydraulic conductivity (k_h) based on soil index, I_c. Maximum hydraulic gradient shown = 0.206. Vertical and horizontal scale is in units of feet.

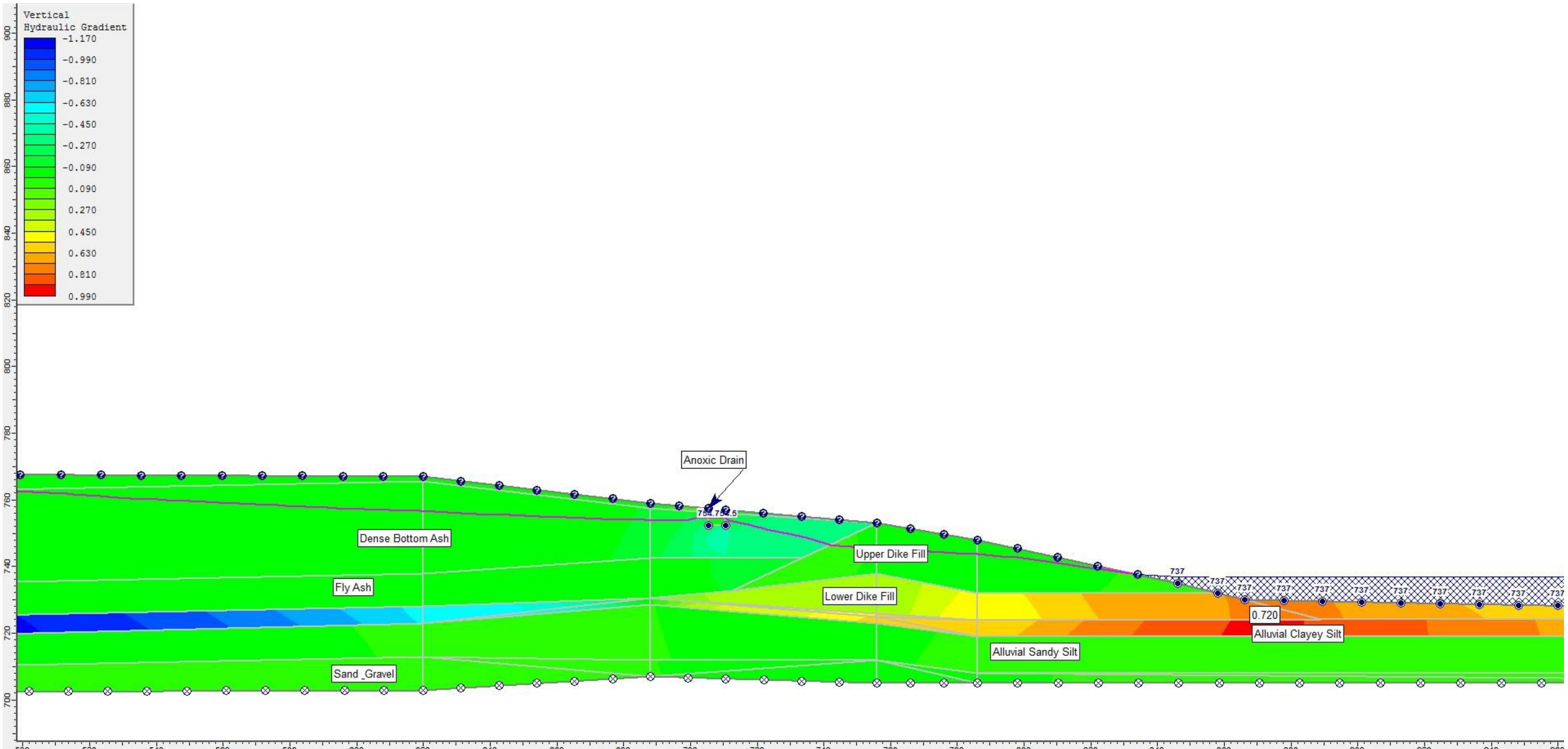


Figure 17b. Results of seepage analysis for cross section B – existing conditions using hydraulic conductivity (k_h) based on soil index, I_c for Upper Dike Fill, Alluvial Sandy Silt, and Sand & Gravel. Hydraulic conductivity for Lower Dike Fill and Alluvial Clayey Silt is based on pore pressure dissipation tests. Maximum hydraulic gradient shown = 0.720. Vertical and horizontal scale is in units of feet.

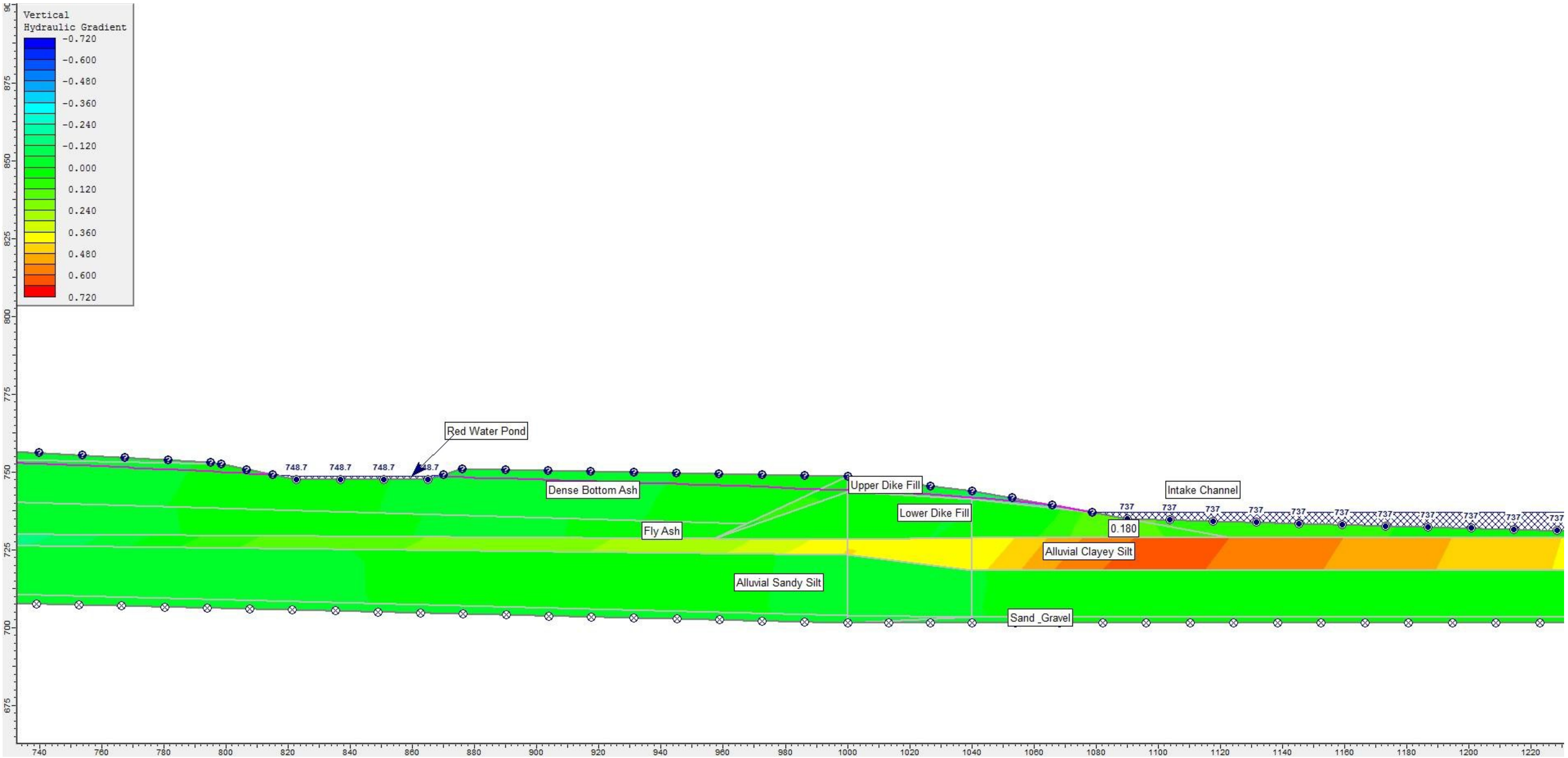


Figure 18a. Results of seepage analysis for cross section C – existing conditions using hydraulic conductivity (k_h) based on soil index, I_c. Maximum hydraulic gradient shown = 0.180. Vertical and horizontal scale is in units of feet.

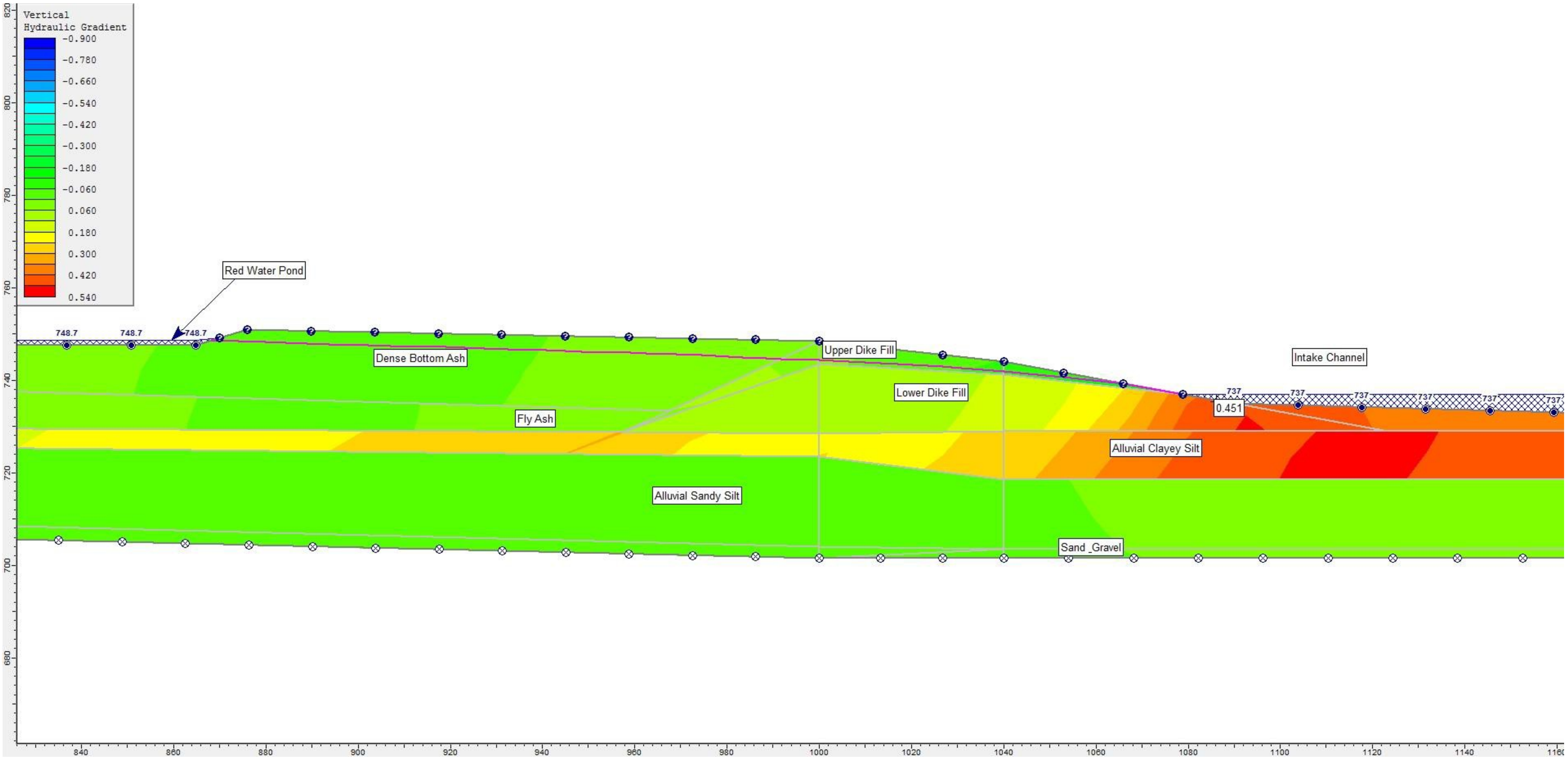


Figure 18b. Results of seepage analysis for cross section C – existing conditions using hydraulic conductivity (k_h) based on soil index, I_c for Upper Dike Fill, Alluvial Sandy Silt, and Sand & Gravel. Hydraulic conductivity for Lower Dike Fill and Alluvial Clayey Silt is based on pore pressure dissipation tests. Maximum hydraulic gradient shown = 0.451. Vertical and horizontal scale is in units of feet.

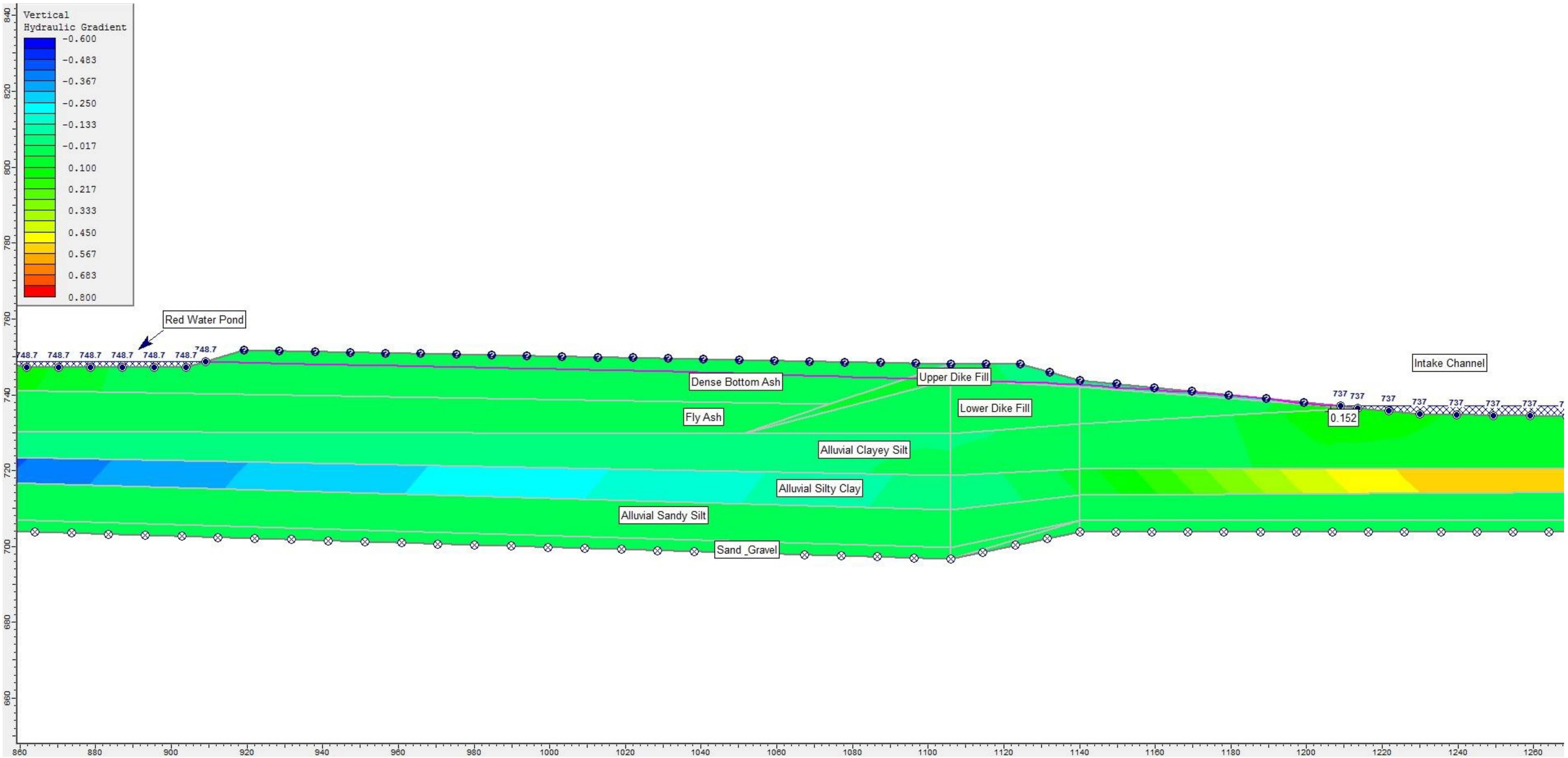


Figure 19a. Results of seepage analysis for cross section D – existing conditions using hydraulic conductivity (k_h) based on soil index, I_c. Maximum hydraulic gradient shown = 0.152. Vertical and horizontal scale is in units of feet.

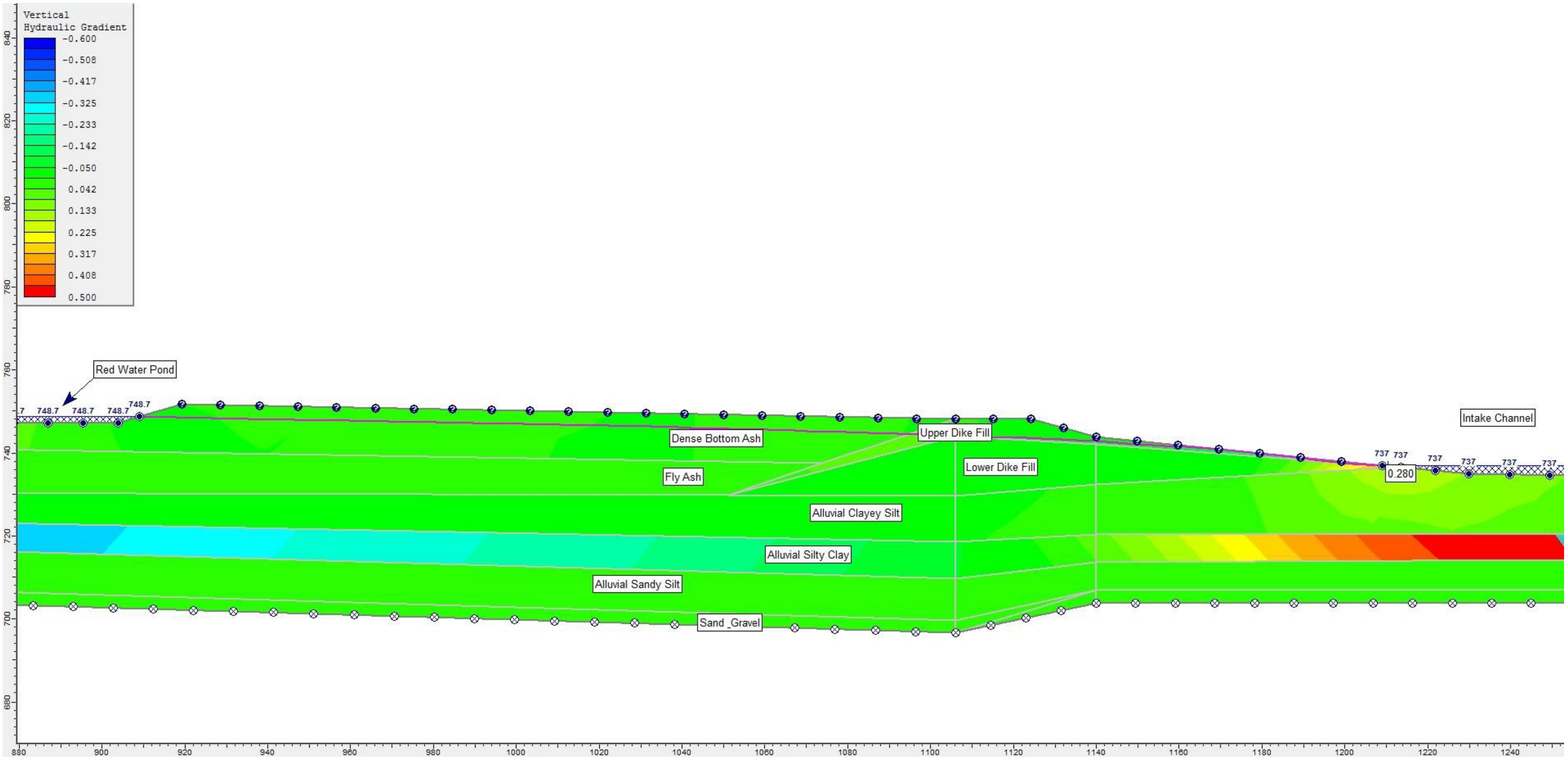


Figure 19b. Results of seepage analysis for cross section D – existing conditions using hydraulic conductivity (k_h) based on soil index, I_c for Upper Dike Fill, Alluvial Sandy Silt, Alluvial Silty Clay and Sand & Gravel. Hydraulic conductivity for Lower Dike Fill and Alluvial Clayey Silt is based on pore pressure dissipation tests. Maximum hydraulic gradient shown = 0.280. Vertical and horizontal scale is in units of feet.

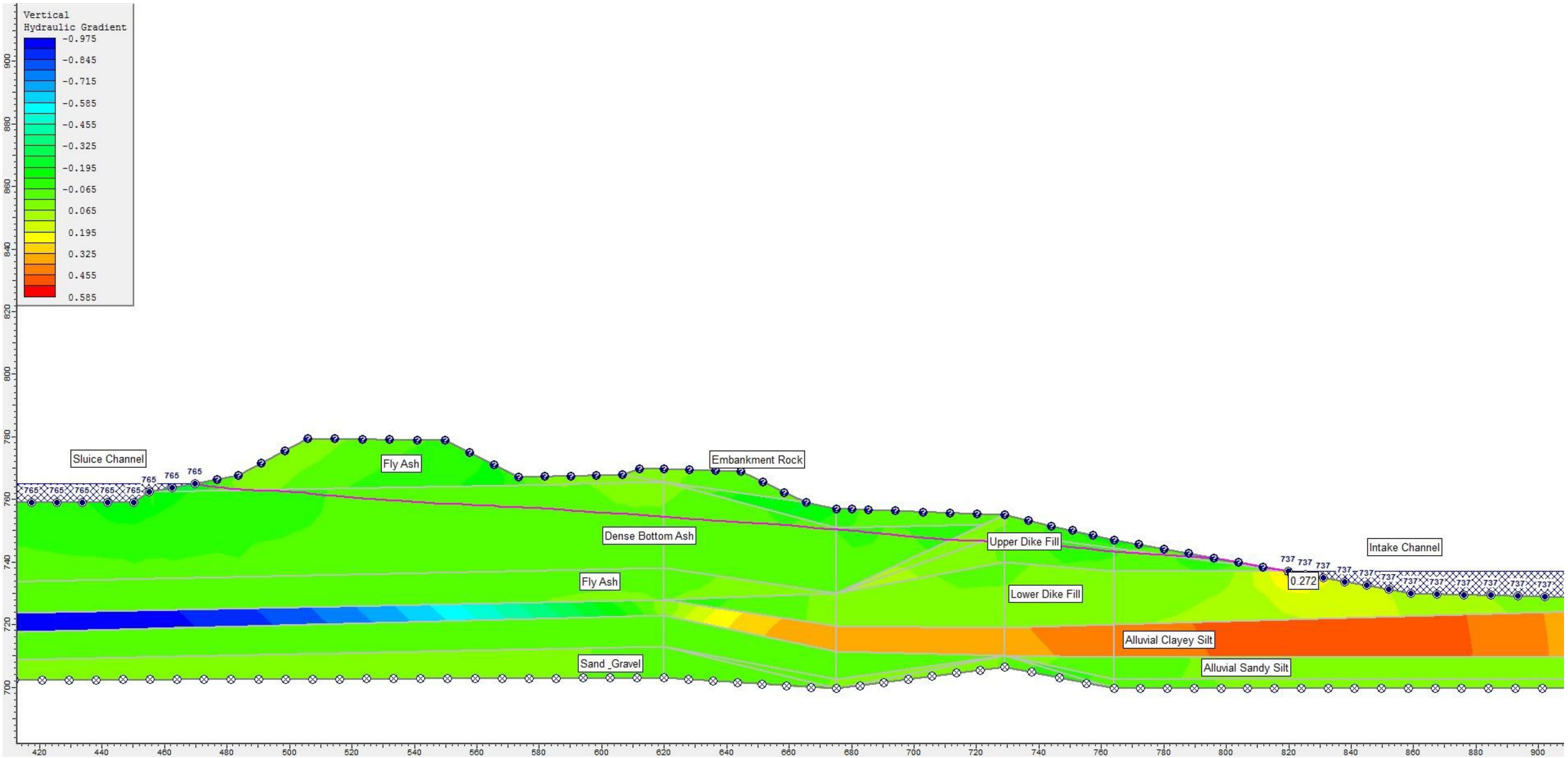


Figure 20a. Results of seepage analysis for cross section A – without the anoxic drain using hydraulic conductivity (k_h) based on soil index, I_c. Maximum hydraulic gradient shown = 0.272. Vertical and horizontal scale is in units of feet.

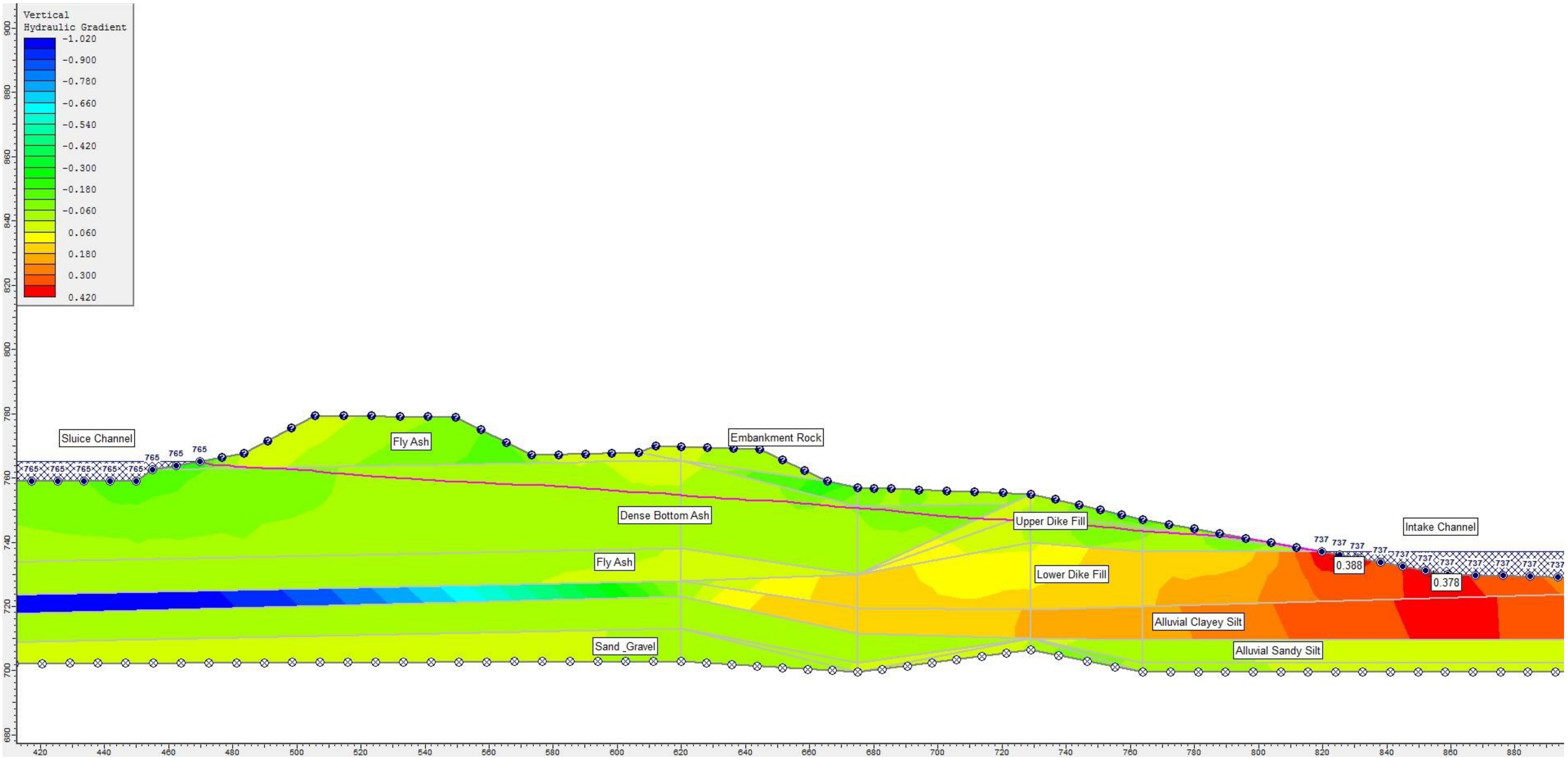


Figure 20b. Results of seepage analysis for cross section A – without the anoxic drain using hydraulic conductivity (k_h) based on soil index, I_c for Upper Dike Fill, Alluvial Sandy Silt, and Sand & Gravel. Hydraulic conductivity for Lower Dike Fill and Alluvial Clayey Silt is based on pore pressure dissipation tests. Maximum hydraulic gradient shown = 0.388. Vertical and horizontal scale is in units of feet.

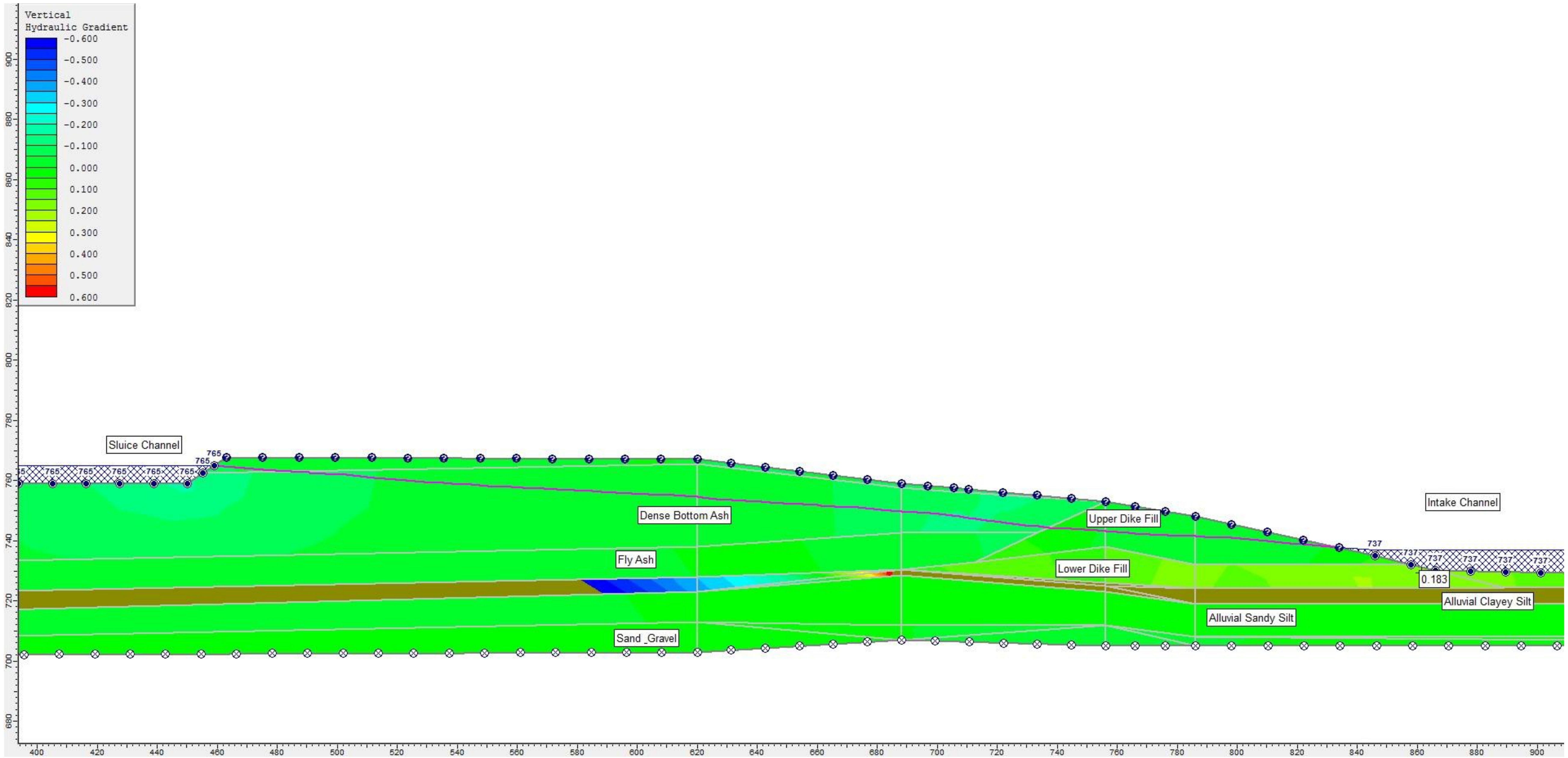


Figure 21a. Results of seepage analysis for cross section B – without the anoxic drain using hydraulic conductivity (k_h) based on soil index, I_c. Maximum hydraulic gradient shown = 0.183. Vertical and horizontal scale is in units of feet.

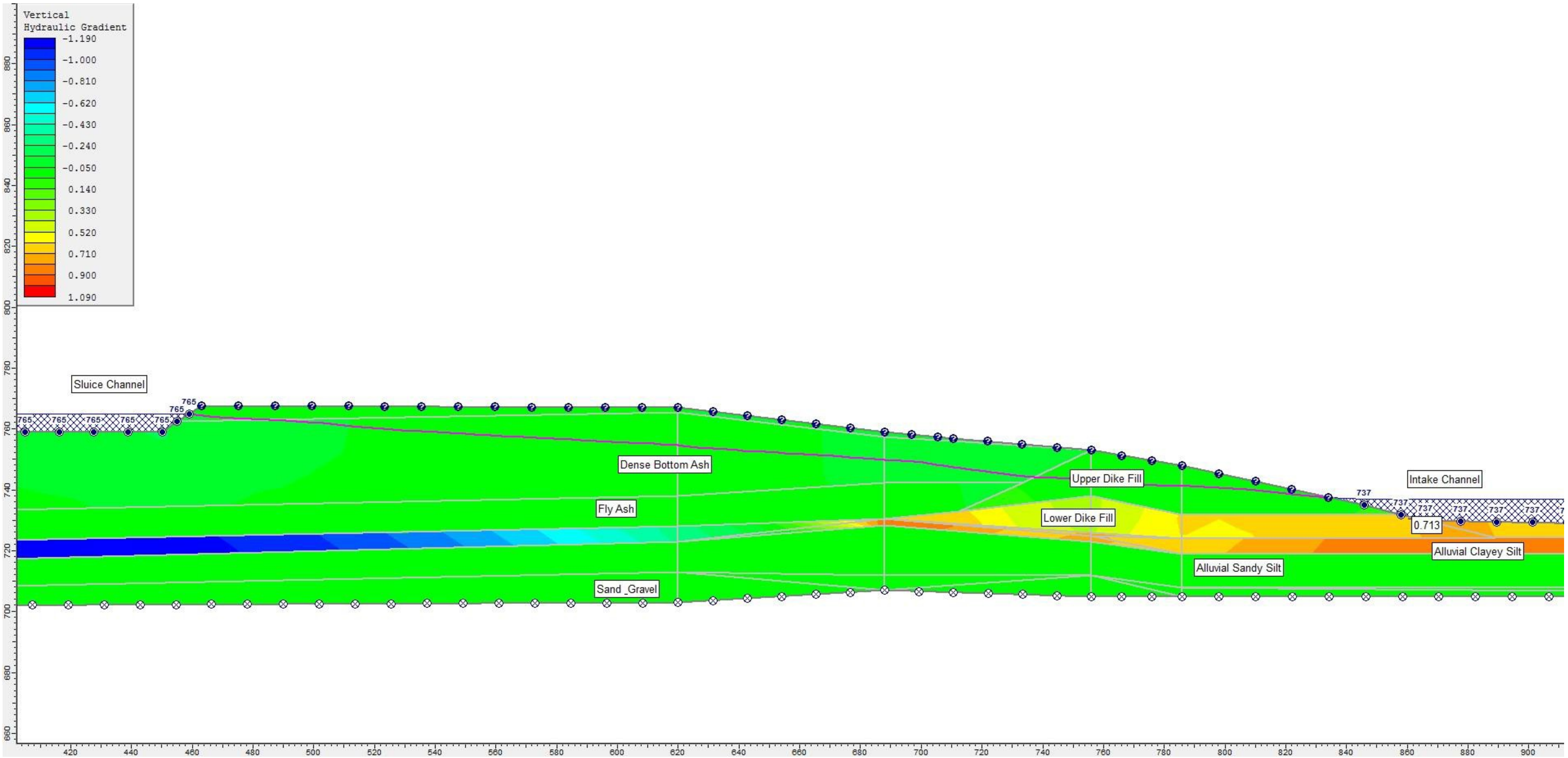


Figure 21b. Results of seepage analysis for cross section B – without the anoxic drain using hydraulic conductivity (k_h) based on soil index, I_c for Upper Dike Fill, Alluvial Sandy Silt, and Sand & Gravel. Hydraulic conductivity for Lower Dike Fill and Alluvial Clayey Silt is based on pore pressure dissipation tests. Maximum hydraulic gradient shown = 0.713. Vertical and horizontal scale is in units of feet.

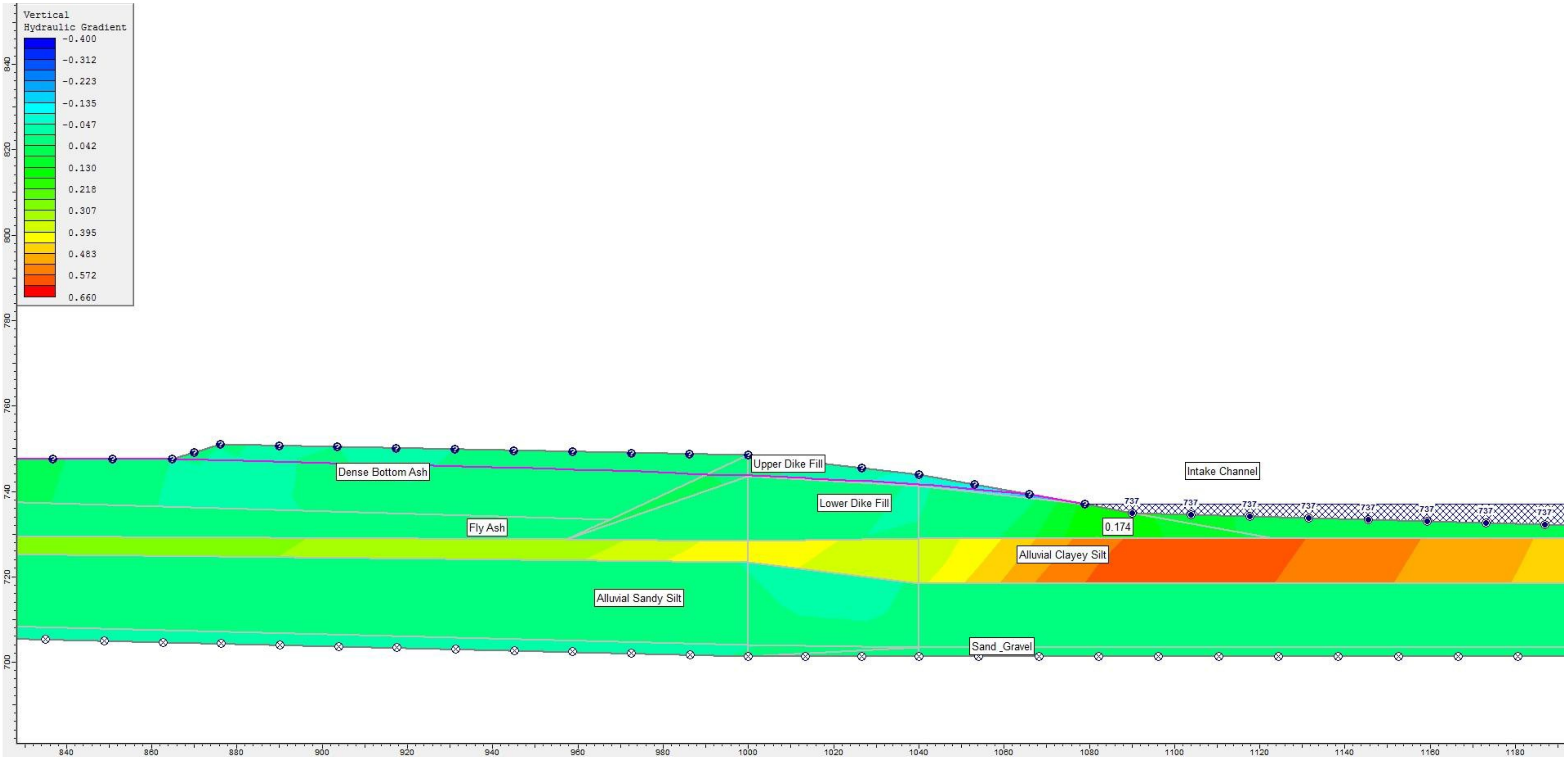


Figure 22a. Results of seepage analysis for cross section C – without the red water pond using hydraulic conductivity (k_h) based on soil index, I_c. Maximum hydraulic gradient shown = 0.174. Vertical and horizontal scale is in units of feet.

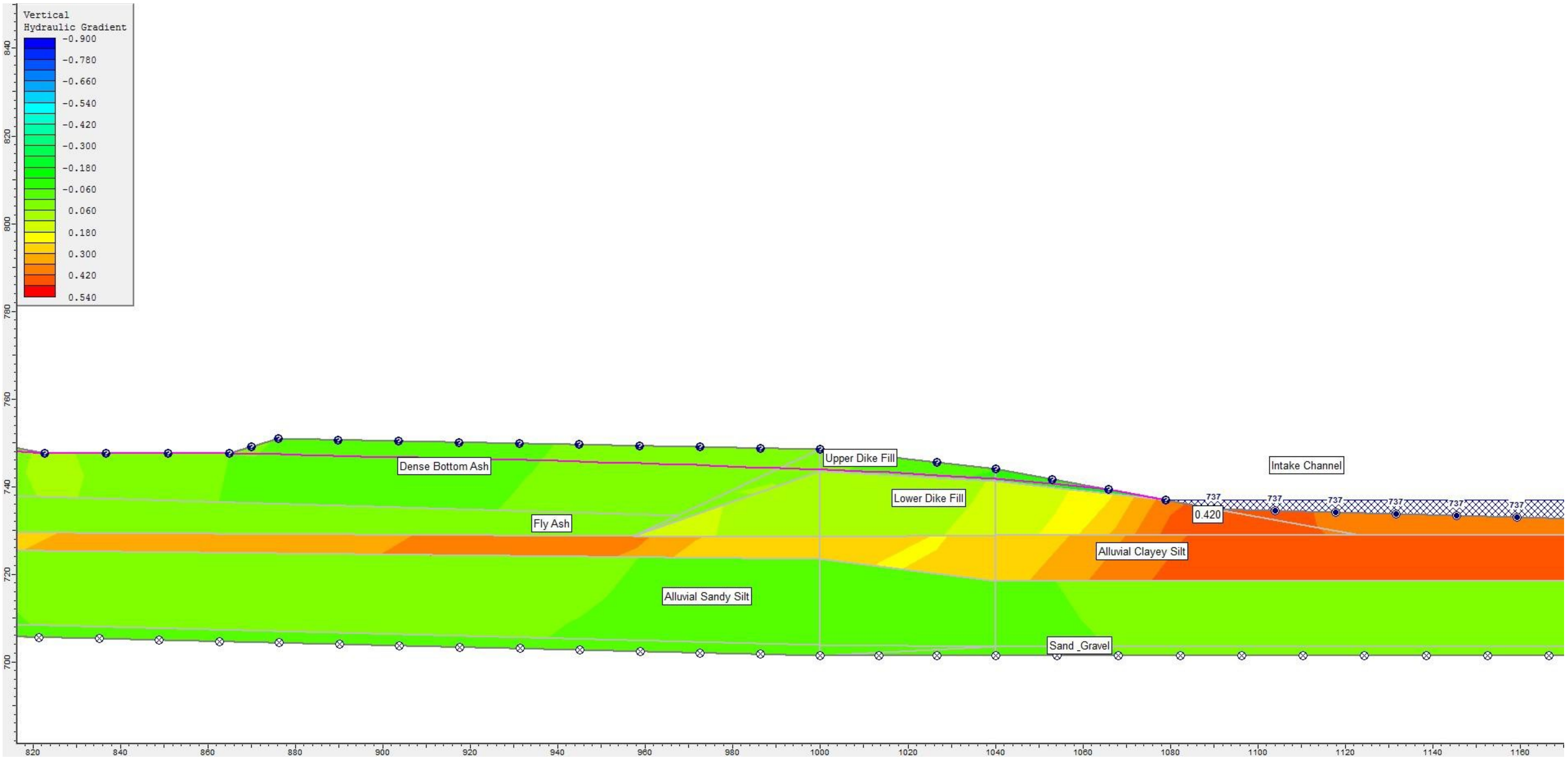


Figure 22b. Results of seepage analysis for cross section C – without the red water pond using hydraulic conductivity (k_h) based on soil index, I_c for Upper Dike Fill, Alluvial Sandy Silt, and Sand & Gravel. Hydraulic conductivity for Lower Dike Fill and Alluvial Clayey Silt is based on pore pressure dissipation tests. Maximum hydraulic gradient shown = 0.420. Vertical and horizontal scale is in units of feet.

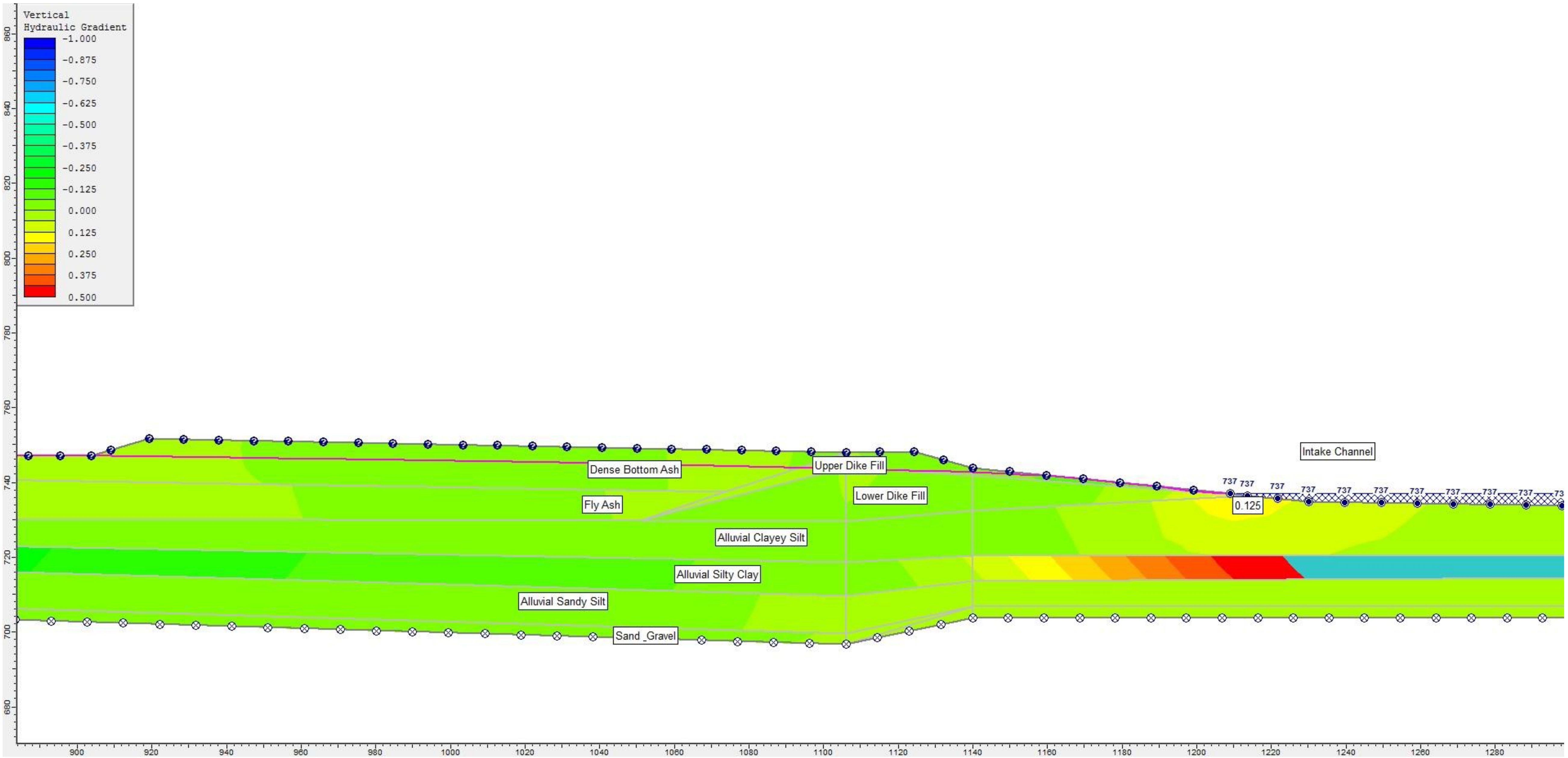


Figure 23a. Results of seepage analysis for cross section D – without the red water pond using hydraulic conductivity (k_h) based on soil index, I_c. Maximum hydraulic gradient shown = 0.125. Vertical and horizontal scale is in units of feet.

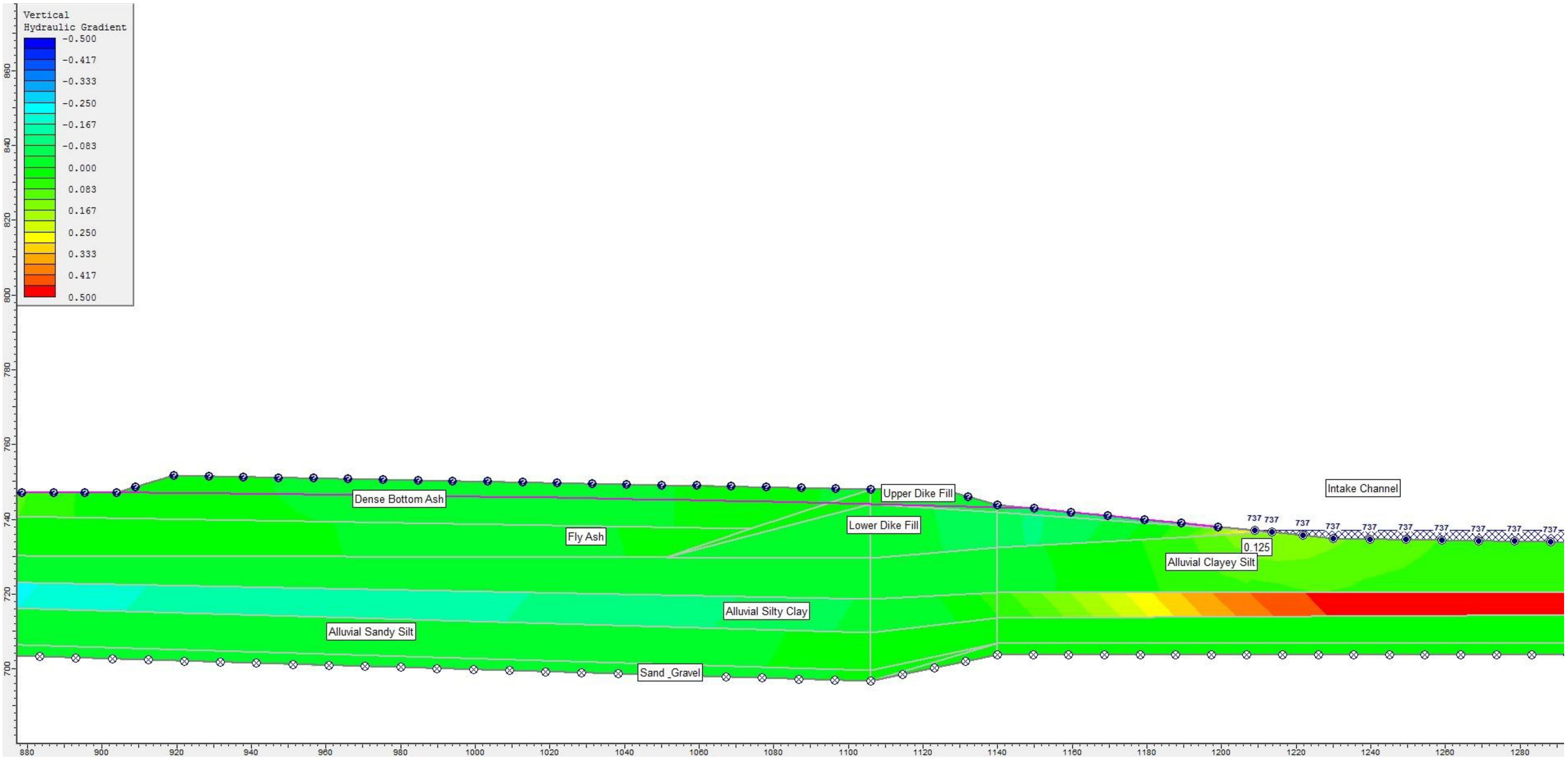


Figure 23b. Results of seepage analysis for cross section D – without the red water pond using hydraulic conductivity (k_h) based on soil index, I_c for Upper Dike Fill, Alluvial Sandy Silt, Alluvial Silty Clay and Sand & Gravel. Hydraulic conductivity for Lower Dike Fill and Alluvial Clayey Silt is based on pore pressure dissipation tests. Maximum hydraulic gradient shown = 0.125. Vertical and horizontal scale is in units of feet.

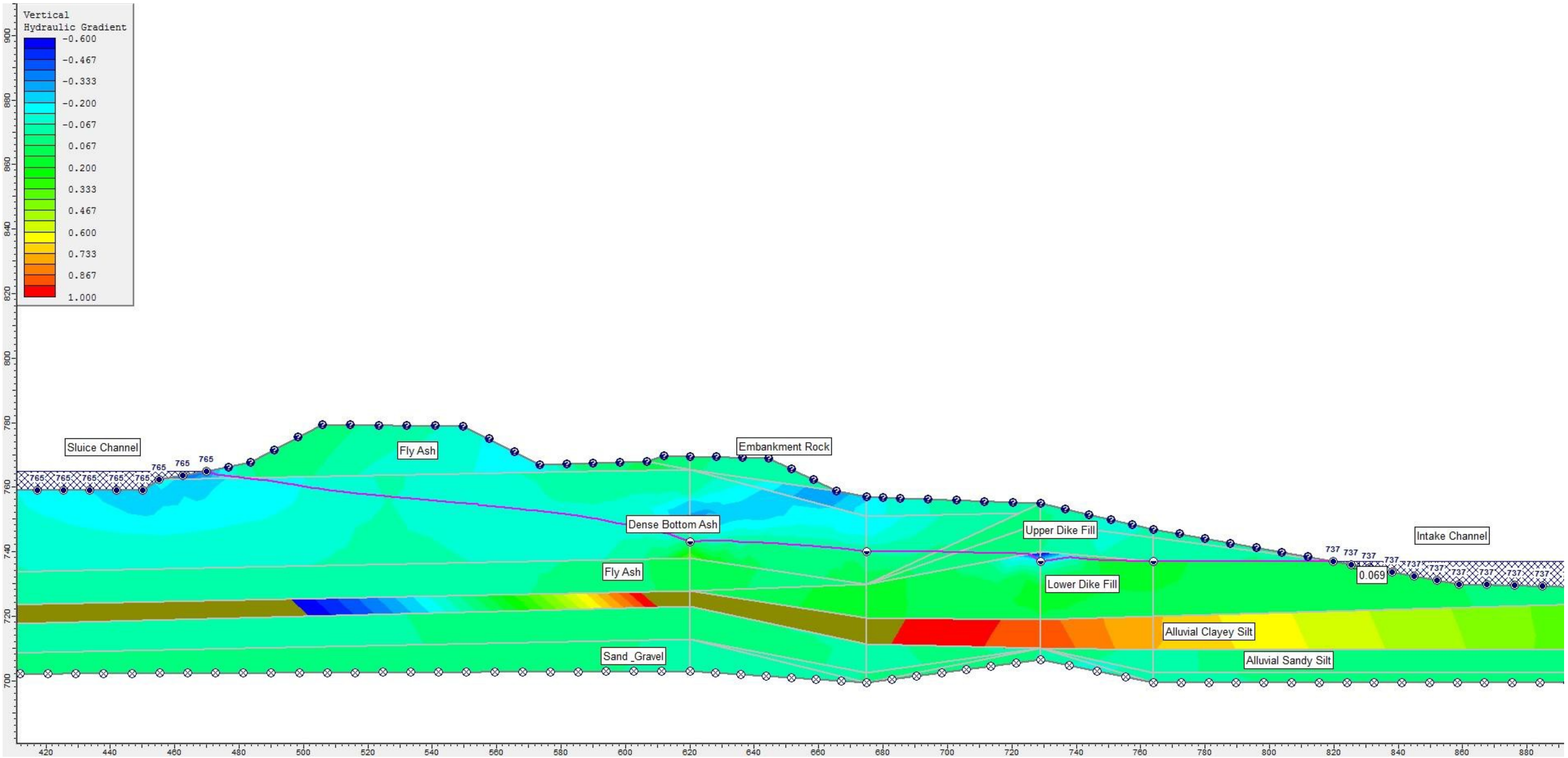


Figure 24a. Results of seepage analysis for cross section A – assuming a depressed water table using hydraulic conductivity (k_h) based on soil index, I_c. Maximum hydraulic gradient shown = 0.069. Vertical and horizontal scale is in units of feet.

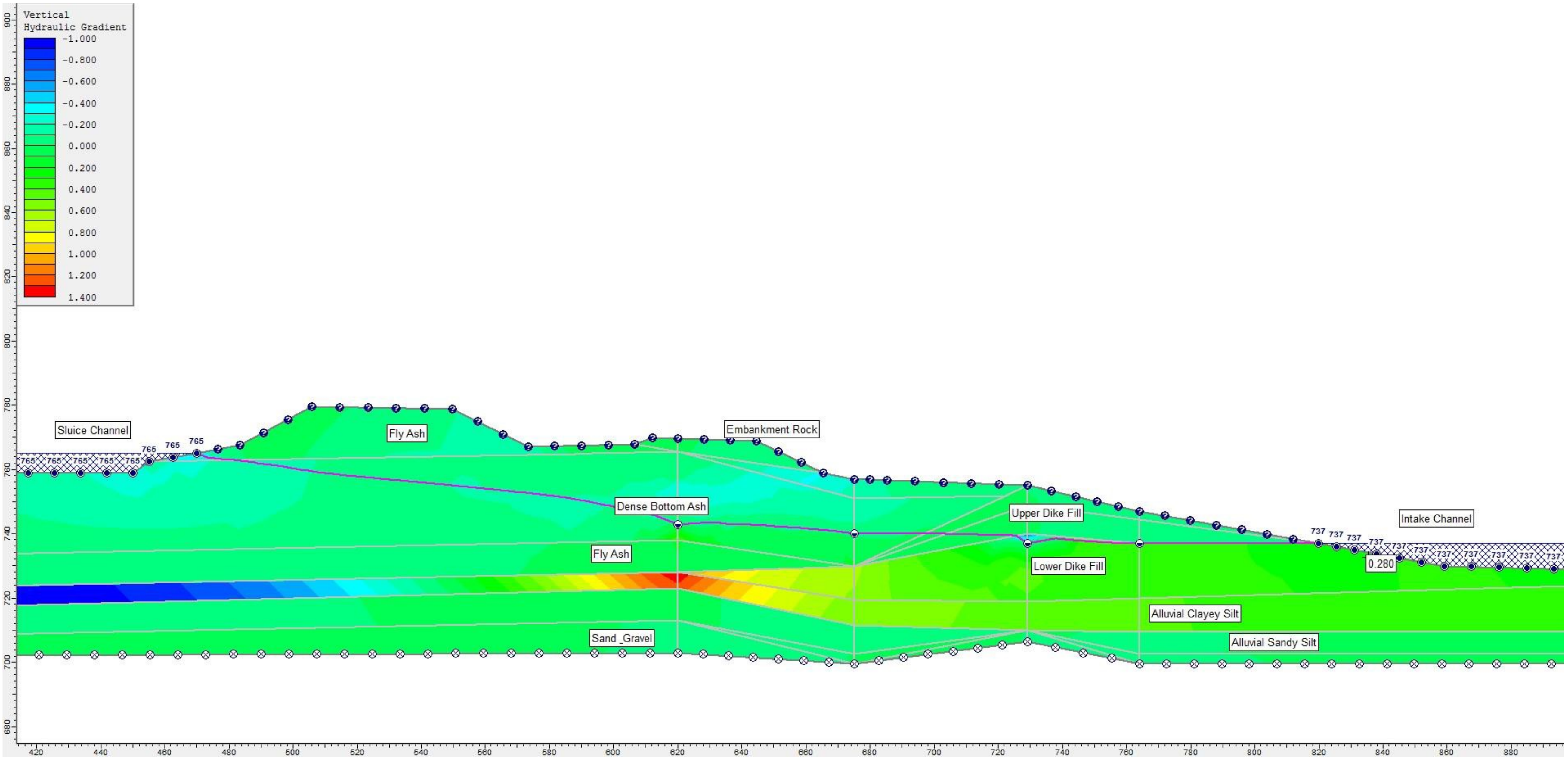


Figure 24b. Results of seepage analysis for cross section A – assuming a depressed water table using hydraulic conductivity (k_h) based on soil index, I_c for Upper Dike Fill, Alluvial Sandy Silt, and Sand & Gravel. Hydraulic conductivity for Lower Dike Fill and Alluvial Clayey Silt is based on pore pressure dissipation tests. Maximum hydraulic gradient shown = 0.280. Vertical and horizontal scale is in units of feet.

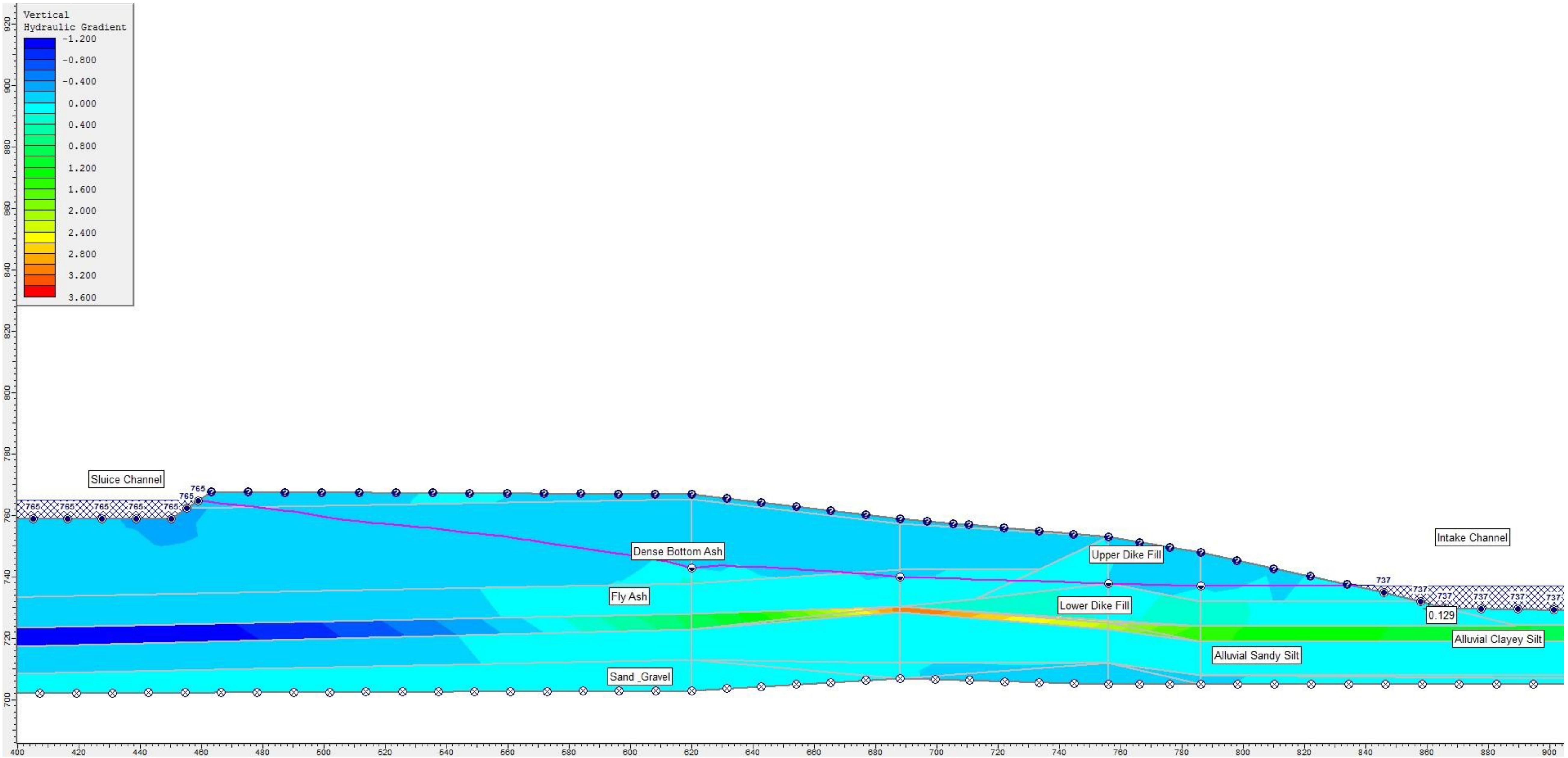


Figure 25a. Results of seepage analysis for cross section B – assuming a depressed water table using hydraulic conductivity (k_h) based on soil index, I_c. Maximum hydraulic gradient shown = 0.129. Vertical and horizontal scale is in units of feet.

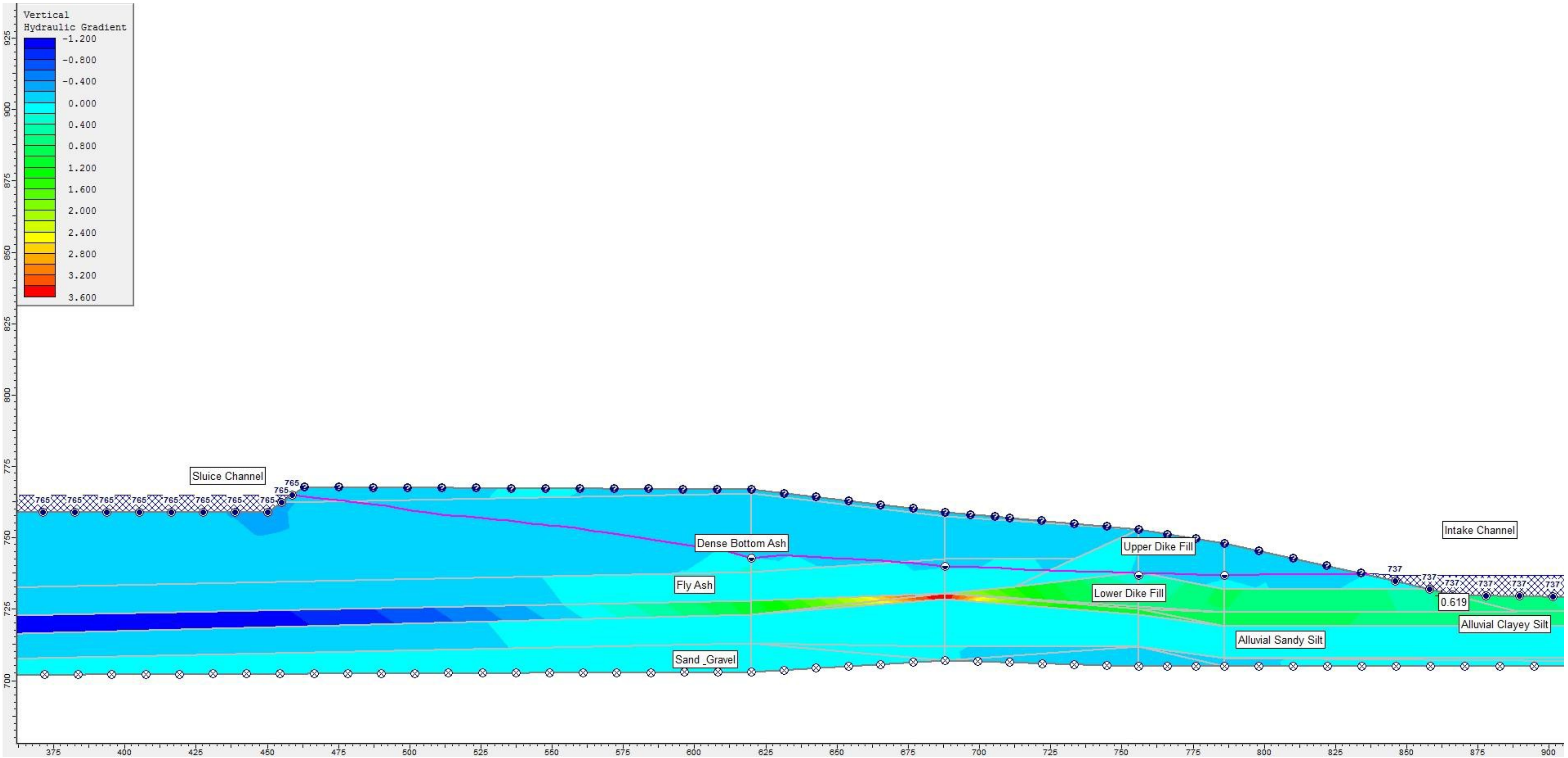


Figure 25b. Results of seepage analysis for cross section B – assuming a depressed water table using hydraulic conductivity (k_h) based on soil index, I_c for Upper Dike Fill, Alluvial Sandy Silt, and Sand & Gravel. Hydraulic conductivity for Lower Dike Fill and Alluvial Clayey Silt is based on pore pressure dissipation tests. Maximum hydraulic gradient shown = 0.619. Vertical and horizontal scale is in units of feet.

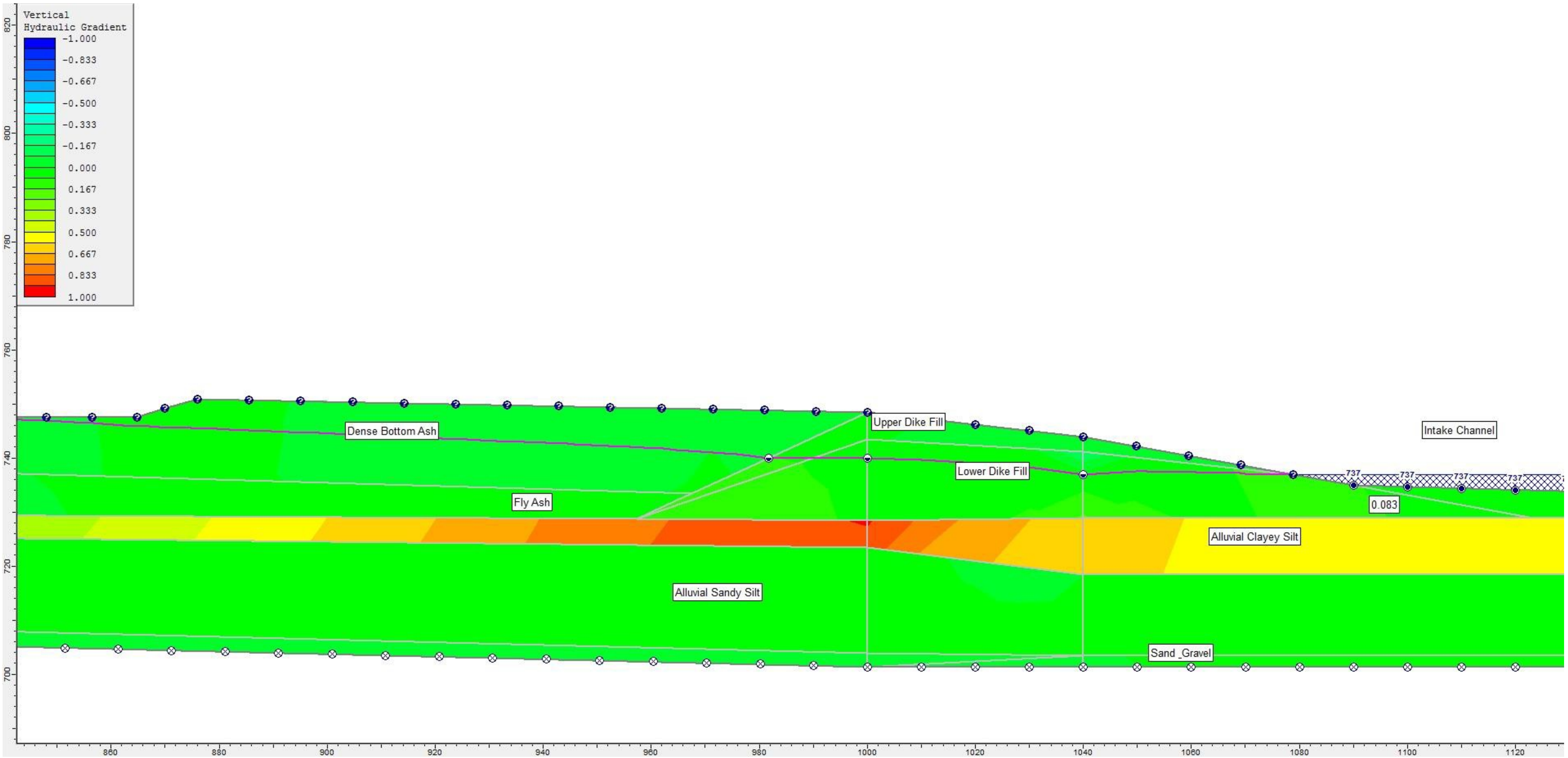


Figure 26a. Results of seepage analysis for cross section C – assuming a depressed water table using hydraulic conductivity (k_h) based on soil index, I_c. Maximum hydraulic gradient shown = 0.083. Vertical and horizontal scale is in units of feet.

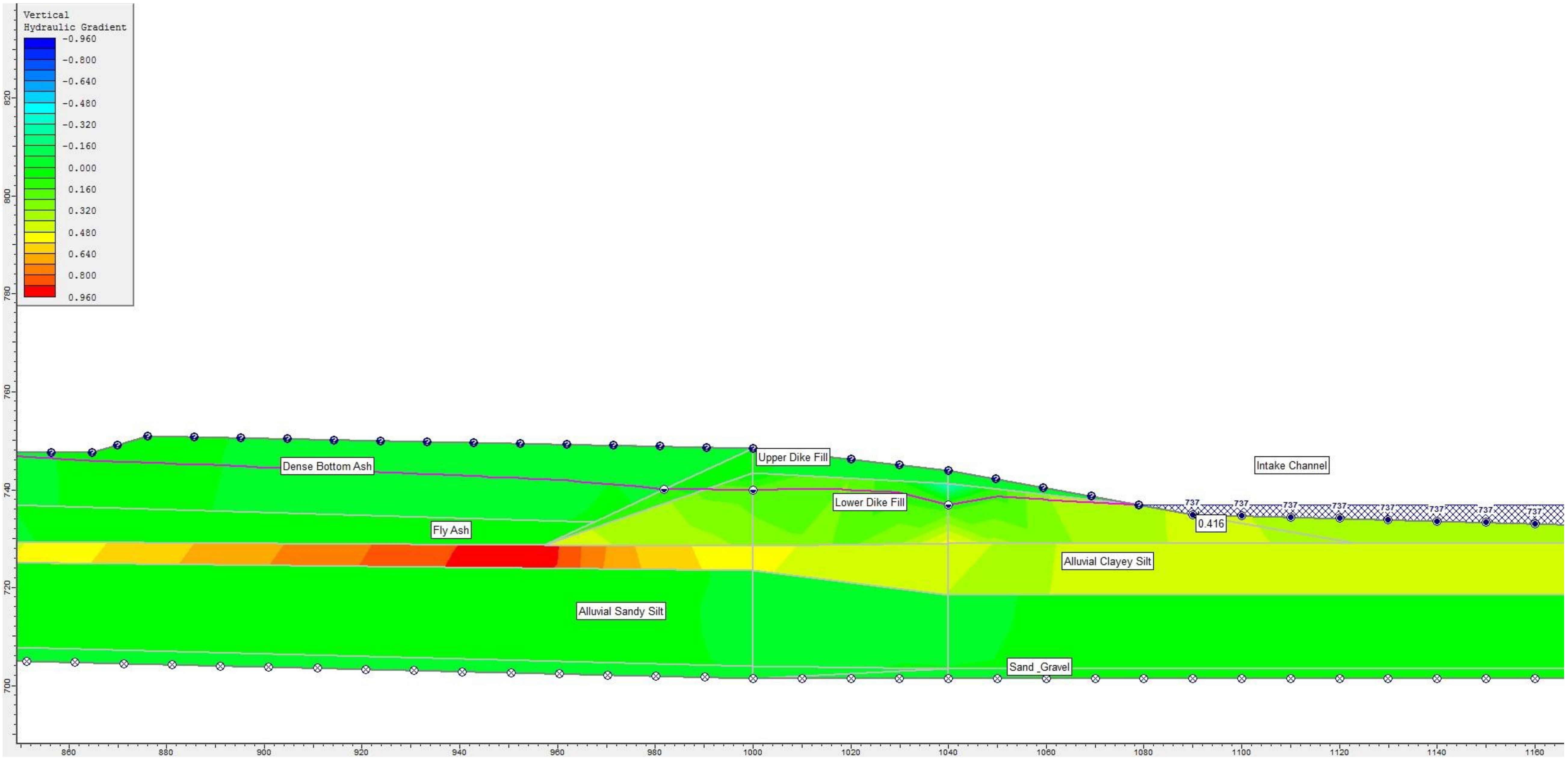


Figure 26b. Results of seepage analysis for cross section C – assuming a depressed water table using hydraulic conductivity (k_h) based on soil index, I_c for Upper Dike Fill, Alluvial Sandy Silt, and Sand & Gravel. Hydraulic conductivity for Lower Dike Fill and Alluvial Clayey Silt is based on pore pressure dissipation tests. Maximum hydraulic gradient shown = 0.416. Vertical and horizontal scale is in units of feet.

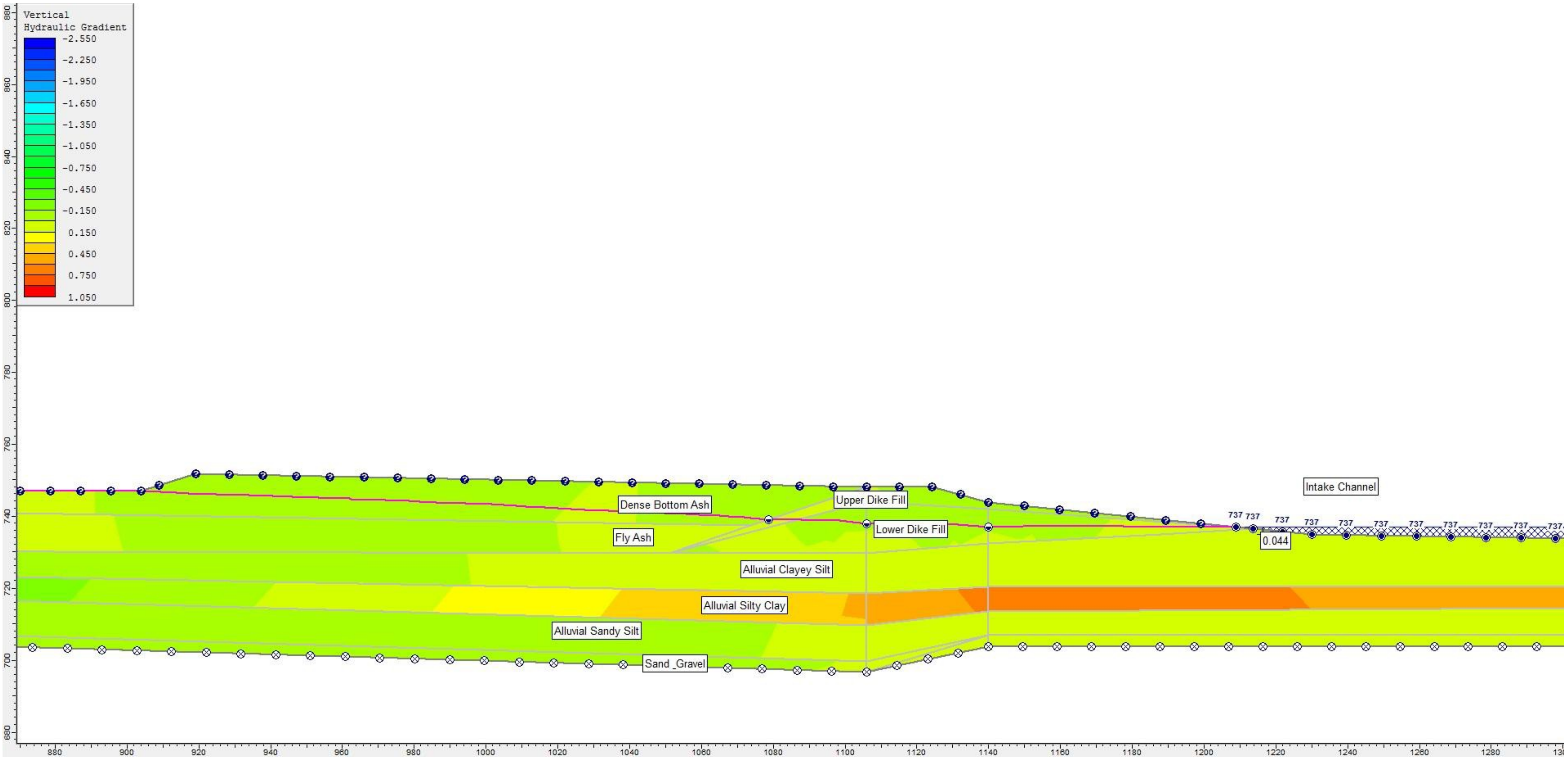


Figure 27a. Results of seepage analysis for cross section D – assuming a depressed water table using hydraulic conductivity (k_h) based on soil index, I_c. Maximum hydraulic gradient shown = 0.044. Vertical and horizontal scale is in units of feet.

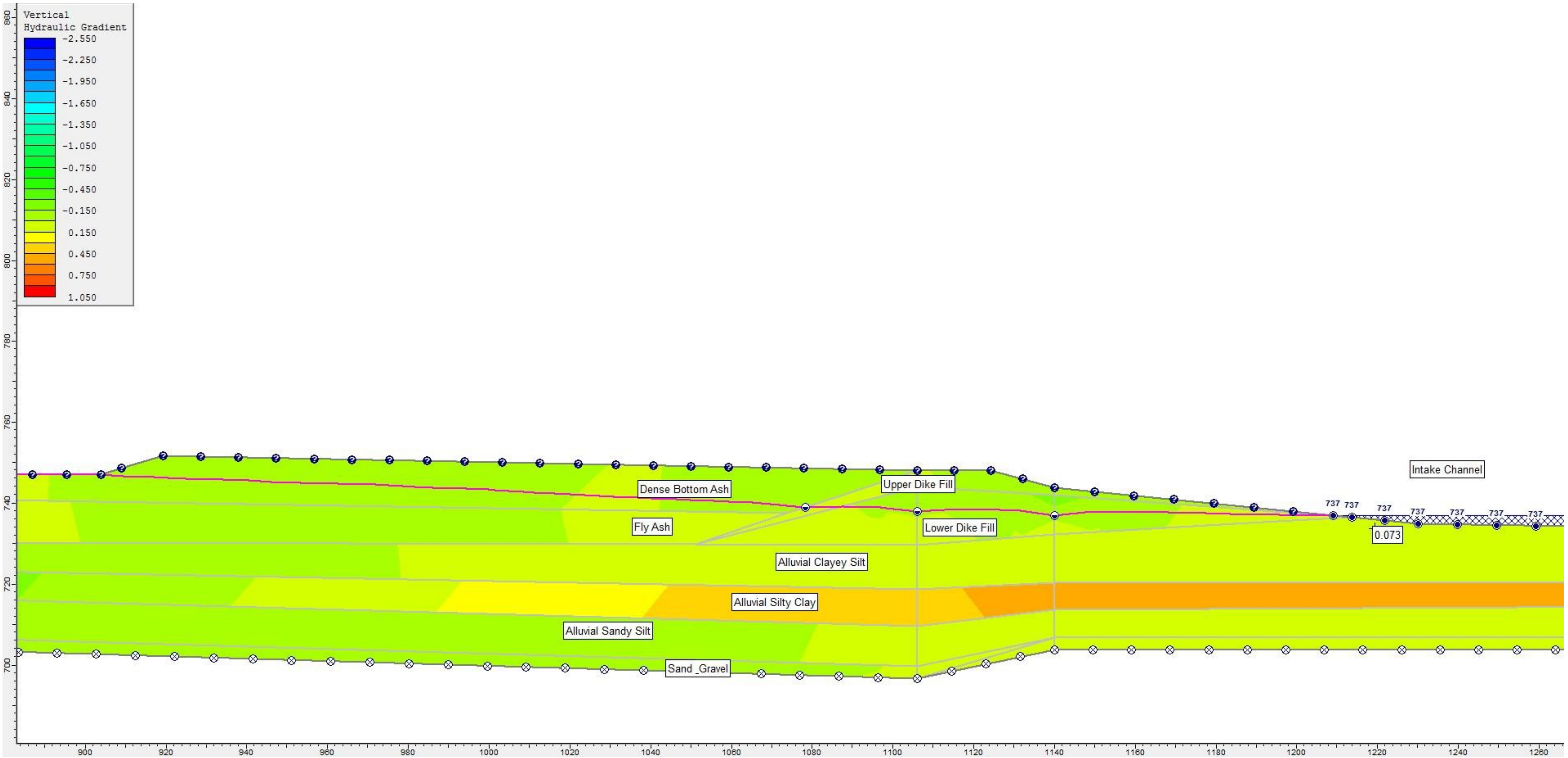
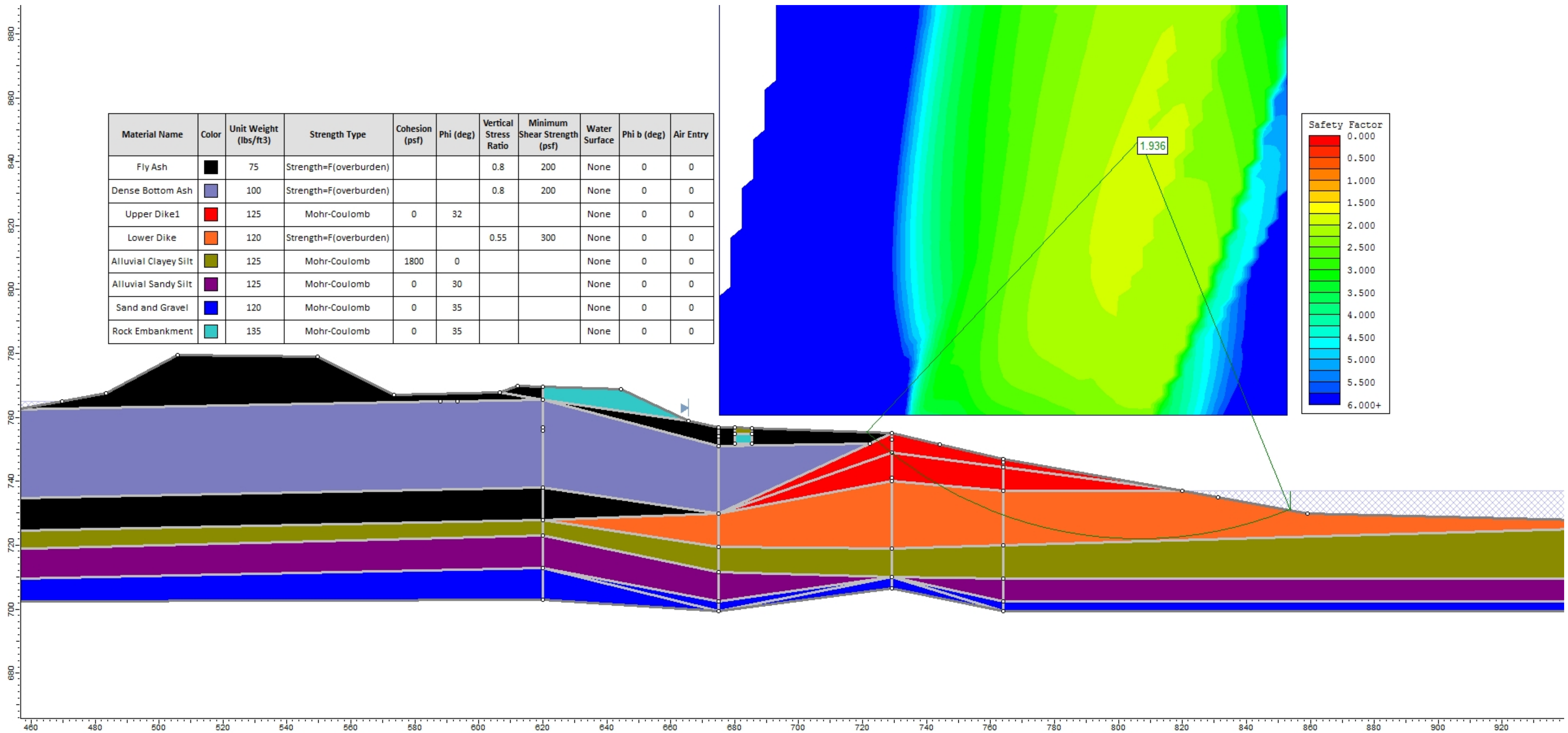
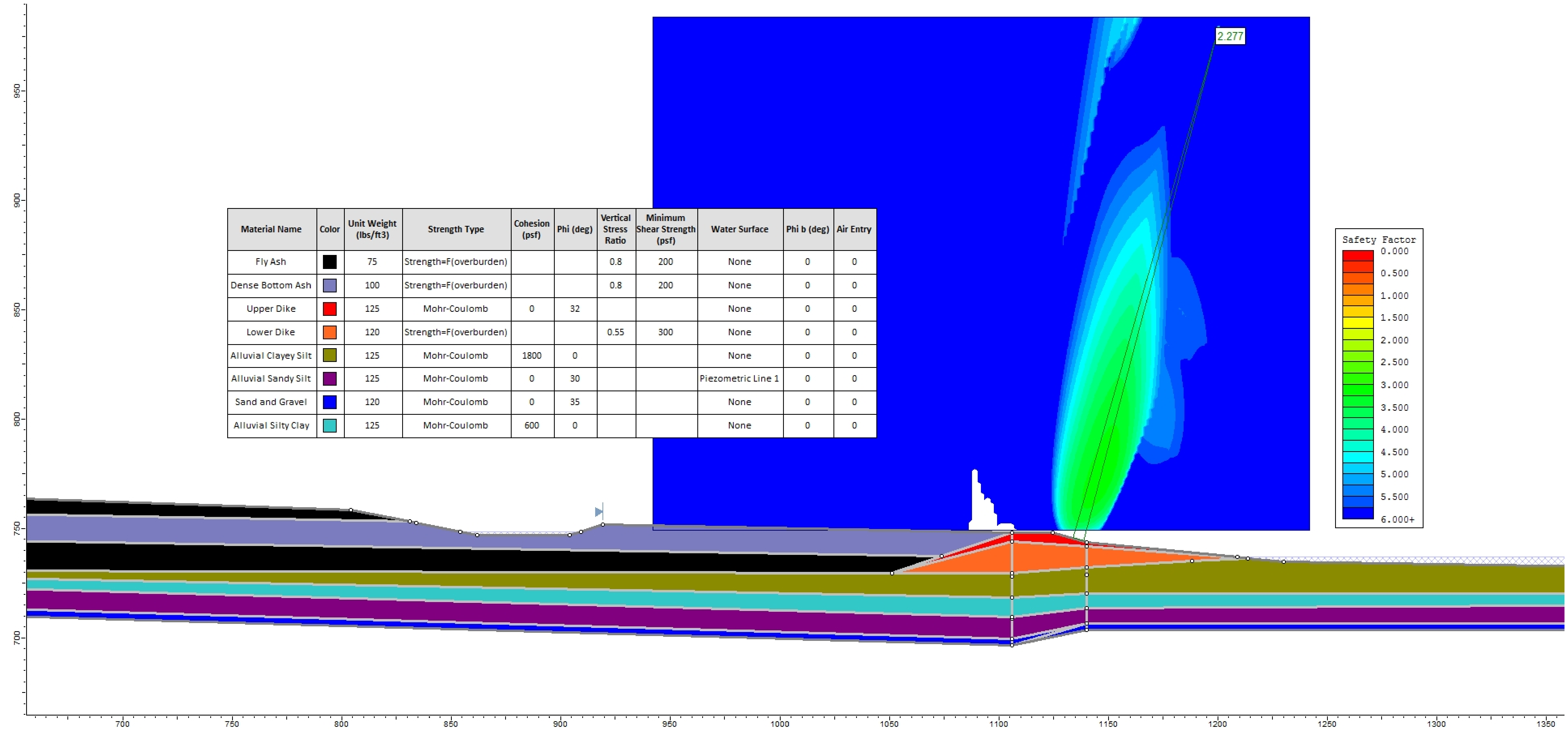


Figure 27b. Results of seepage analysis for cross section D – assuming a depressed water table using hydraulic conductivity (k_h) based on soil index, I_c for Upper Dike Fill, Alluvial Sandy Silt, Alluvial Silty Clay and Sand & Gravel. Hydraulic conductivity for Lower Dike Fill and Alluvial Clayey Silt is based on pore pressure dissipation tests. Maximum hydraulic gradient shown = 0.073. Vertical and horizontal scale is in units of feet.



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Vertical Stress Ratio	Minimum Shear Strength (psf)	Water Surface	Phi b (deg)	Air Entry
Fly Ash		75	Strength=F(overburden)			0.8	200	None	0	0
Dense Bottom Ash		100	Strength=F(overburden)			0.8	200	None	0	0
Upper Dike1		125	Mohr-Coulomb	0	32			None	0	0
Lower Dike		120	Strength=F(overburden)			0.55	300	None	0	0
Alluvial Clayey Silt		125	Mohr-Coulomb	1800	0			None	0	0
Alluvial Sandy Silt		125	Mohr-Coulomb	0	30			None	0	0
Sand and Gravel		120	Mohr-Coulomb	0	35			None	0	0
Rock Embankment		135	Mohr-Coulomb	0	35			None	0	0

Figure 28. Results of stability analysis for cross section A- Existing Conditions using pore pressures calculated from the seepage FEA (scale in feet).



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Vertical Stress Ratio	Minimum Shear Strength (psf)	Water Surface	Phi b (deg)	Air Entry
Fly Ash		75	Strength=F(overburden)			0.8	200	None	0	0
Dense Bottom Ash		100	Strength=F(overburden)			0.8	200	None	0	0
Upper Dike		125	Mohr-Coulomb	0	32			None	0	0
Lower Dike		120	Strength=F(overburden)			0.55	300	None	0	0
Alluvial Clayey Silt		125	Mohr-Coulomb	1800	0			None	0	0
Alluvial Sandy Silt		125	Mohr-Coulomb	0	30			Piezometric Line 1	0	0
Sand and Gravel		120	Mohr-Coulomb	0	35			None	0	0
Alluvial Silty Clay		125	Mohr-Coulomb	600	0			None	0	0

Figure 29. Results of stability analysis for cross section D – Existing Conditions using pore pressures calculated from the seepage FEA (scale in feet).

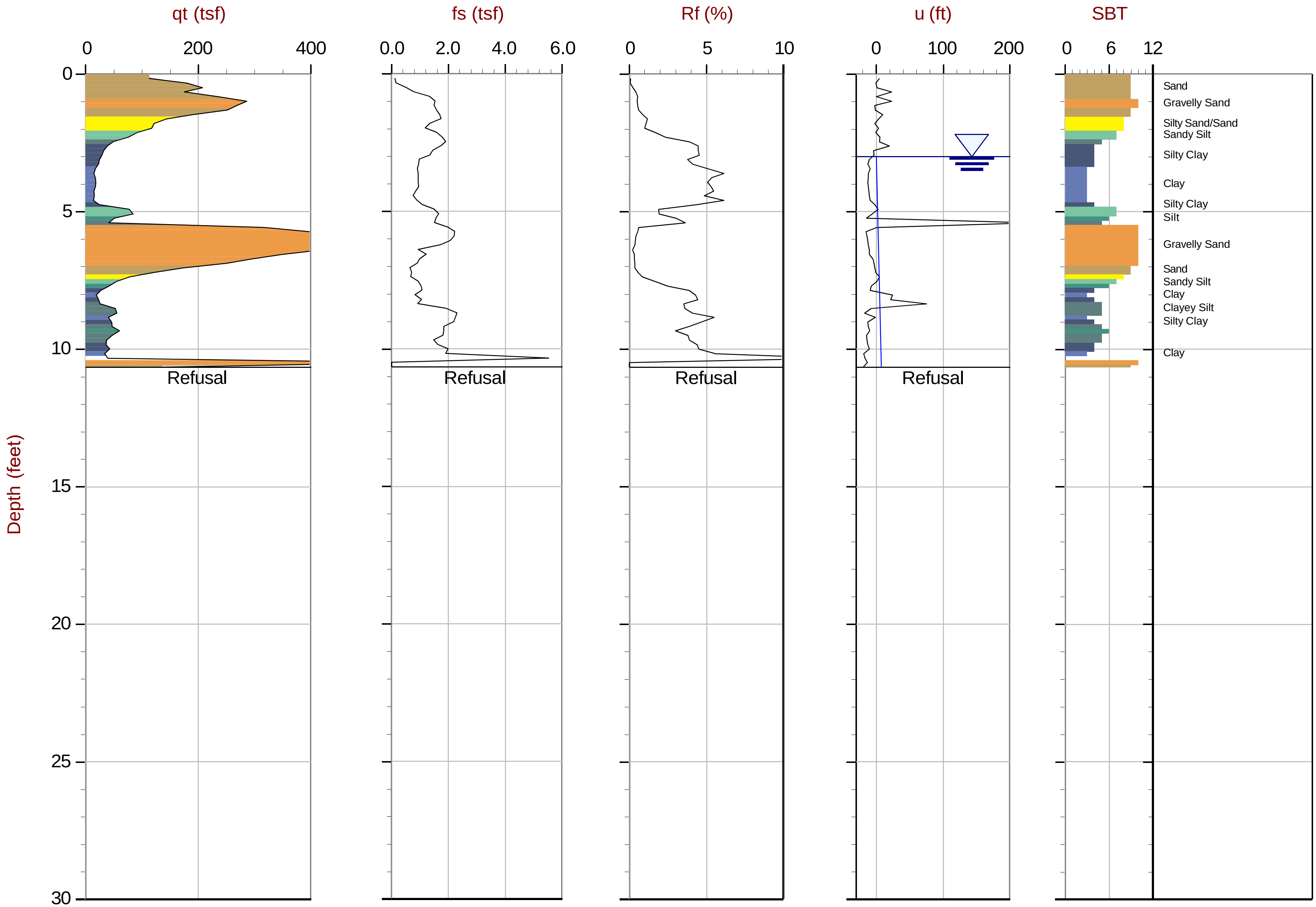
ATTACHMENT A – CPT LOGS

CONETEC

Geosyntec

Job No: 11-959
Date: 07:09:11 10:41
Site: East Dike

Sounding: CPT-01
Cone: 214:T1500F15U500



Max Depth: 3.250 m / 10.66 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: Every Point

File: 959CP01.COR
Unit Wt: SBT Chart Soil Zones

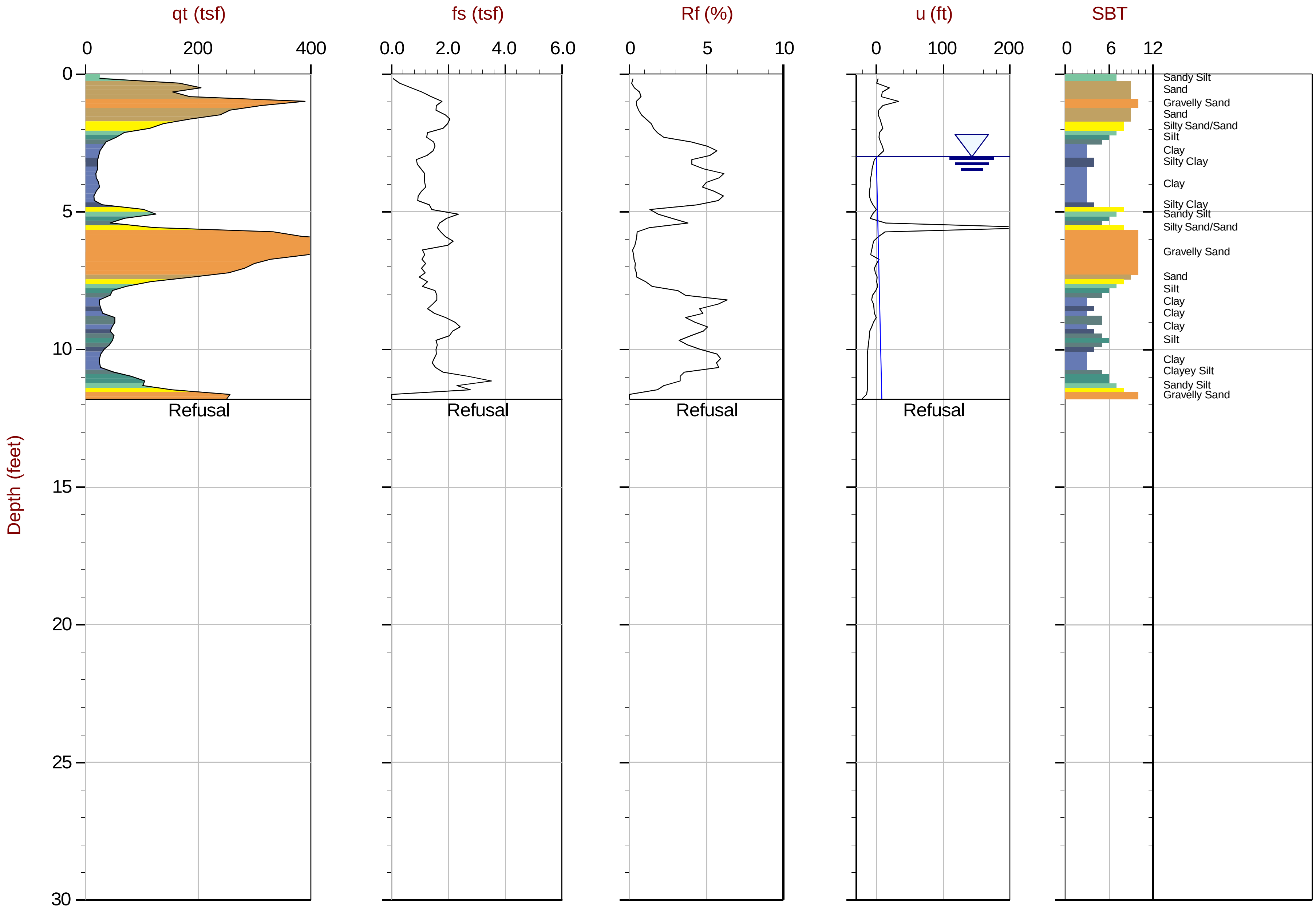
SBT: Lunne, Robertson and Powell, 1997
Page No: 1 of 1

ConeTec

Geosyntec

Job No: 11-959
Date: 07:09:11 12:09
Site: East Dike

Sounding: CPT-01A
Cone: 214:T1500F15U500



Max Depth: 3.600 m / 11.81 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: Every Point

File: 959CP01A.COR
Unit Wt: SBT Chart Soil Zones

SBT: Lunne, Robertson and Powell, 1997
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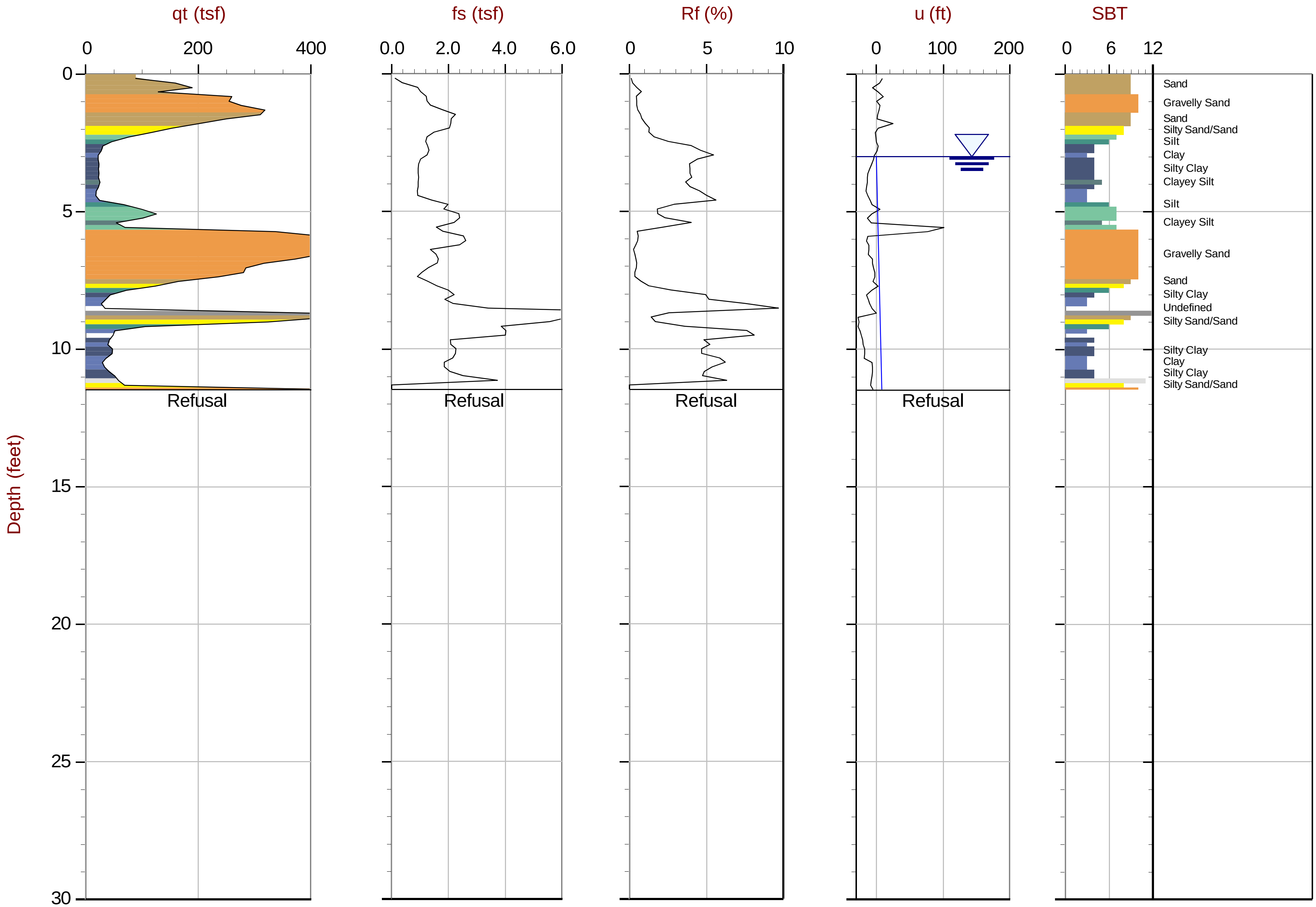
CONETEC

Geosyntec

Job No: 11-959
Date: 07:09:11 12:53
Site: East Dike

Sounding: CPT-01B
Cone: 214:T1500F15U500



Max Depth: 3.500 m / 11.48 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: Every Point

File: 959CP01B.COR
Unit Wt: SBT Chart Soil Zones

SBT: Lunne, Robertson and Powell, 1997
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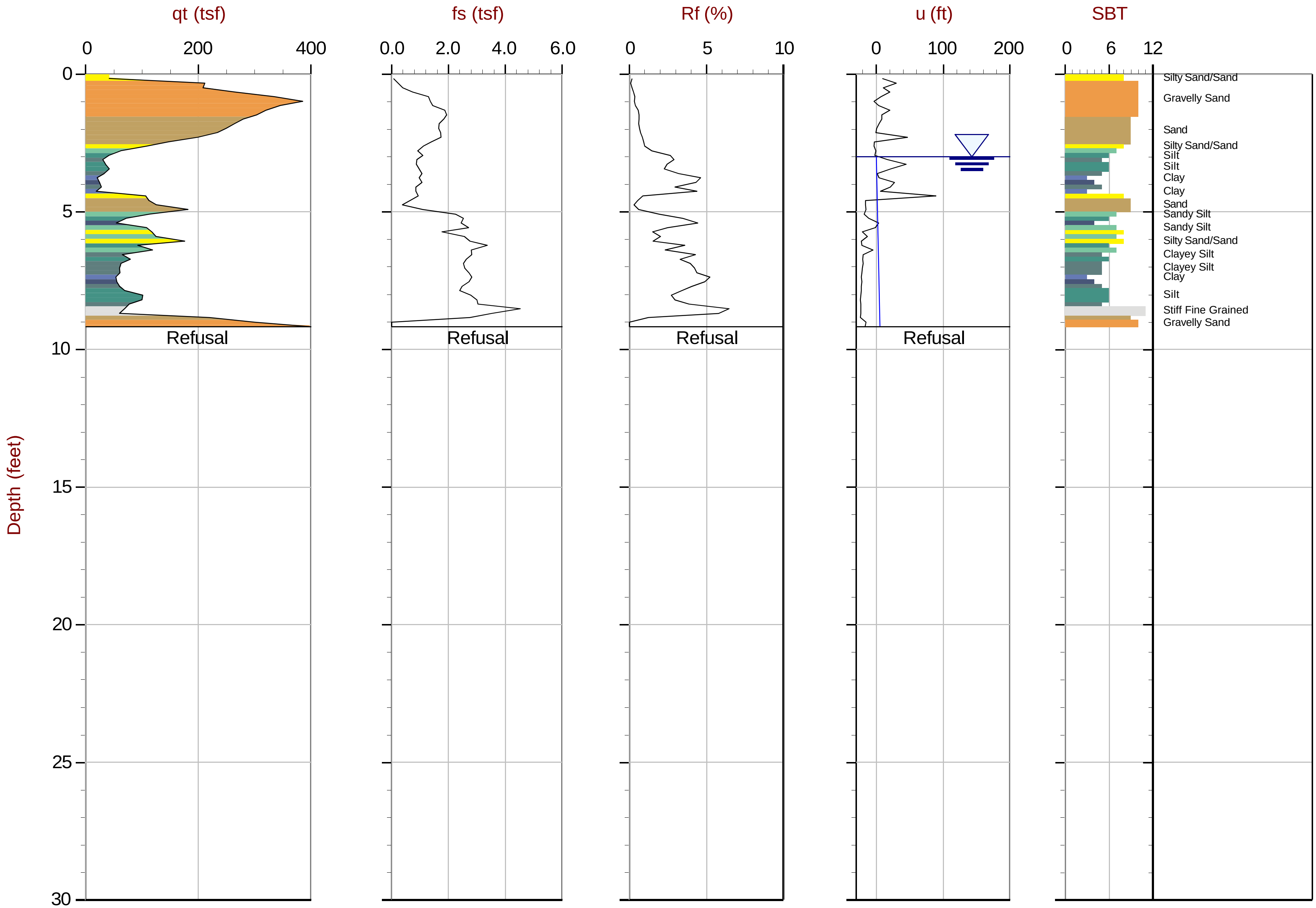
Geosyntec

Job No: 11-959
Date: 07:09:11 07:59
Site: East Dike

Sounding: CPT-02
Cone: 214:T1500F15U500



Max Depth: 2.800 m / 9.19 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: Every Point

File: 959CP02.COR
Unit Wt: SBT Chart Soil Zones

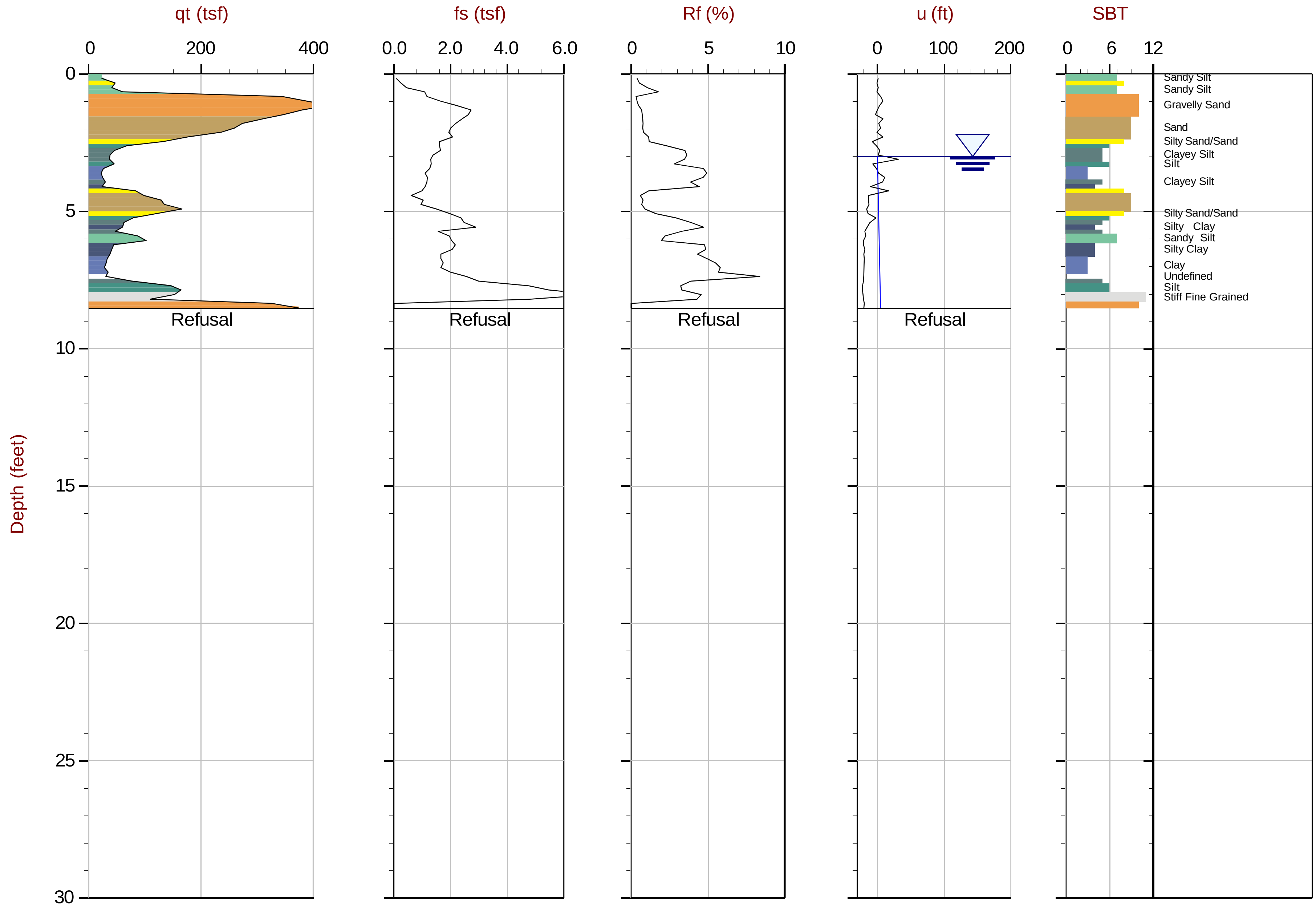
SBT: Lunne, Robertson and Powell, 1997

ConeTec

Geosyntec

Job No: 11-959
Date: 07:09:11 08:25
Site: East Dike

Sounding: CPT-02A
Cone: 214:T1500F15U500



Max Depth: 2.600 m / 8.53 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: Every Point

File: 959CP02A.COR
Unit Wt: SBT Chart Soil Zones

SBT: Lunne, Robertson and Powell, 1997
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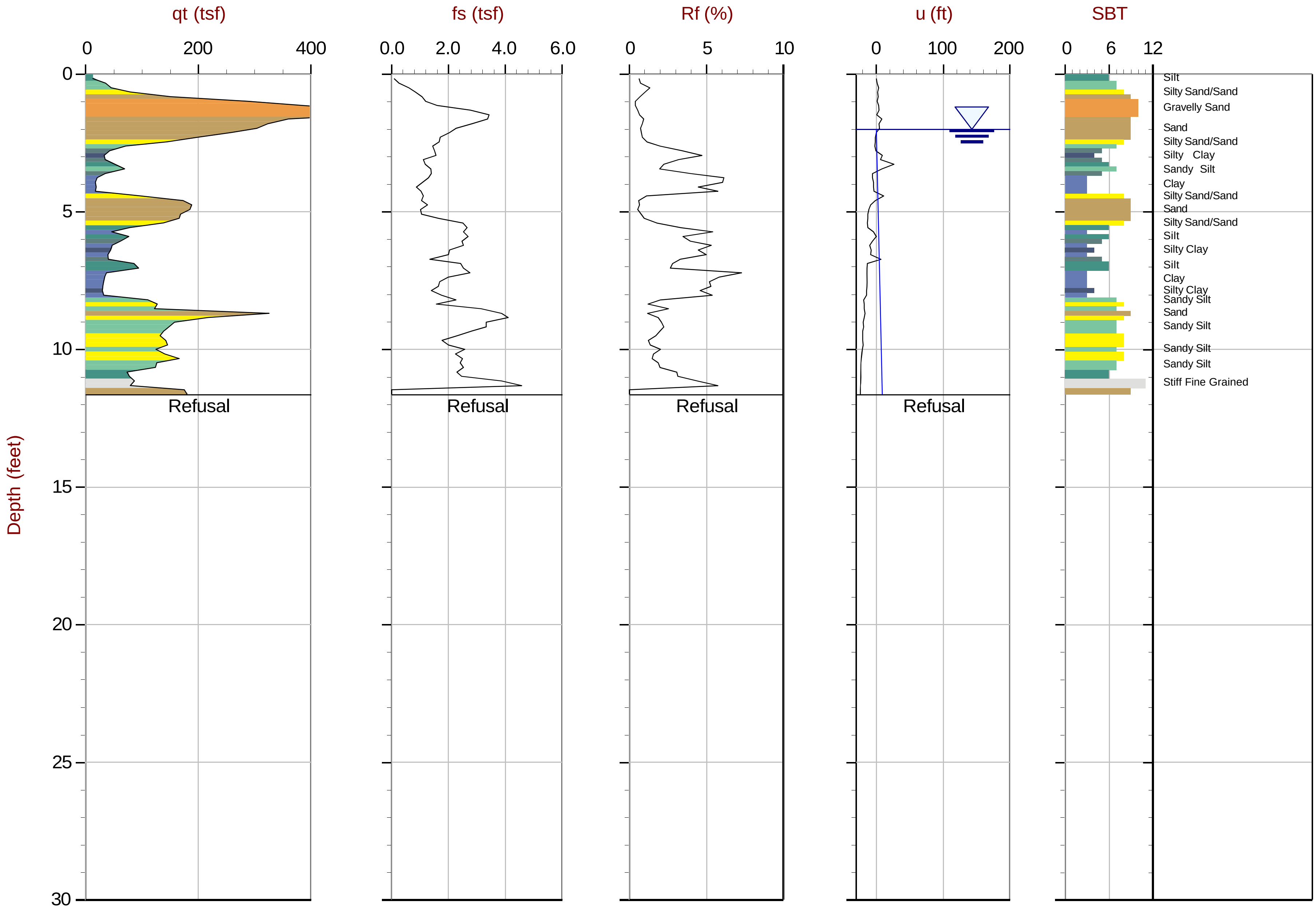
CONETEC

Geosyntec

Job No: 11-959
Date: 07:09:11 08:46
Site: East Dike

Sounding: CPT-02B
Cone: 214:T1500F15U500



Max Depth: 3.550 m / 11.65 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: Every Point

File: 959CP02B.COR
Unit Wt: SBT Chart Soil Zones

SBT: Lunne, Robertson and Powell, 1997
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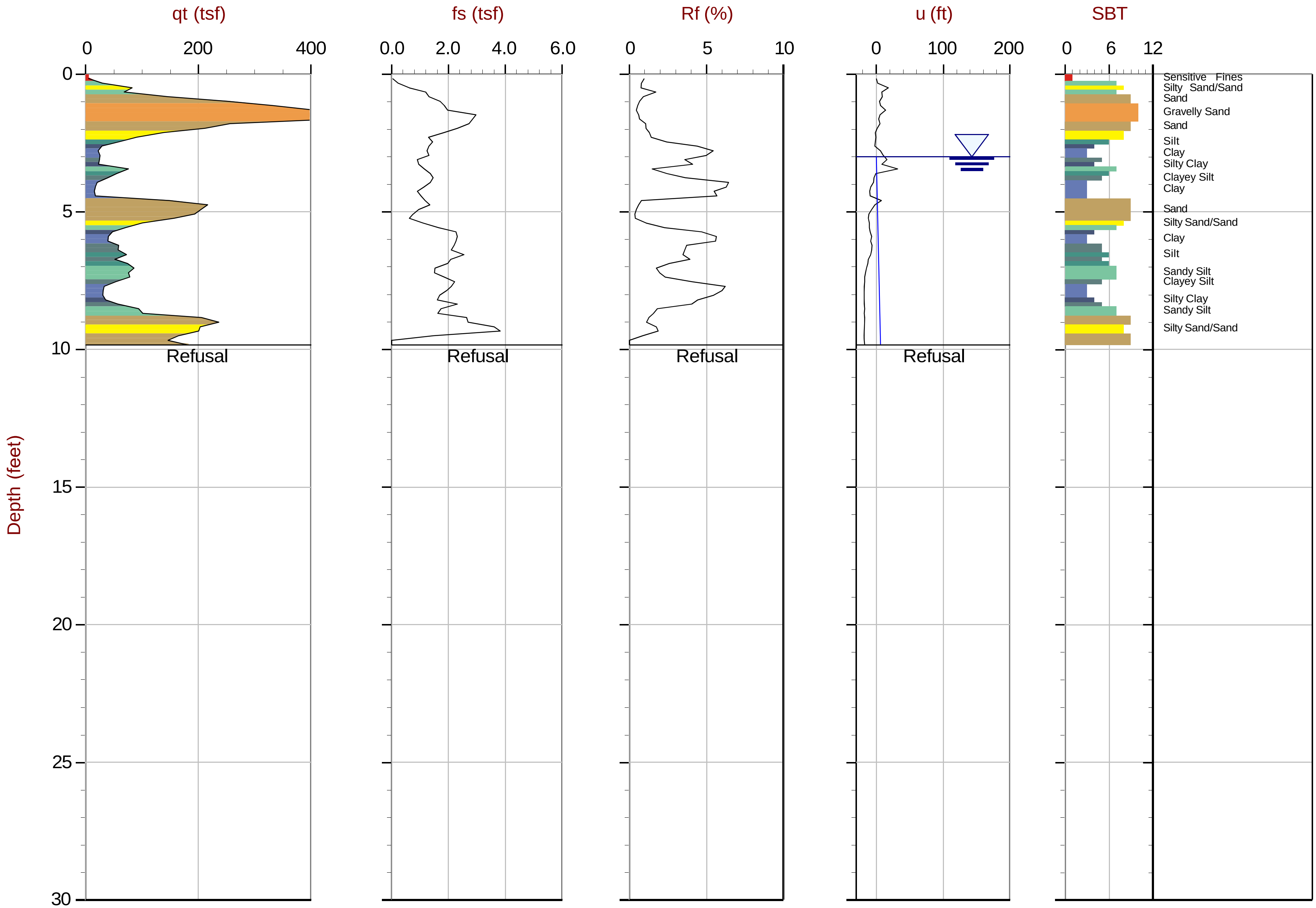
ConeTec

Geosyntec

Job No: 11-959
Date: 07:09:11 10:19
Site: East Dike

Sounding: CPT-02C
Cone: 214:T1500F15U500



Max Depth: 3.000 m / 9.84 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: Every Point

File: 959CP02C.COR
Unit Wt: SBT Chart Soil Zones

SBT: Lunne, Robertson and Powell, 1997
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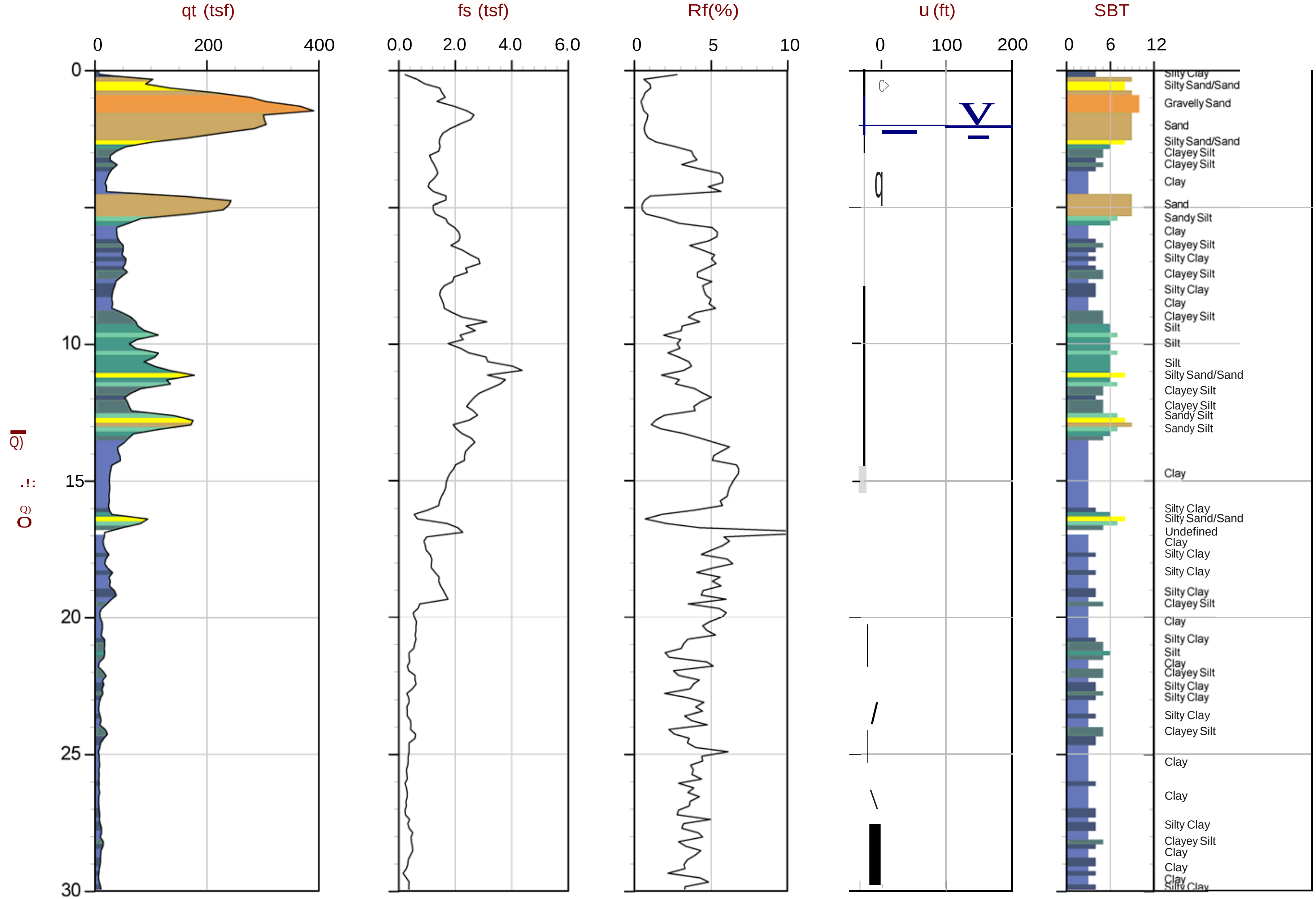
CONETEC

Geosyntec

Job No: 11-959
Date: 07:09:11 10:41
Site: East Dike

Sounding: CPT-02D
Cone: 214:T1500F15U500



Max Depth: 15.500 m / 50.85 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: Every Point

File: 959CP02D.COR
Unit Wt: SBT Chart Soil Zones

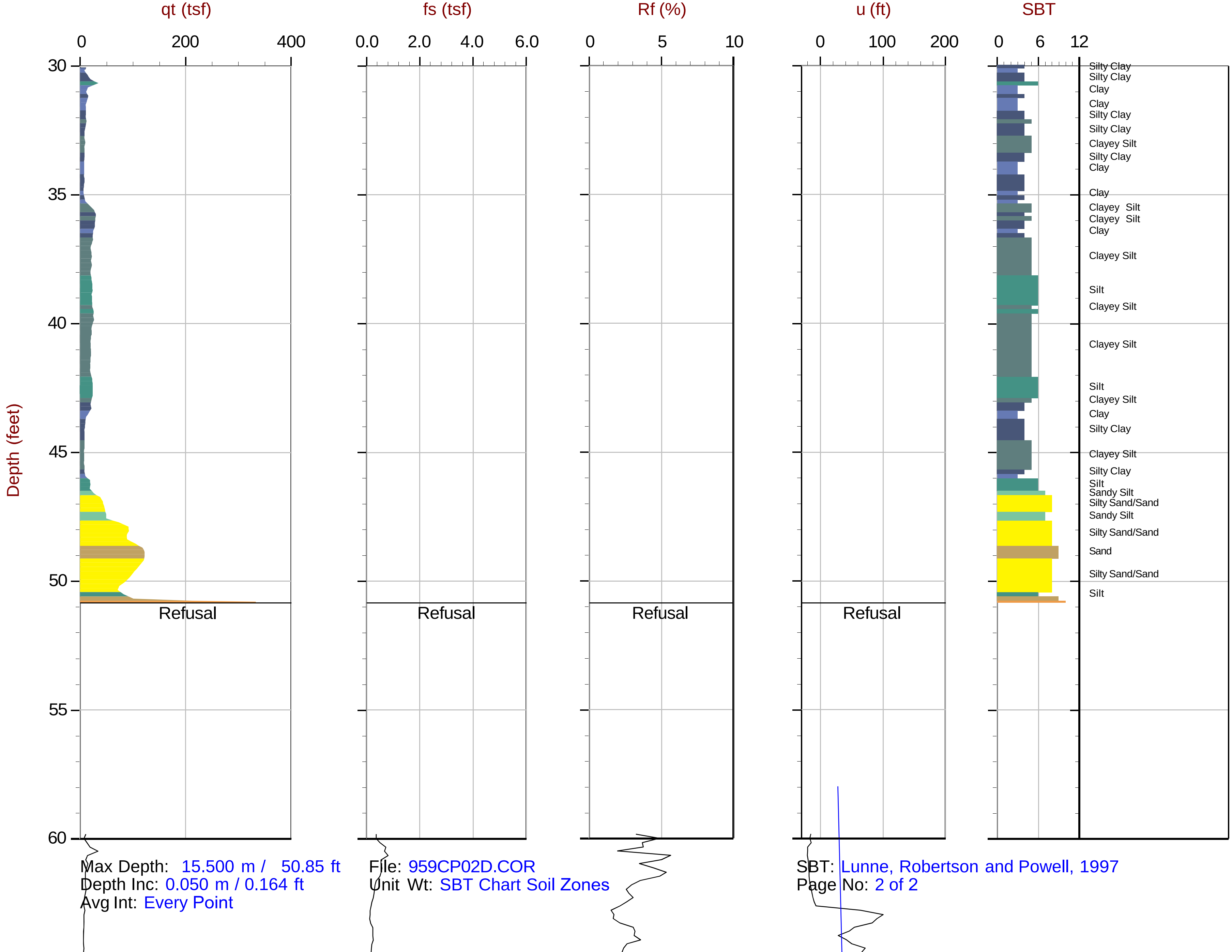
SBT: Lunne, Robertson and Powell, 1997
Page No: 1 of 2

ConeTec

Geosyntec

Job No: 11-959
Date: 07:09:11 10:41
Site: East Dike

Sounding: CPT-02D
Cone: 214:T1500F15U500

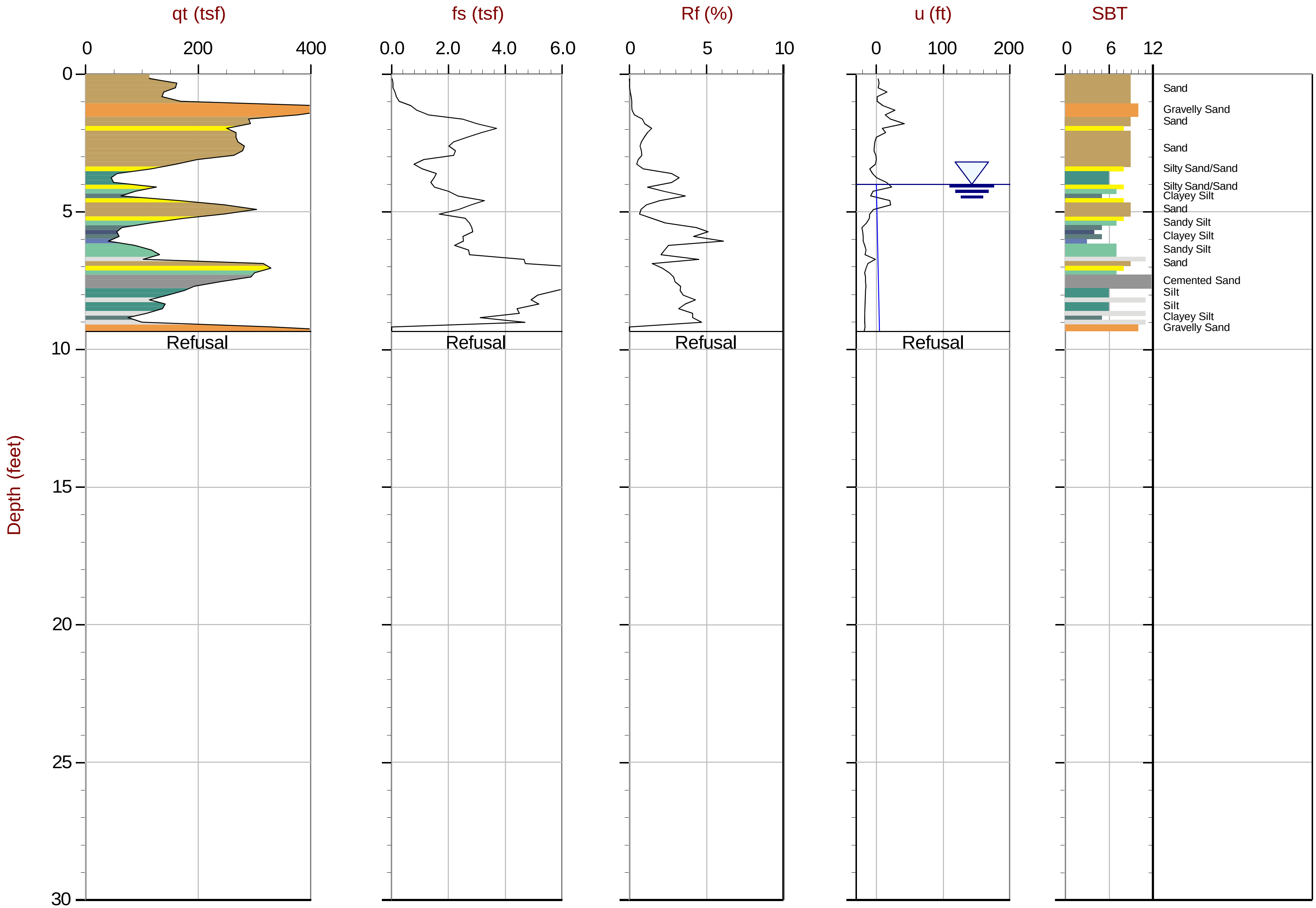


Max Depth: 15.500 m / 50.85 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: Every Point

File: 959CP02D.COR
Unit Wt: SBT Chart Soil Zones

SBT: Lunne, Robertson and Powell, 1997
Page No: 2 of 2

Geosyntec	Job No: 11-959 Date: 07:08:11 12:30 Site: East Dike	Sounding: CPT-03 Cone: 184:T1500F15U500



Max Depth: 2.850 m / 9.35 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: Every Point

File: 959CP03.COR
Unit Wt: SBT Chart Soil Zones

SBT: Lunne, Robertson and Powell, 1997
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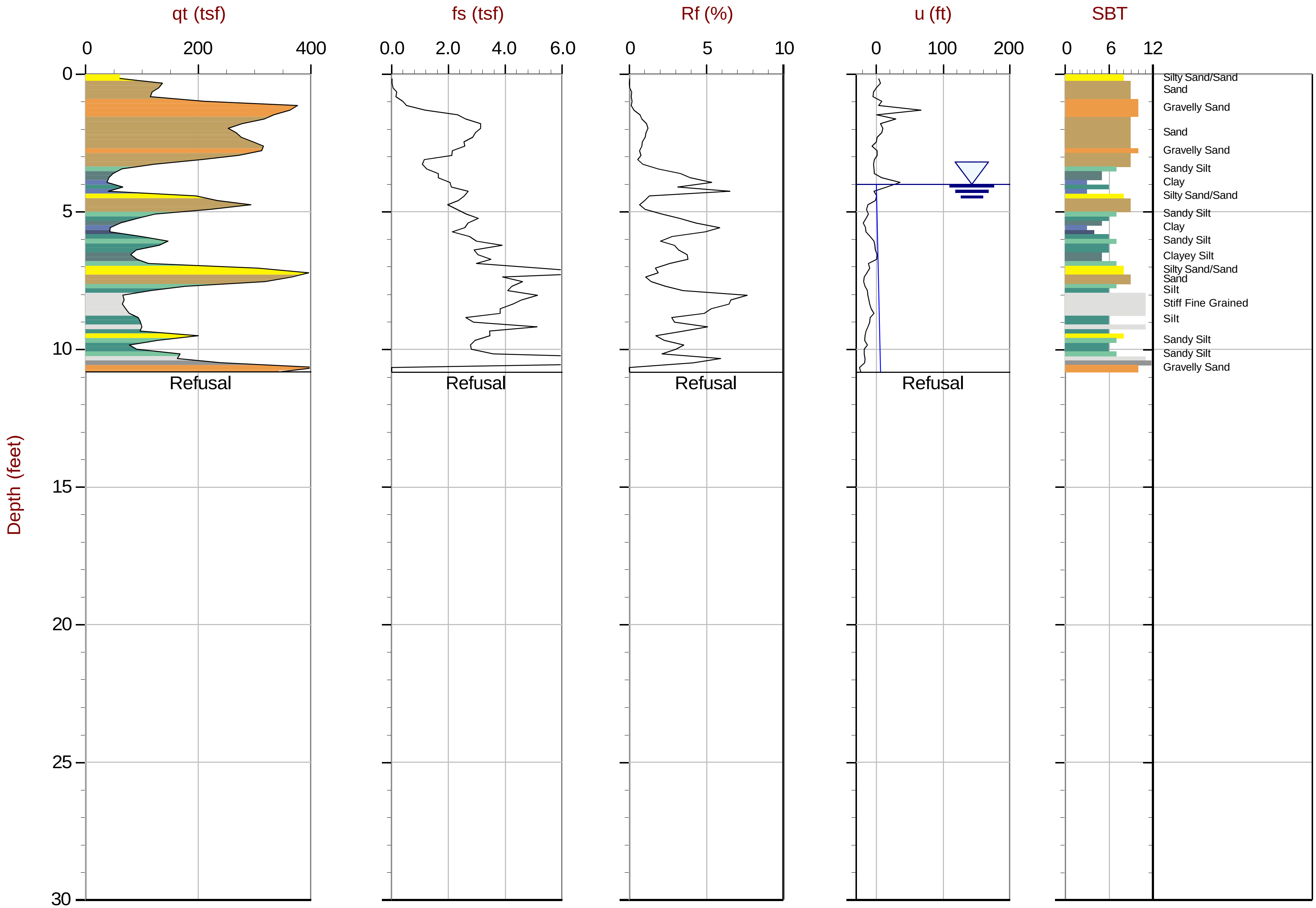
ConeTec

Geosyntec

Job No: 11-959
Date: 07:08:11 13:57
Site: East Dike

Sounding: CPT-03A
Cone: 184:T1500F15U500



Max Depth: 3.300 m / 10.83 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: Every Point

File: 959CP03A.COR
Unit Wt: SBT Chart Soil Zones

SBT: Lunne, Robertson and Powell, 1997
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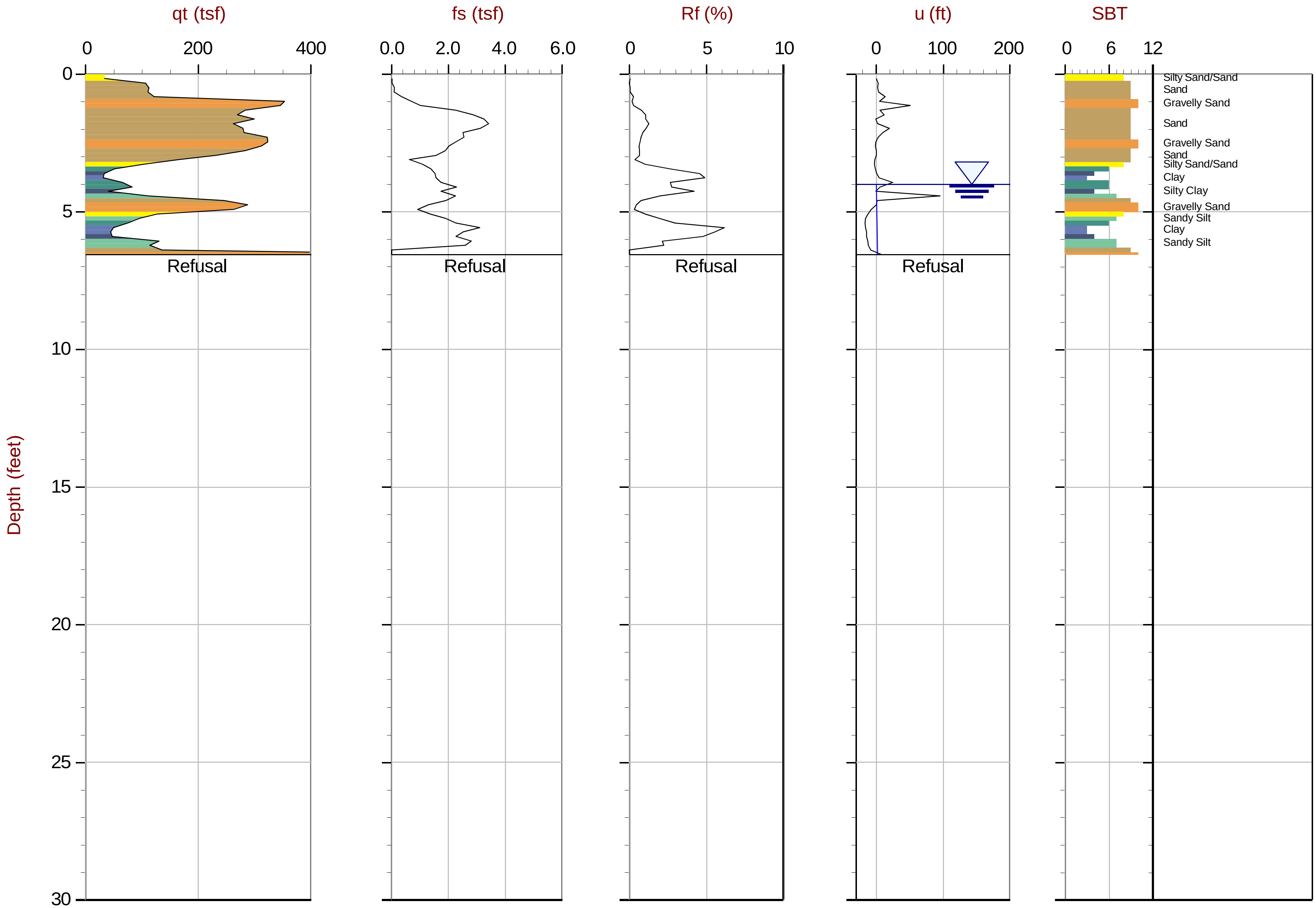
Geosyntec	Job No: 11-959	Sounding: CPT-03B
	Date: 07:08:11 15:02	Cone: 214:T1500F15U500
	Site: East Dike	



Max Depth: 2.000 m / 6.56 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: Every Point

File: 959CP03B.COR
Unit Wt: SBT Chart Soil Zones

SBT: Lunne, Robertson and Powell, 1997
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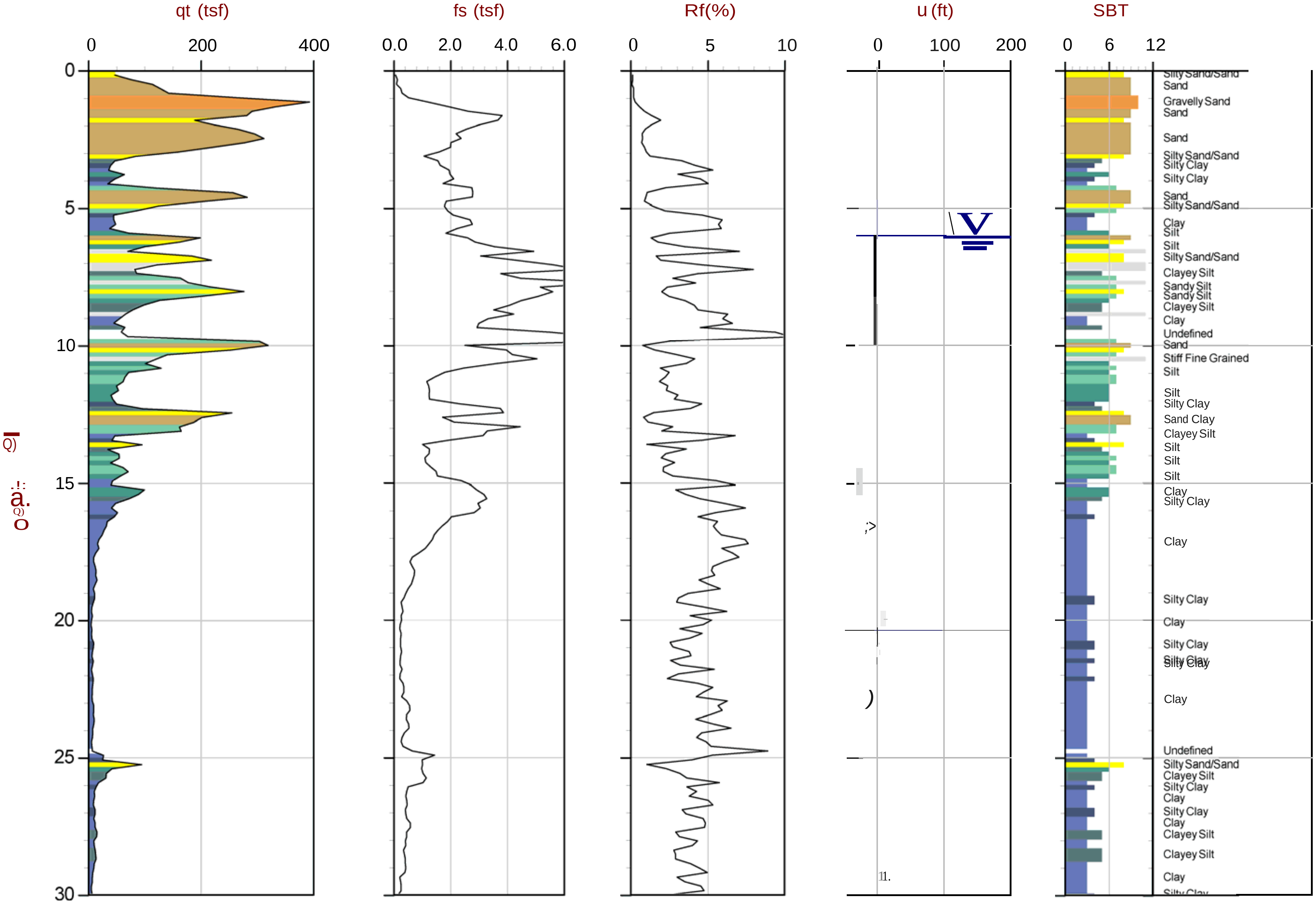
CONETEC

Geosyntec

Job No: 11-959
Date: 07:08:11 15:18
Site: East Dike

Sounding: CPT-03C
Cone: 214:T1500F15U500



Max Depth: 15.300 m / 50.20 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: Every Point

File: 959CP03C.COR
Unit Wt: SBT Chart Soil Zones

SBT: Lunne, Robertson and Powell, 1997
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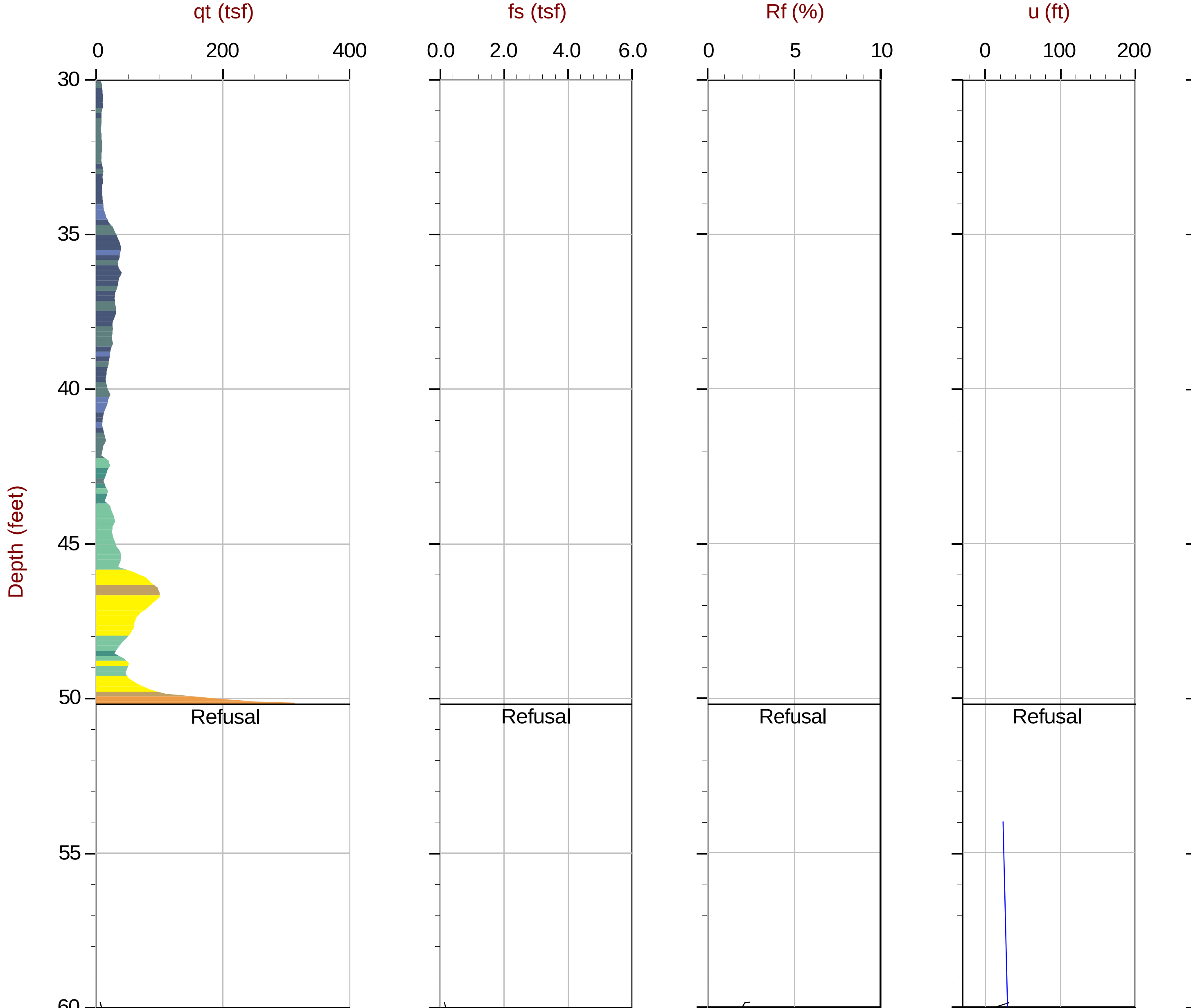
Geosyntec

Job No: 11-959
Date: 07:08:11 15:18
Site: East Dike

Sounding: CP
Cone: 214:T15



Max Depth: 15.300 m / 50.20 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: Every Point

File: 959CP03C.COR
Unit Wt: SBT Chart Soil Zones

SBT: Lunne, Robertson a
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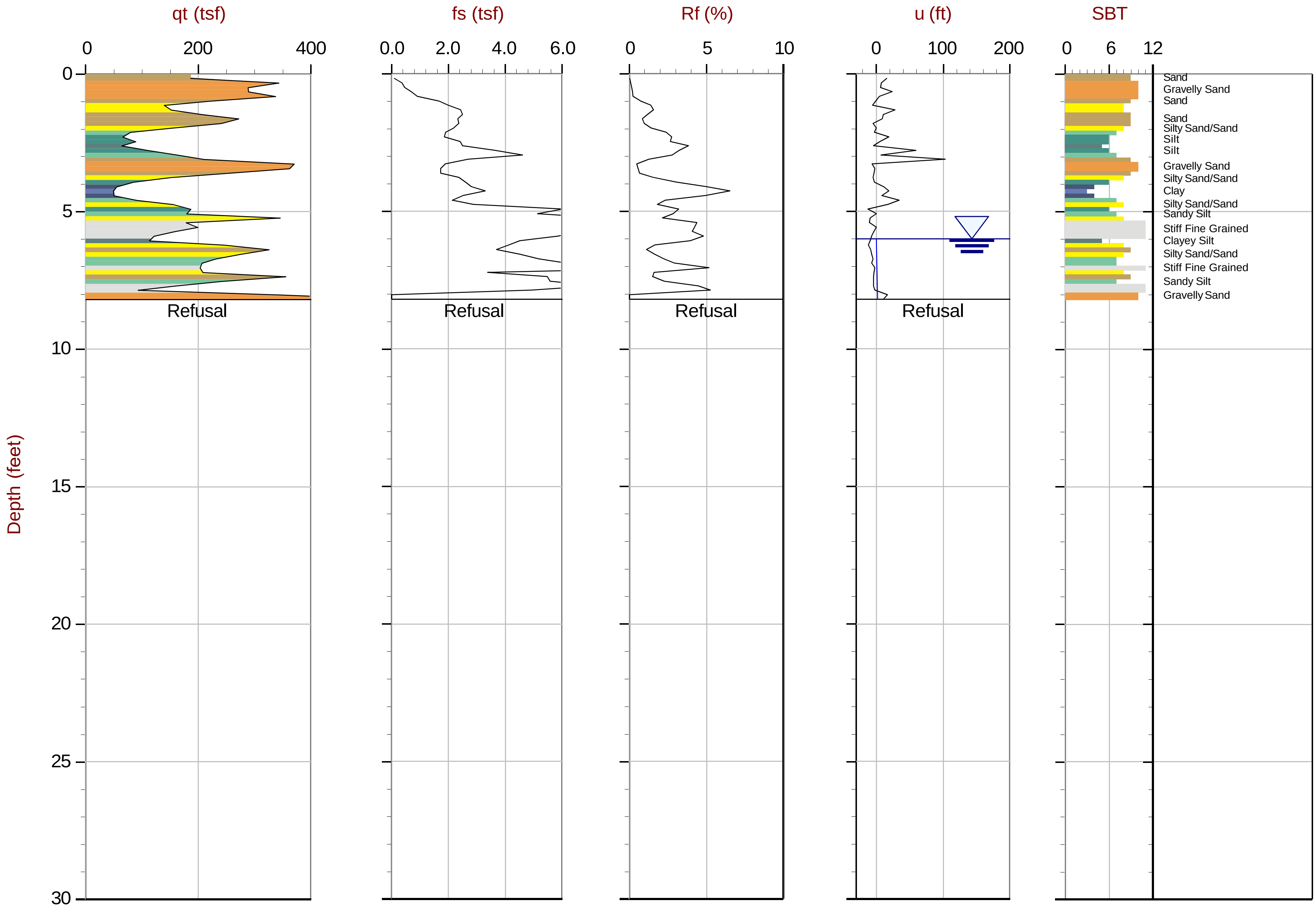
ConeTec

Geosyntec

Job No: 11-959
Date: 07:08:11 12:04
Site: East Dike

Sounding: CPT-04
Cone: 184:T1500F15U500



Max Depth: 2.500 m / 8.20 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: Every Point

File: 959CP04.COR
Unit Wt: SBT Chart Soil Zones

SBT: Lunne, Robertson and Powell, 1997
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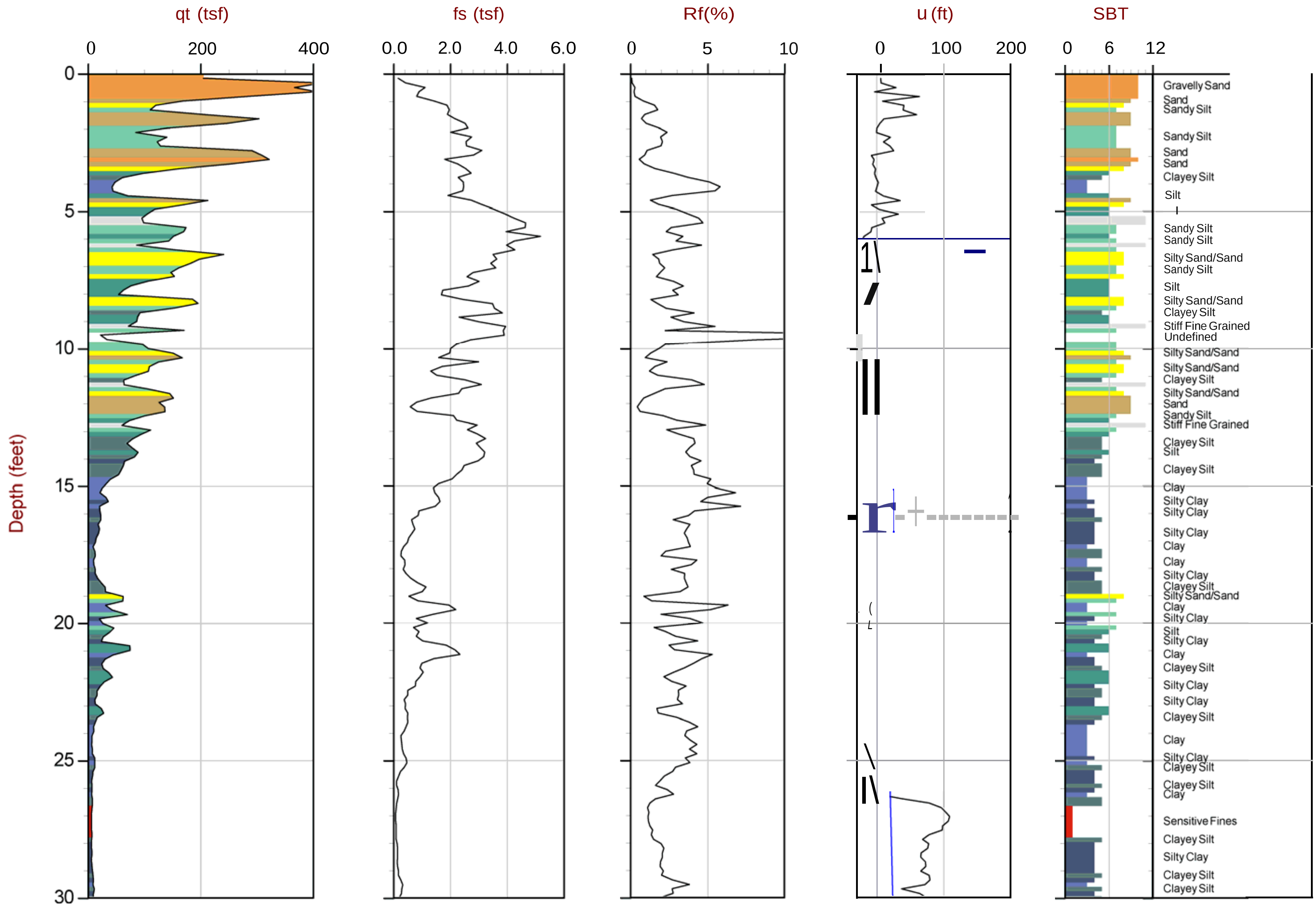
CONETEC **Geosyntec**

Job No: 11-959
Date: 07:08:11 12:30
Site: East Dike

Sounding: CPT-04A
Cone: 184:T1500F15U500



Max Depth: 13.400 m / 43.96 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: Every Point

File: 959CP04A.COR
Unit Wt: SBT Chart Soil Zones

SBT: Lunne, Robertson and Powell, 1997
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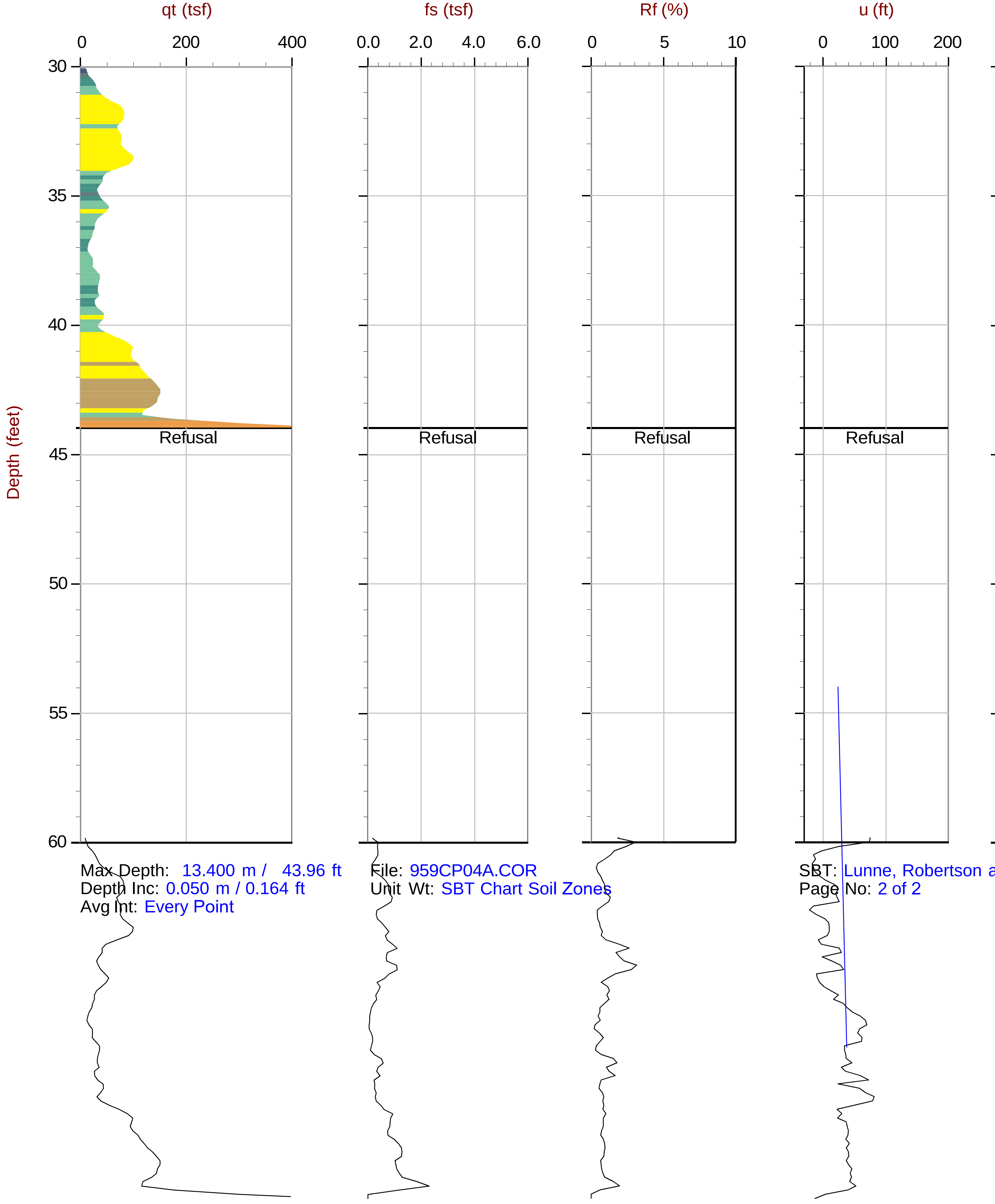


Max Depth: 13.400 m / 43.96 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: Every Point

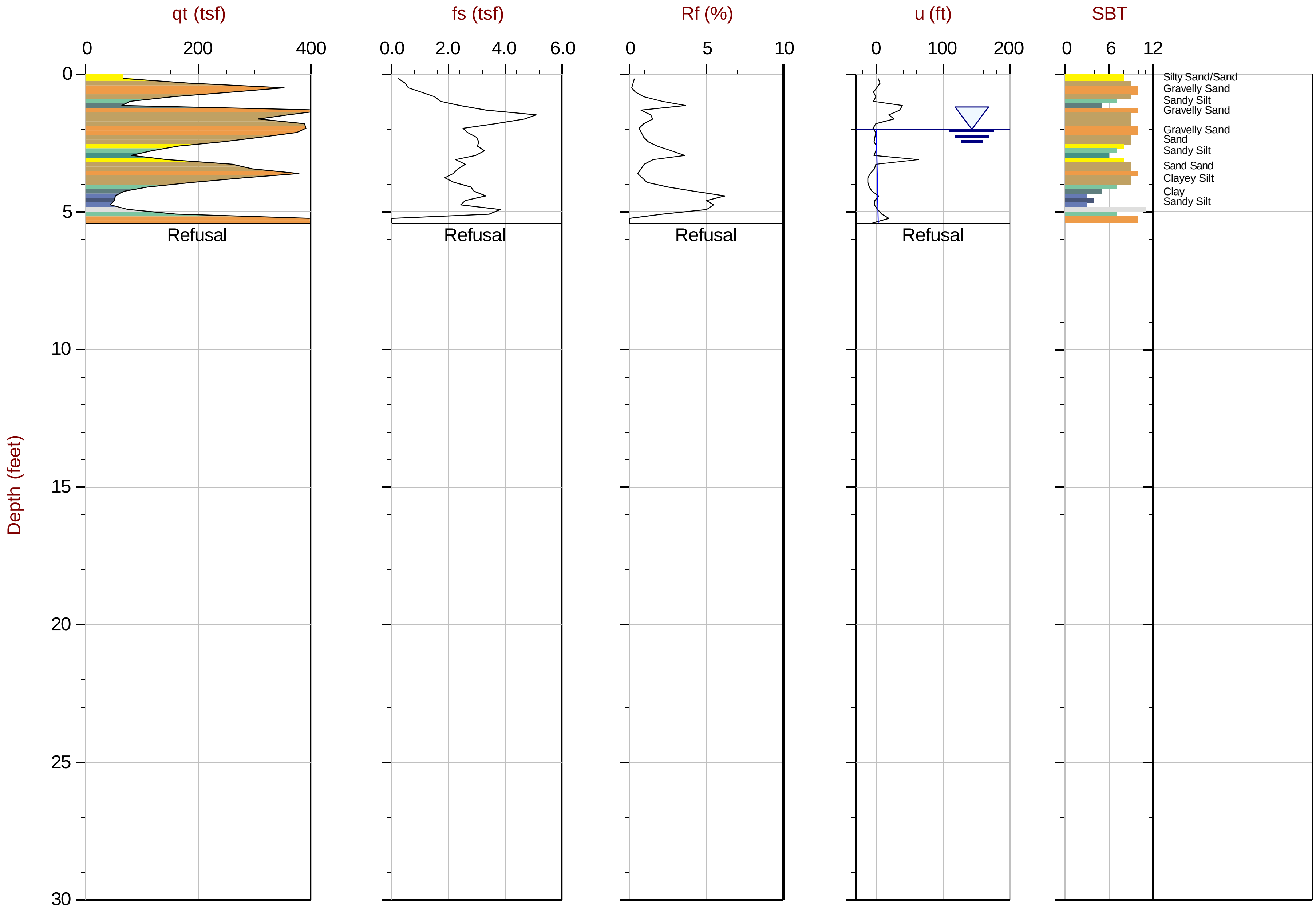
File: 959CP04A.COR
Unit Wt: SBT Chart Soil Zones

SBT: Lunne, Robertson a

Geosyntec

Job No: 11-959
Date: 07:07:11 14:50
Site: East Dike

Sounding: CPT-05
Cone: 184:T1500F15U500



Max Depth: 1.650 m / 5.41 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: Every Point

File: 959CP05.COR
Unit Wt: SBT Chart Soil Zones

SBT: Lunne, Robertson and Powell, 1997
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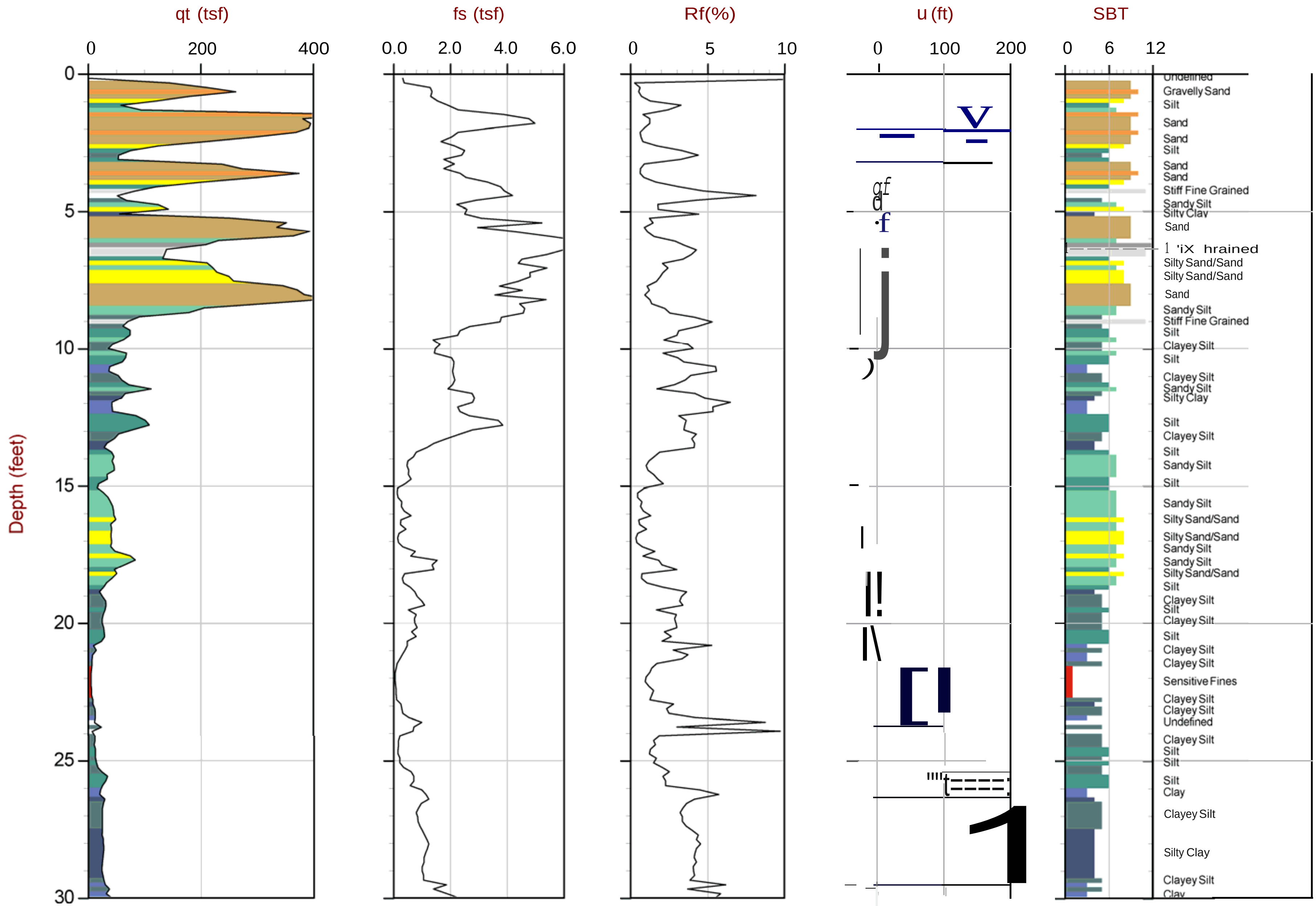
CONETEC

Geosyntec

Job No: 11-959
Date: 07:07:11 15:12
Site: East Dike

Sounding: CPT-05A
Cone: 184:T1500F15U500



Max Depth: 14.250 m / 46.75 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: Every Point

File: 959CP05A.COR
Unit Wt: SBT Chart Soil Zones

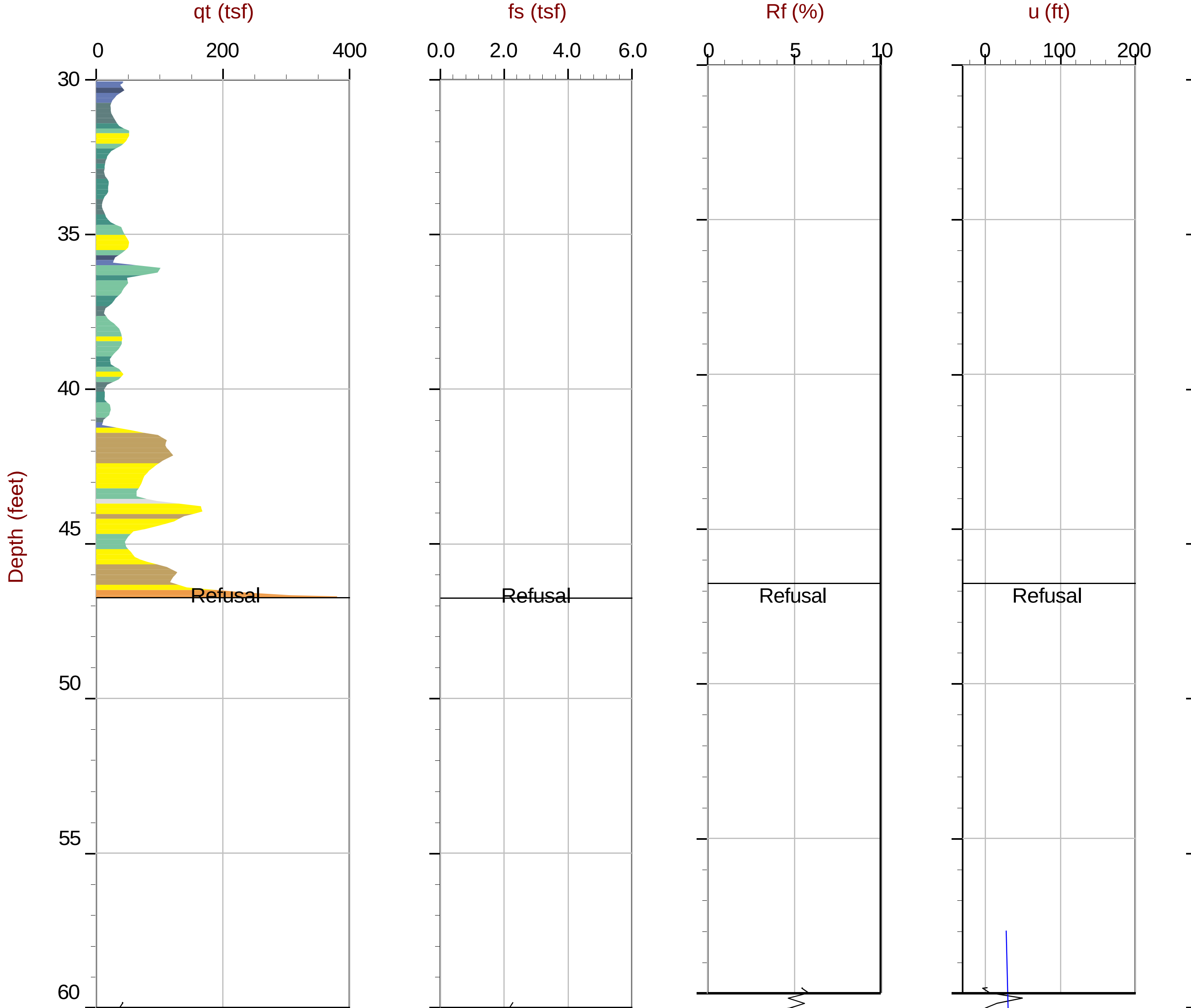
SBT: Lunne, Robertson and Powell, 1997
Page No: 1 of 2

ConeTec

Geosyntec

Job No: 11-959
Date: 07:07:11 15:12
Site: East Dike

Sounding: CP
Cone: 184:T15



Max Depth: 14.250 m / 46.75 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: Every Point

File: 959CP05A.COR
Unit Wt: SBT Chart Soil Zones

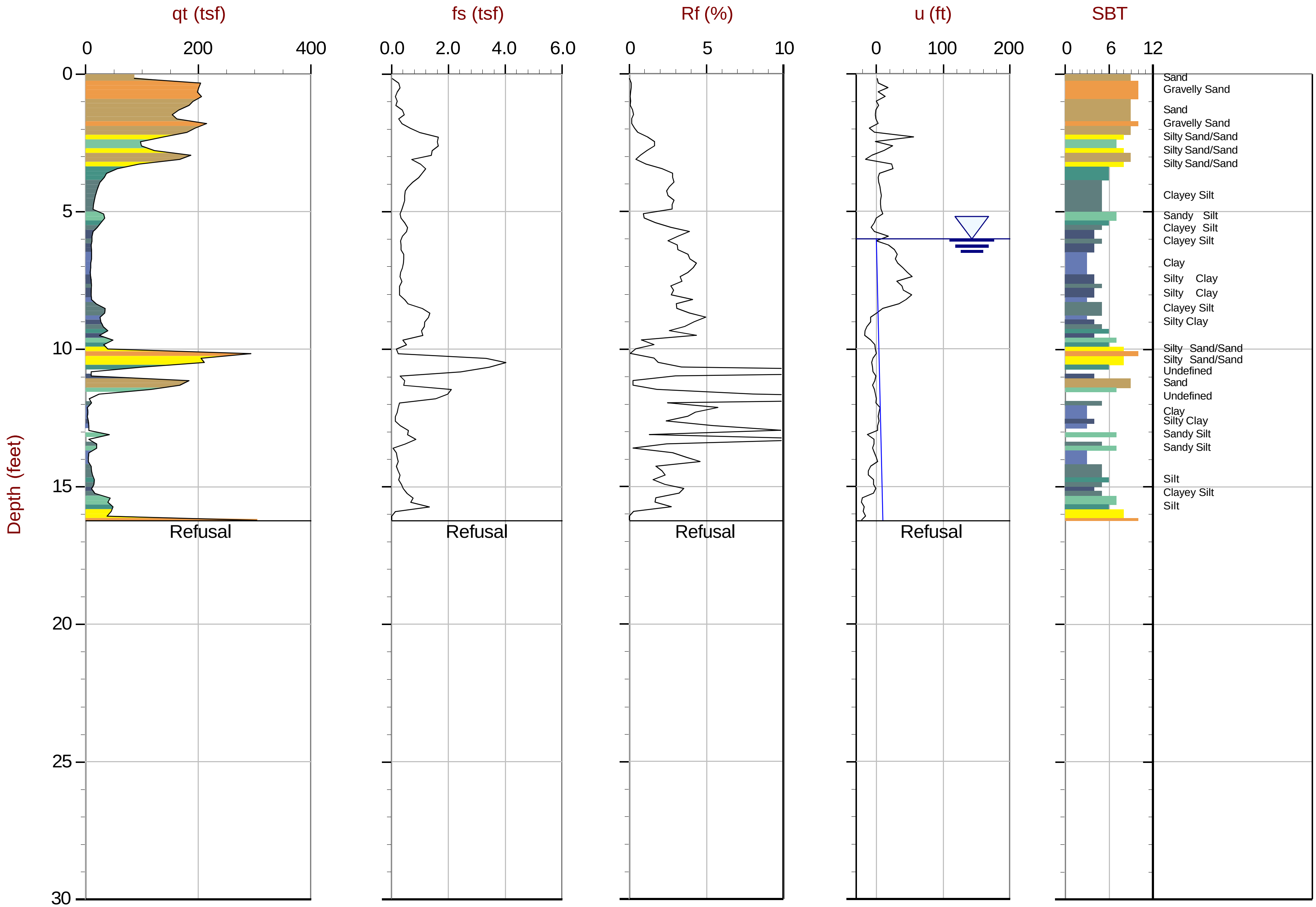
SBT: Lunne, Robertson a

CONETEC

Geosyntec

Job No: 11-959
Date: 07:08:11 08:07
Site: East Dike

Sounding: CPT-07
Cone: 184:T1500F15U500



Max Depth: 4.950 m / 16.24 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: Every Point

File: 959CP07.COR
Unit Wt: SBT Chart Soil Zones

SBT: Lunne, Robertson and Powell, 1997
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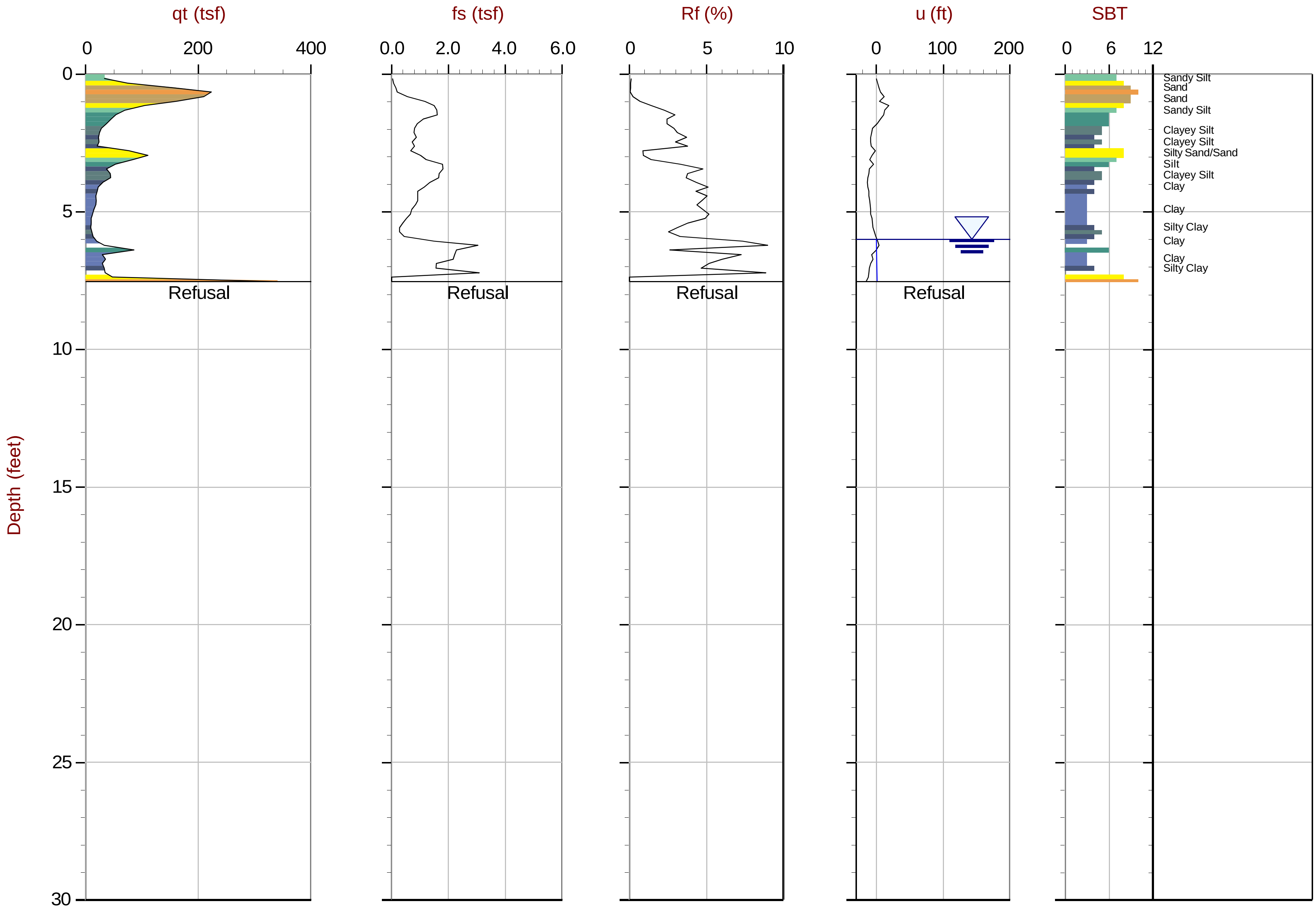
ConeTec

Geosyntec

Job No: 11-959
Date: 07:08:11 11:07
Site: East Dike

Sounding: CPT-07A
Cone: 184:T1500F15U500



Max Depth: 2.300 m / 7.55 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: Every Point

File: 959CP07A.COR
Unit Wt: SBT Chart Soil Zones

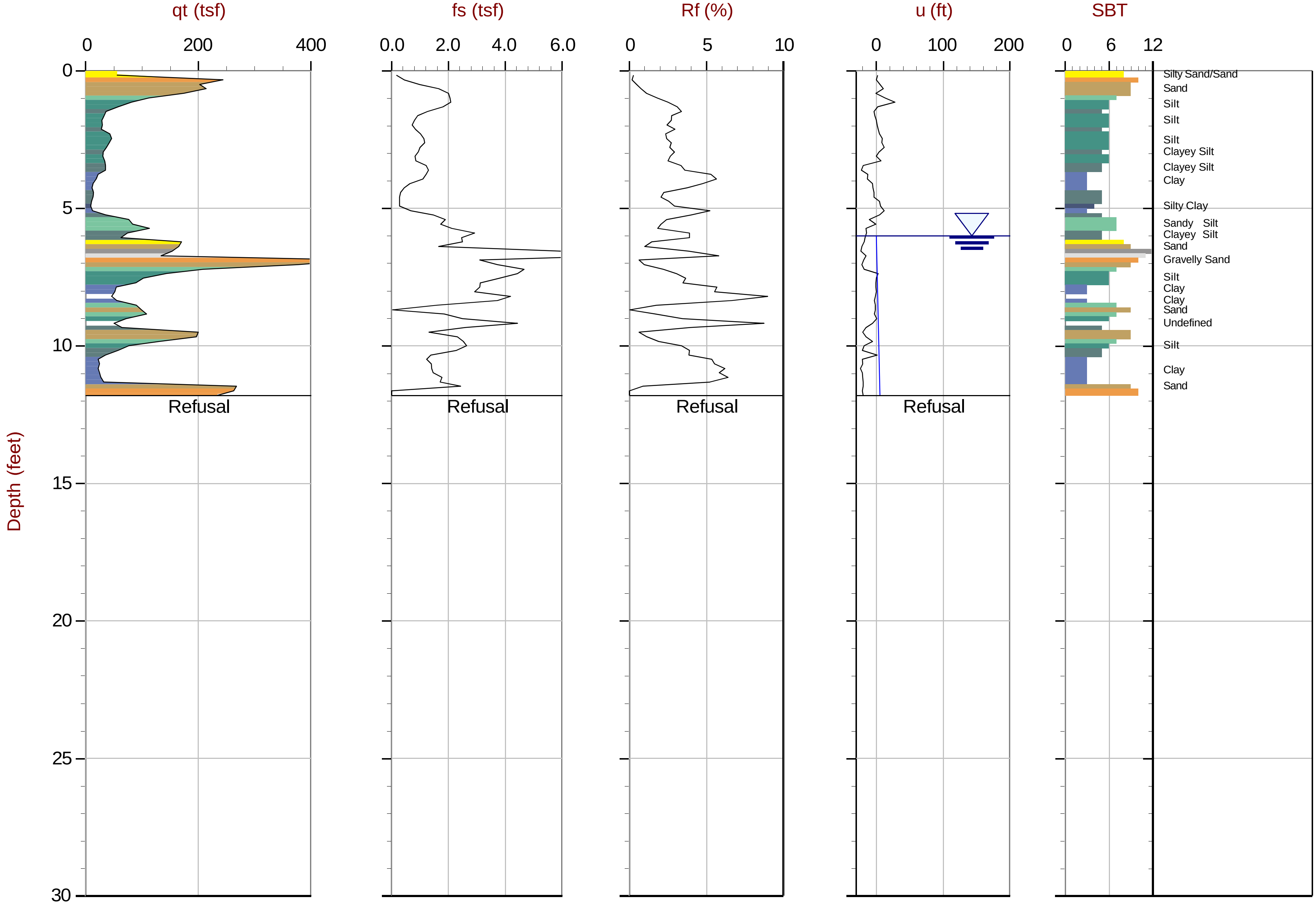
SBT: Lunne, Robertson and Powell, 1997
Page No: 1 of 1

CONETEC

Geosyntec

Job No: 11-959
Date: 07:08:11 11:07
Site: East Dike

Sounding: CPT-07B
Cone: 184:T1500F15U500



Max Depth: 3.600 m / 11.81 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: Every Point

File: 959CP07B.COR
Unit Wt: SBT Chart Soil Zones

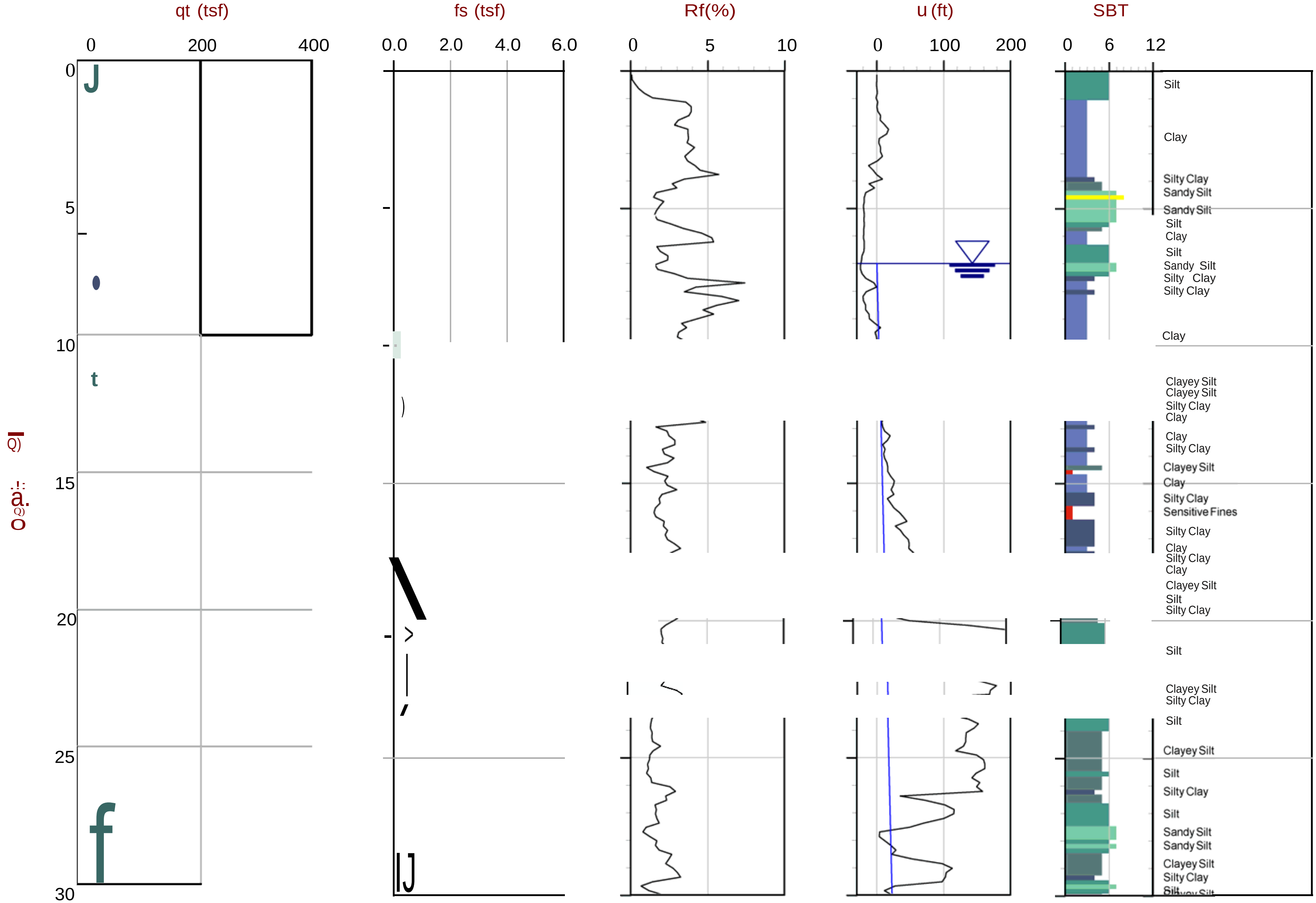
SBT: Lunne, Robertson and Powell, 1997
Page No: 1 of 1

CONETEC

Geosyntec

Job No: 11-959
Date: 07:07:11 12:45
Site: East Dike

Sounding: CPT-08
Cone: 184:T1500F15U500



Max Depth: 13.750 m / 45.11 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: Every Point

File: 959CP08.COR
Unit Wt: SBT Chart Soil Zones

SBT: Lunne, Robertson and Powell, 1997
Page No: 1 of 2

CONETEC

Geosyntec

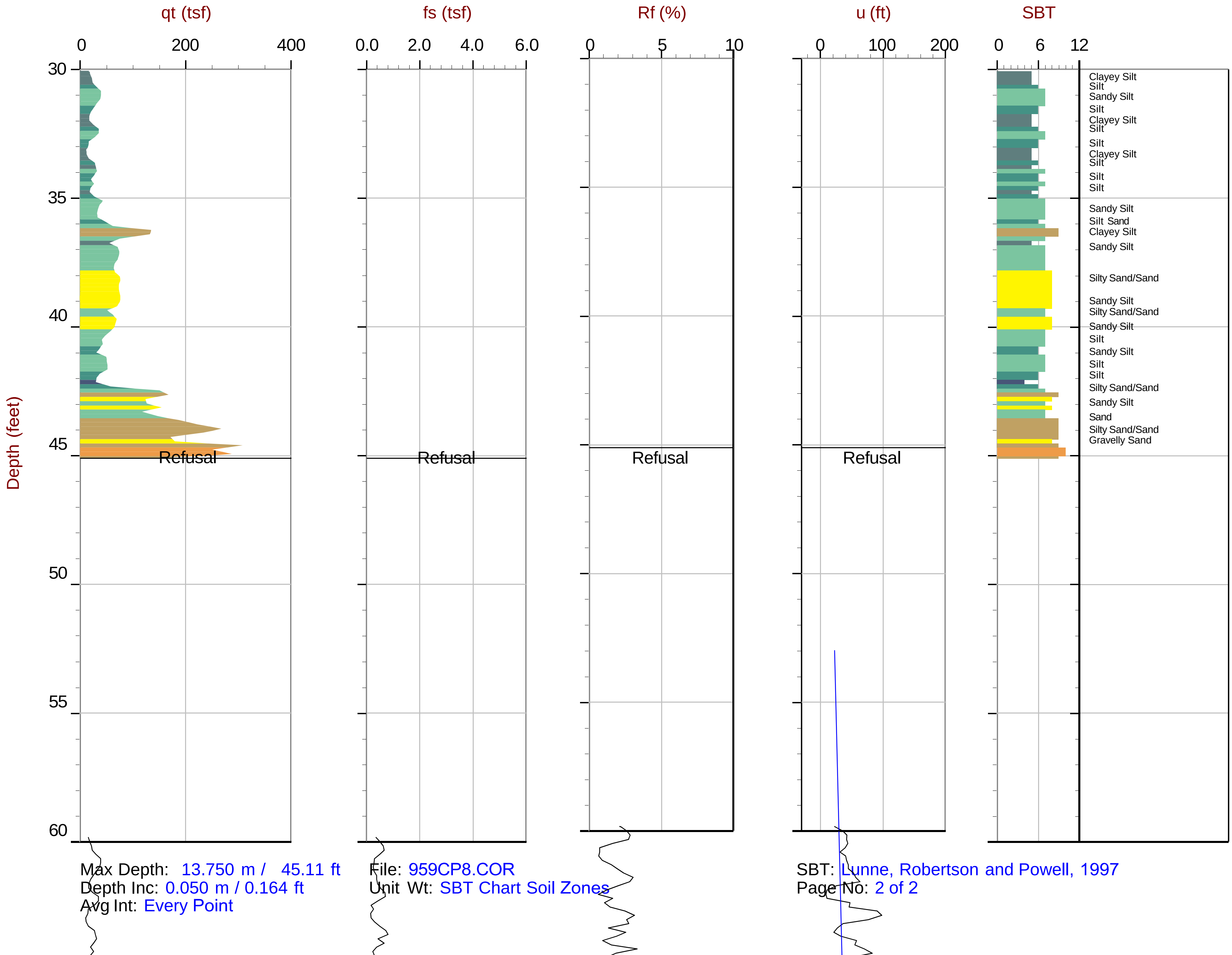
Job No: 11-959

Date: 07:07:11 12:45

Site: East Dike

Sounding: CPT-08

Cone: 184:T1500F15U500



Max Depth: 13.750 m / 45.11 ft

Depth Inc: 0.050 m / 0.164 ft

Avg Int: Every Point

File: 959CP8.COR

Unit Wt: SBT Chart Soil Zones

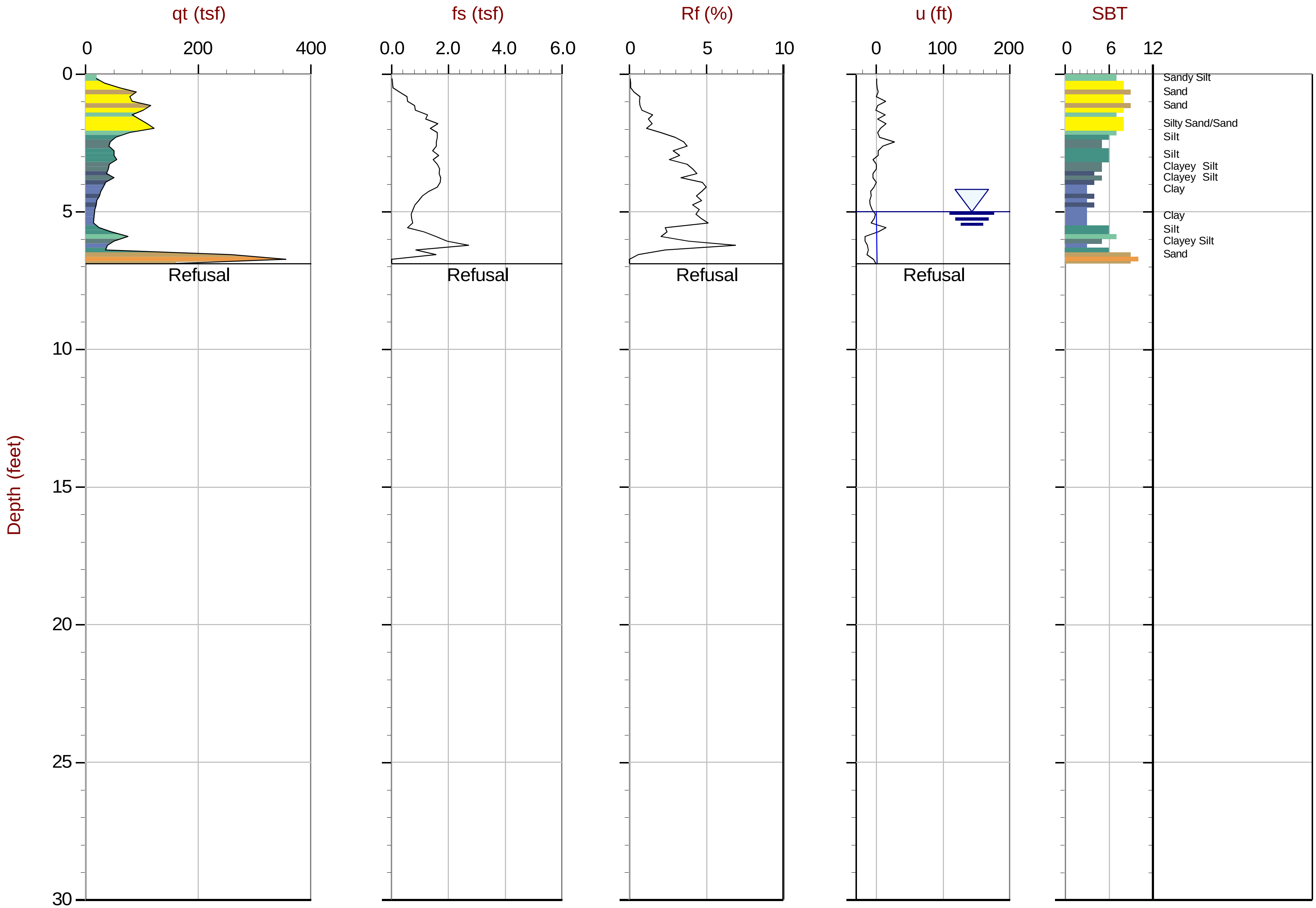
SBT: Lunne, Robertson and Powell, 1997

ConeTec

Geosyntec

Job No: 11-959
Date: 07:07:11 09:57
Site: East Dike

Sounding: CPT-09
Cone: 184:T1500F15U500



Max Depth: 2.100 m / 6.89 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: Every Point

File: 959CP9.COR
Unit Wt: SBT Chart Soil Zones

SBT: Lunne, Robertson and Powell, 1997
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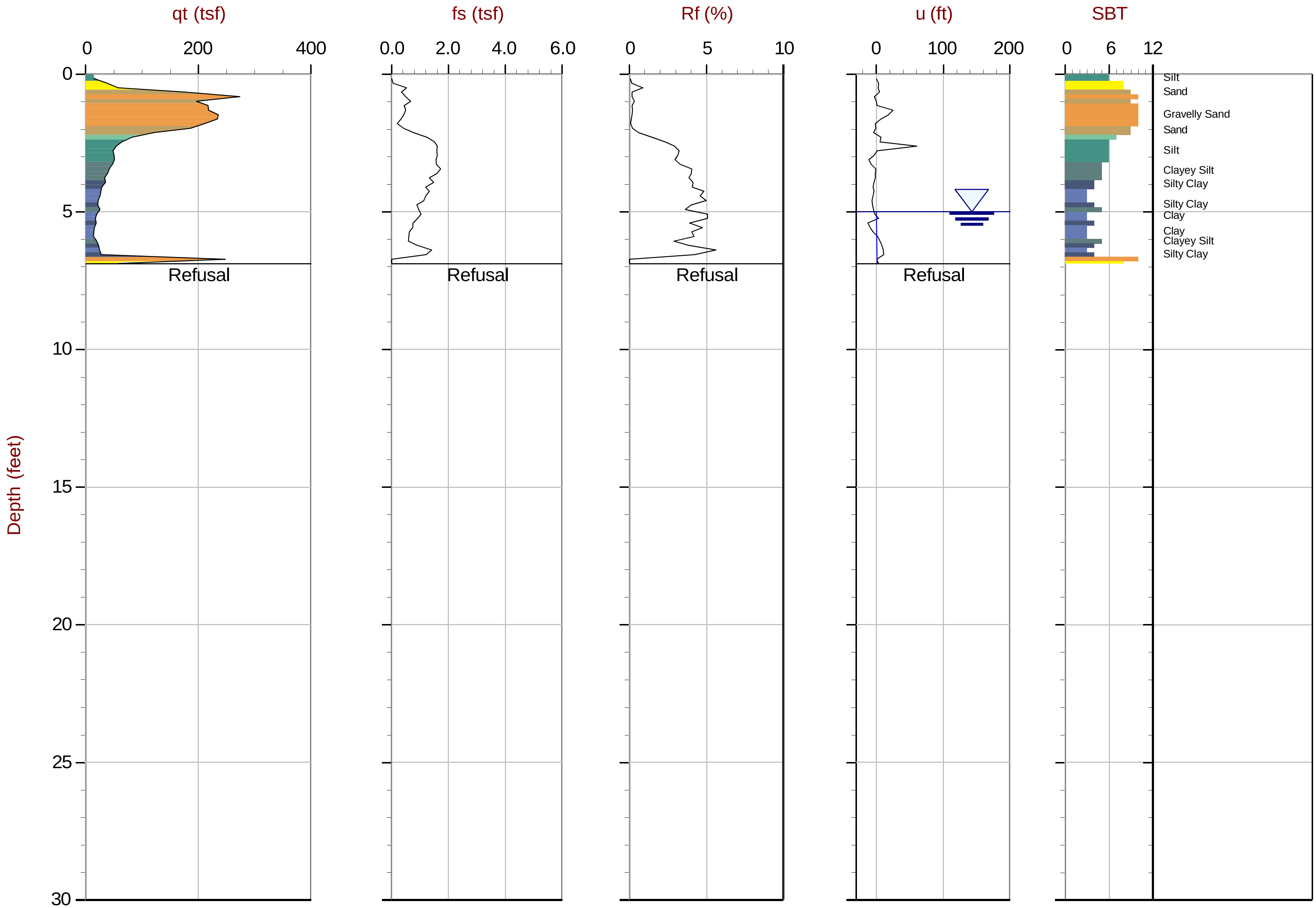
Geosyntec

Job No: 11-959
Date: 07:07:11 10:22
Site: East Dike

Sounding: CPT-09A
Cone: 184:T1500F15U500



Max Depth: 2.100 m / 6.89 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: Every Point

File: 959CP09A.COR
Unit Wt: SBT Chart Soil Zones

SBT: Lunne, Robertson and Powell, 1997
Page No: 1 of 1

CONETEC

Geosyntec

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Site: East Dike

Sounding: CPT-09B
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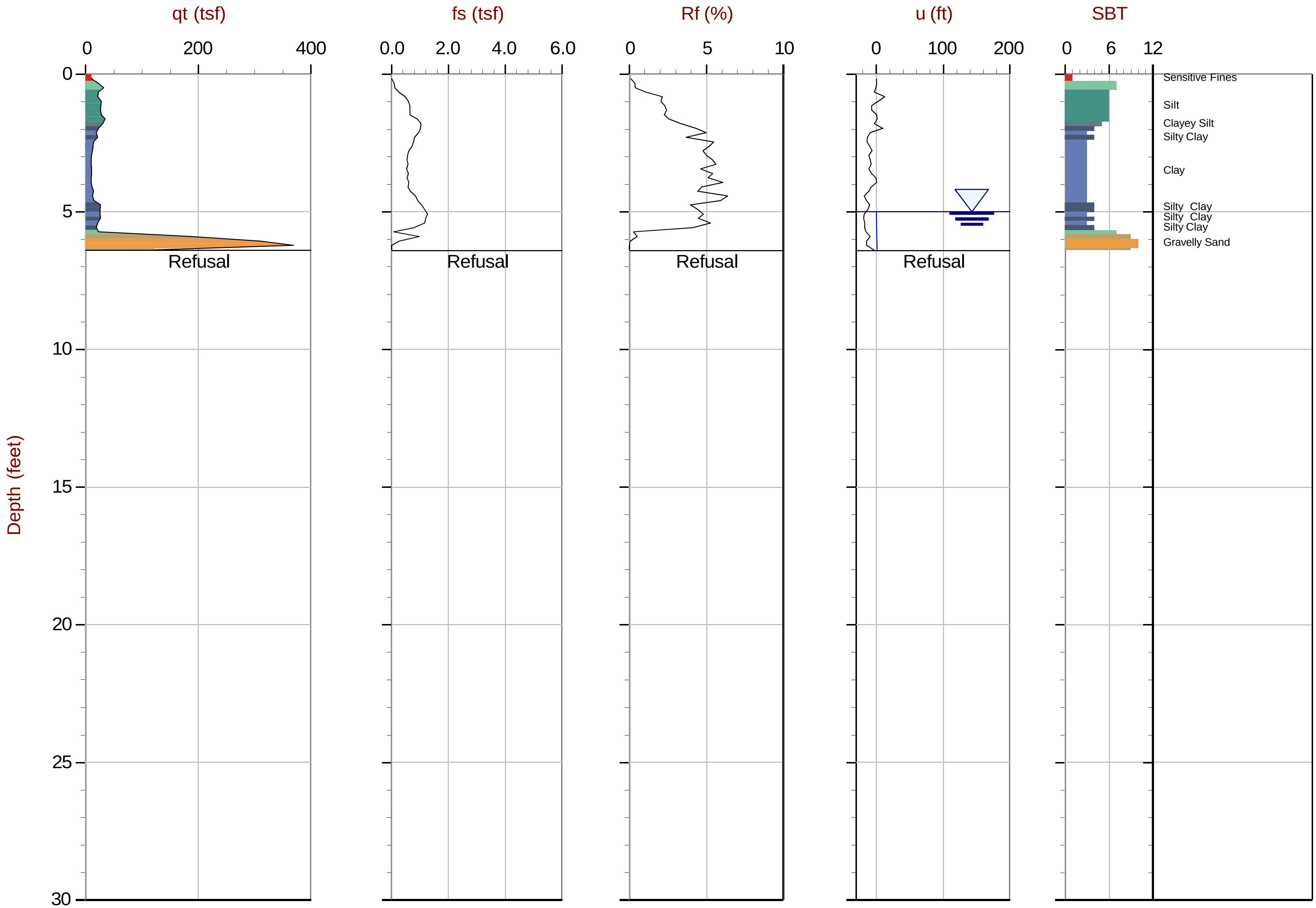

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SBT: Lunne, Robertson and Powell, 1997
Page No: 1 of 1

CONETEC

Geosyntec

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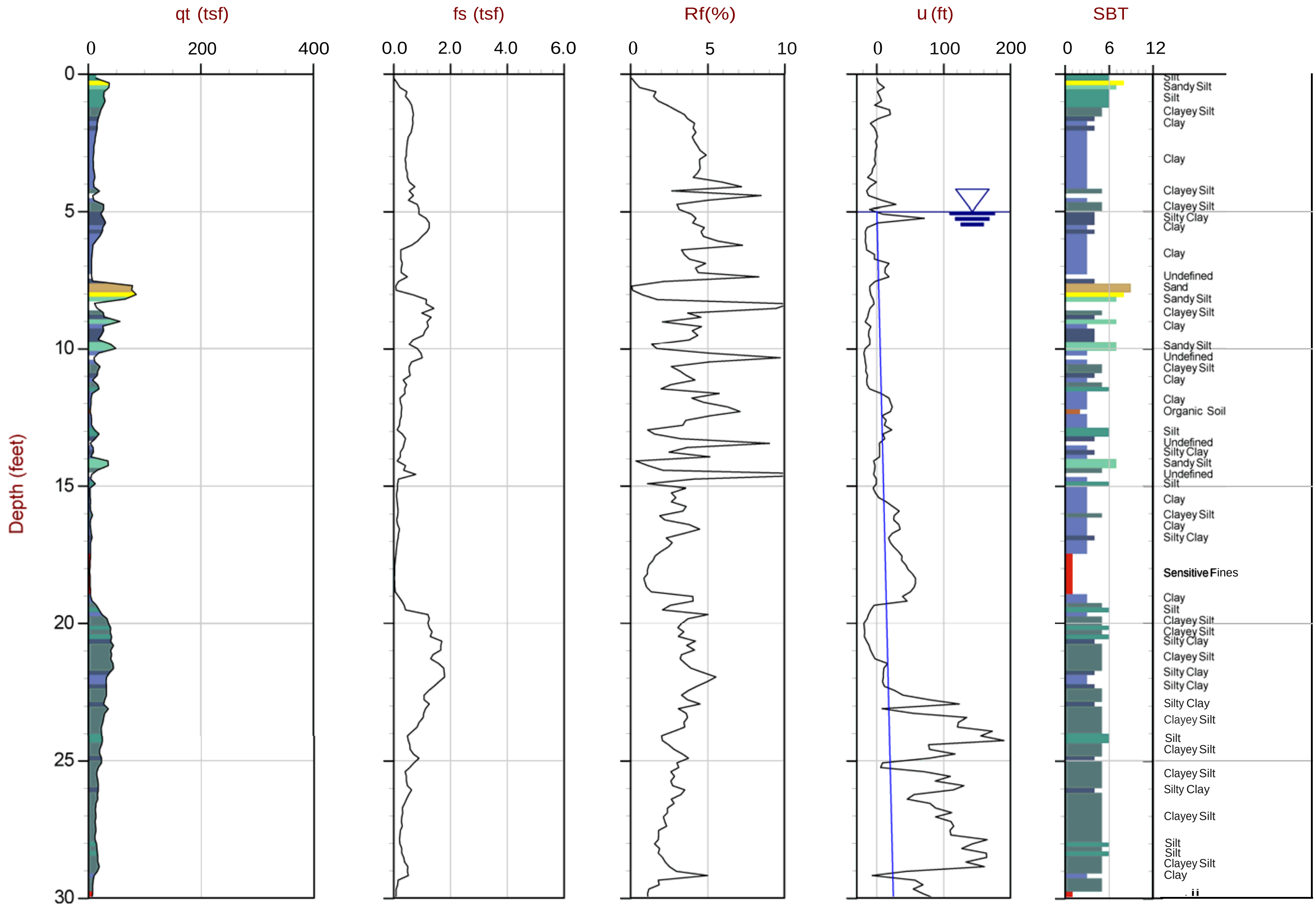

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Unit Wt: SBT Chart Soil Zones

SBT: Lunne, Robertson and Powell, 1997

ConeTec

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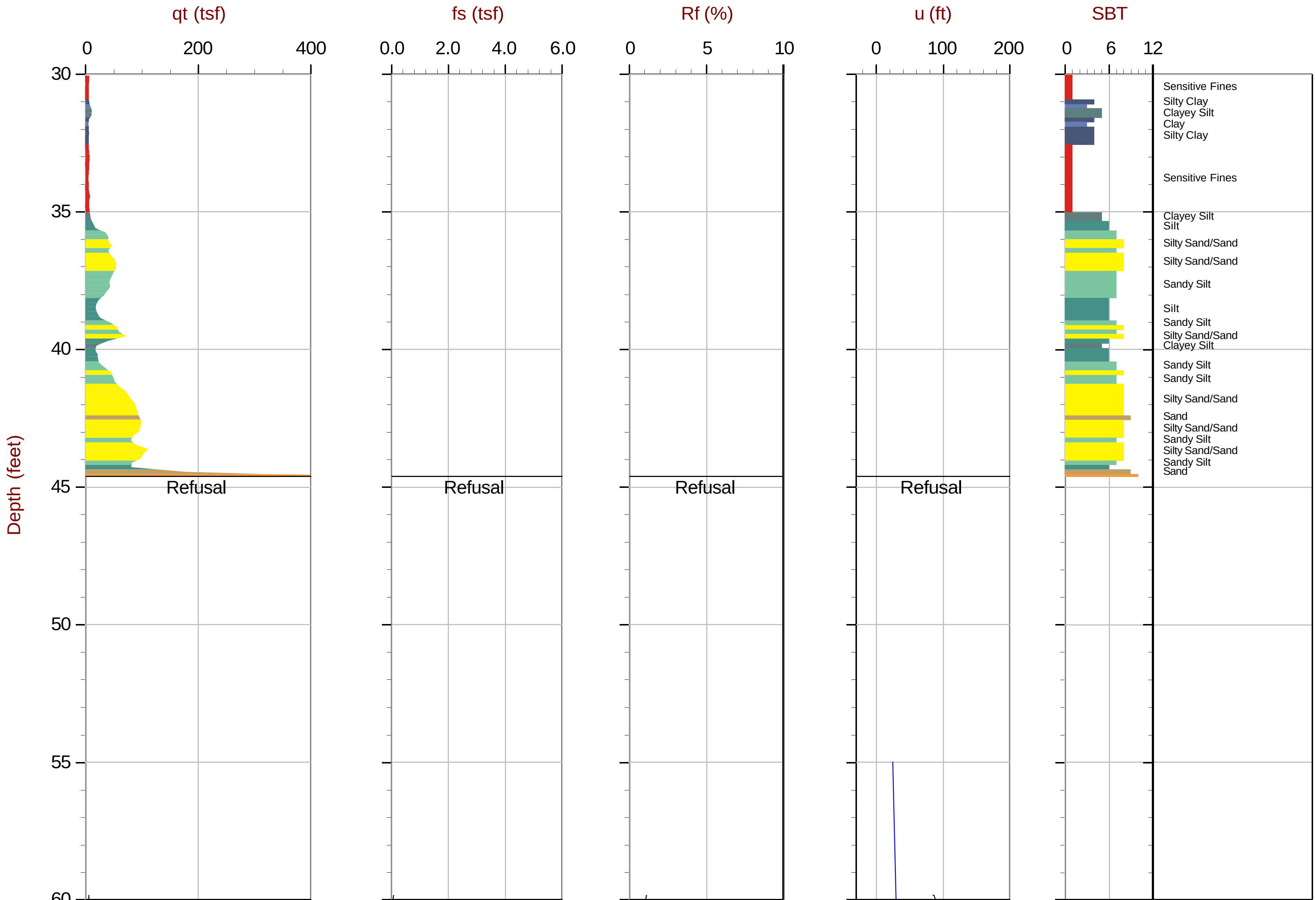

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Unit Wt: SBT Chart Soil Zones

SBT: Lunne, Robertson and Powell, 1997

CONETEC

Geosyntec

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Site: East Dike

Sounding: CPT-11
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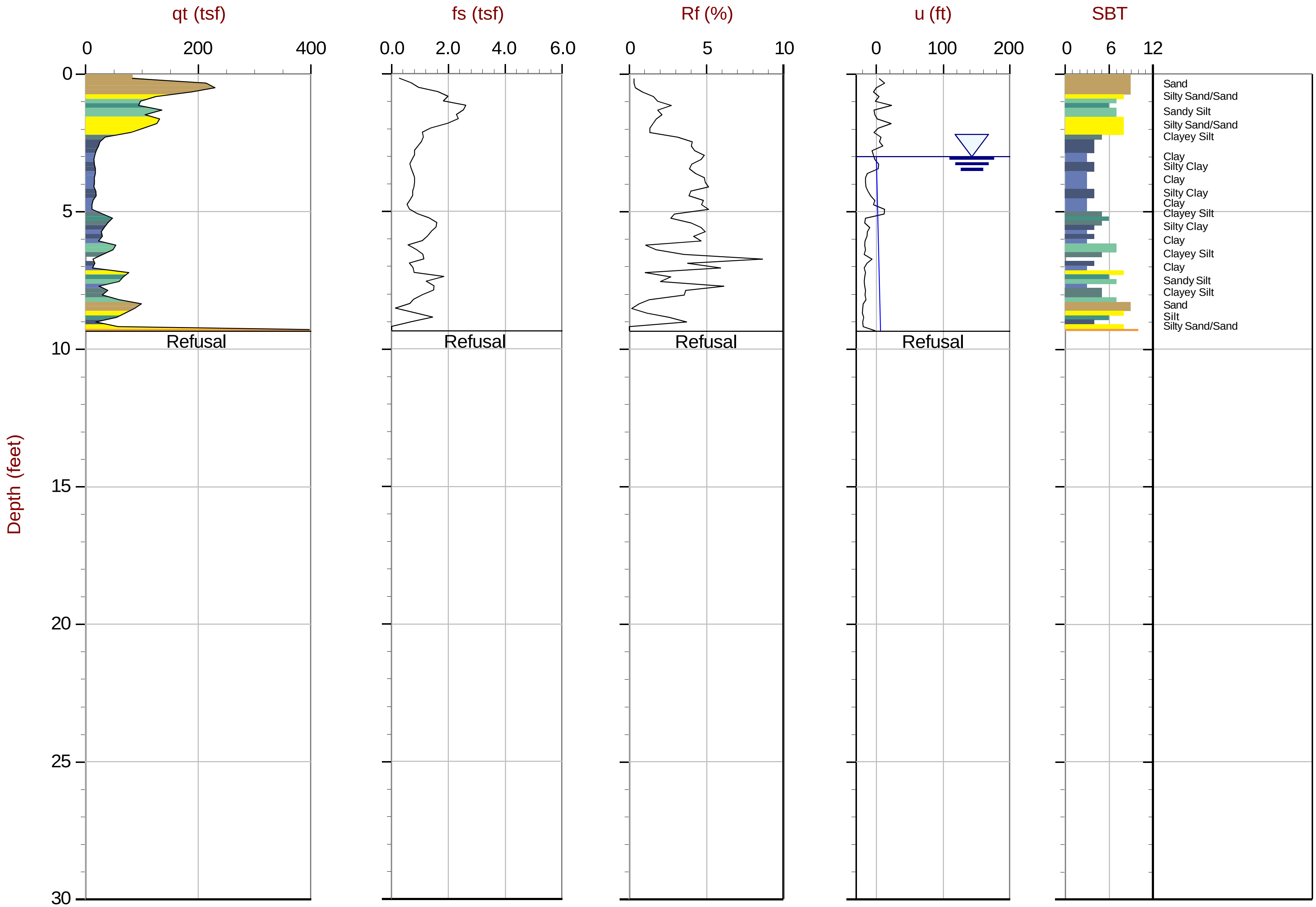

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Unit Wt: SBT Chart Soil Zones

SBT: Lunne, Robertson and Powell, 1997
Page No: 1 of 1

CONETEC

Geosyntec

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Site: East Dike

Sounding: CPT-11A
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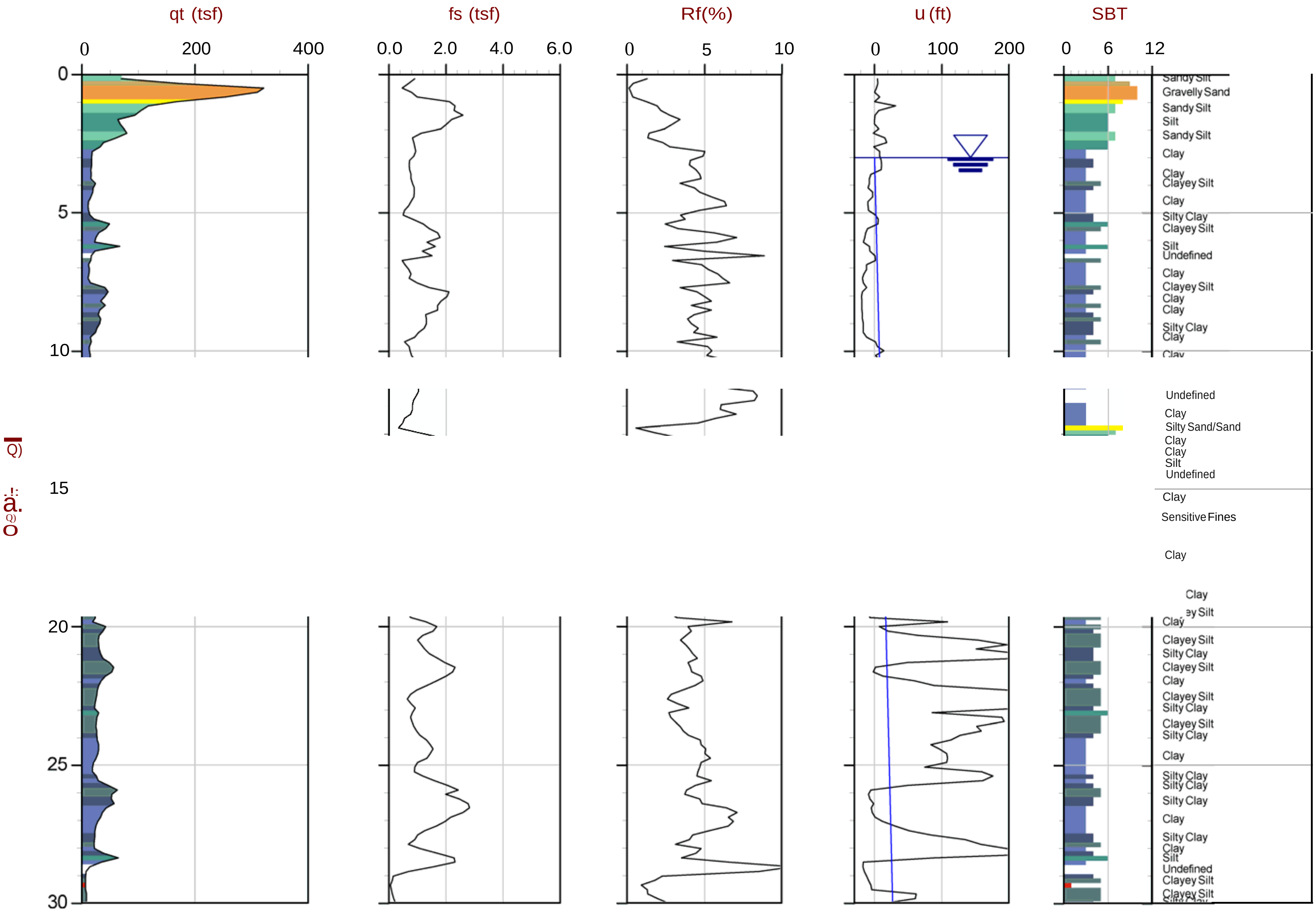

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Unit Wt: SBT Chart Soil Zones

SBT: Lunne, Robertson and Powell, 1997
Page No: 1 of 2

CONETEC

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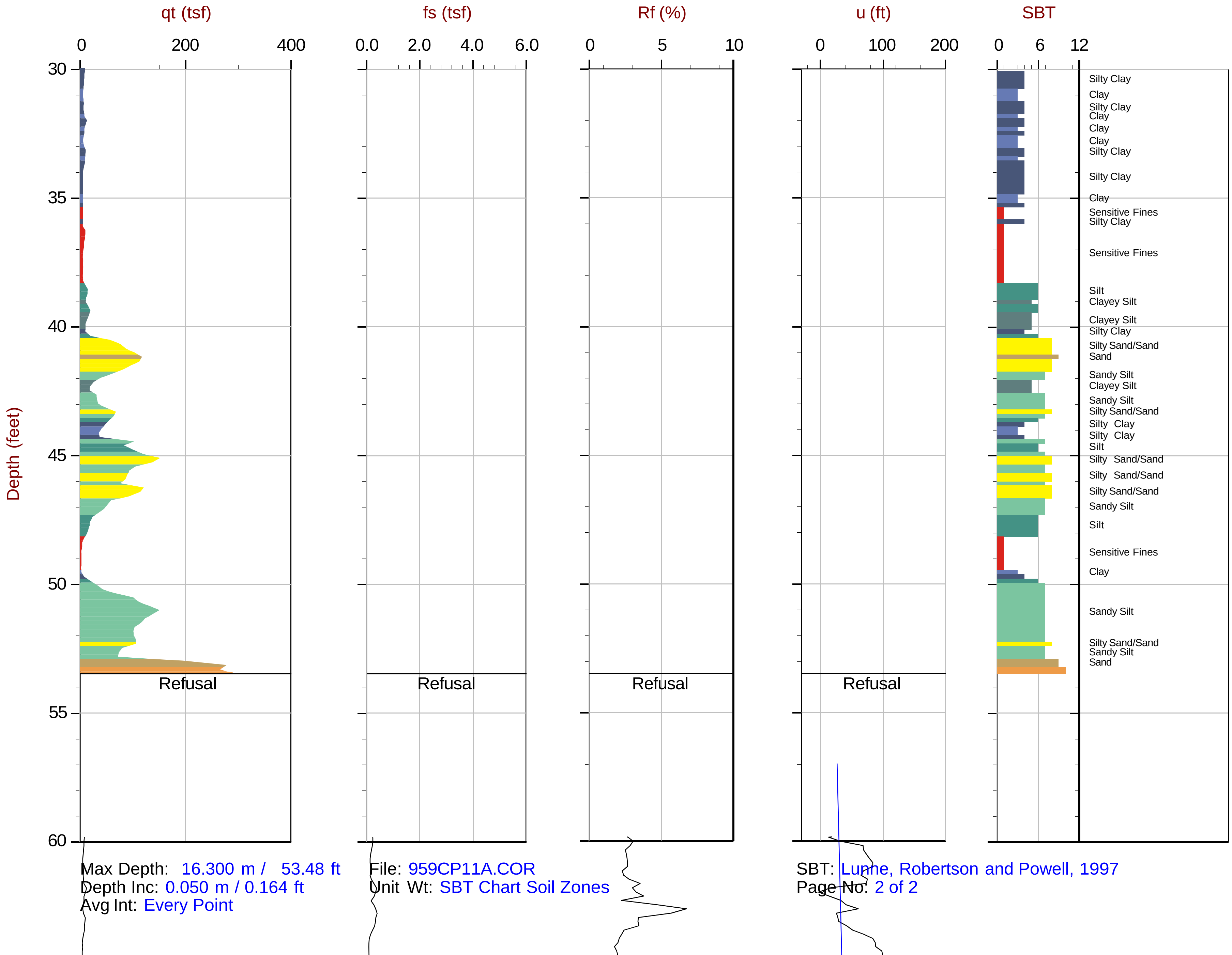

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SBT: Lunne, Robertson and Powell, 1997

CONETEC

Geosyntec

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Site: East Dike

Sounding: CPT-12
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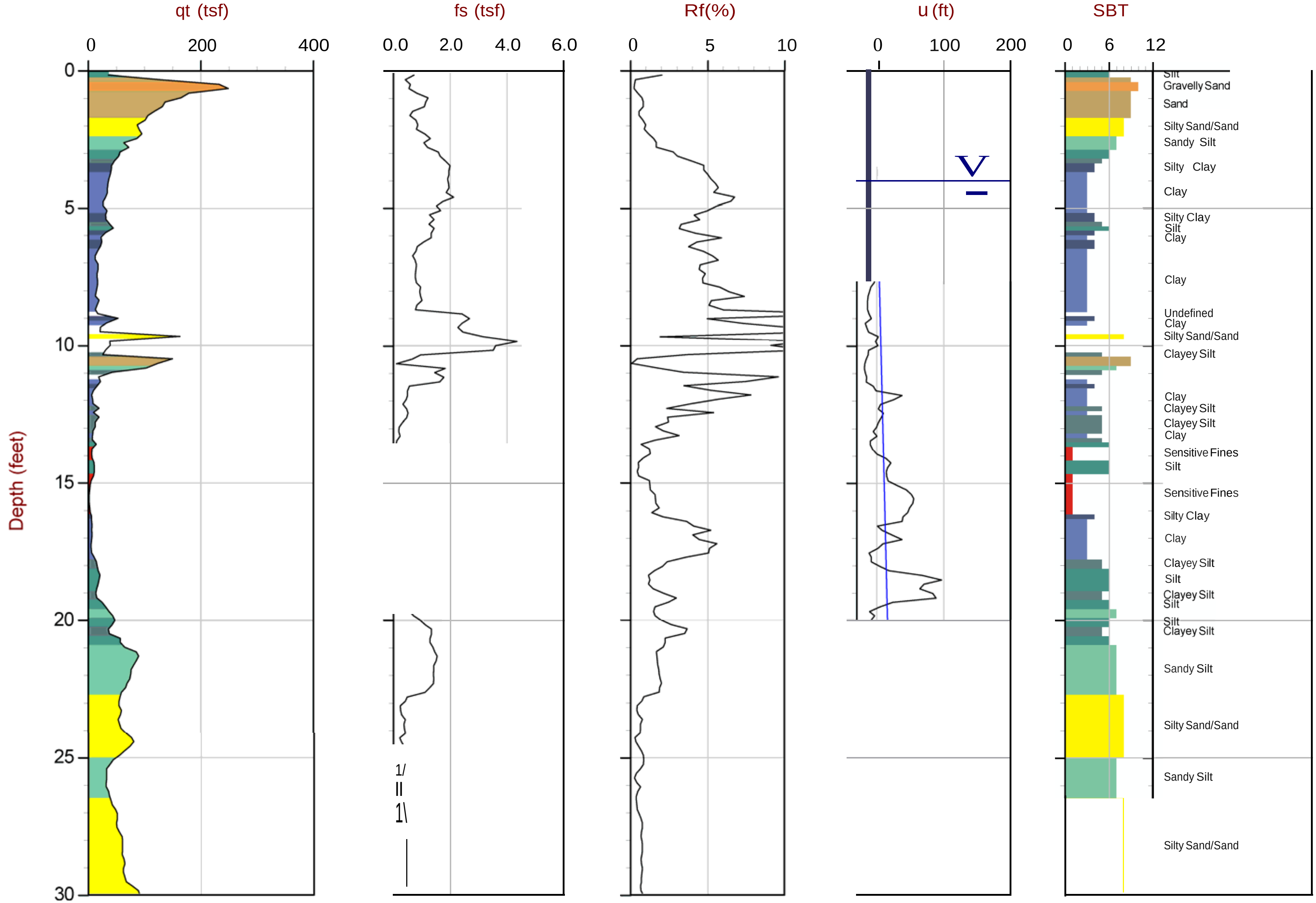

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Avg Int: Every Point

File: 959CP12.COR
Unit Wt: SBT Chart Soil Zones

SBT: Lunne, Robertson and Powell, 1997
Page No: 1 of 2

CONETEC

Geosyntec

Job No: 11-959
Date: 07:06:11 13:11
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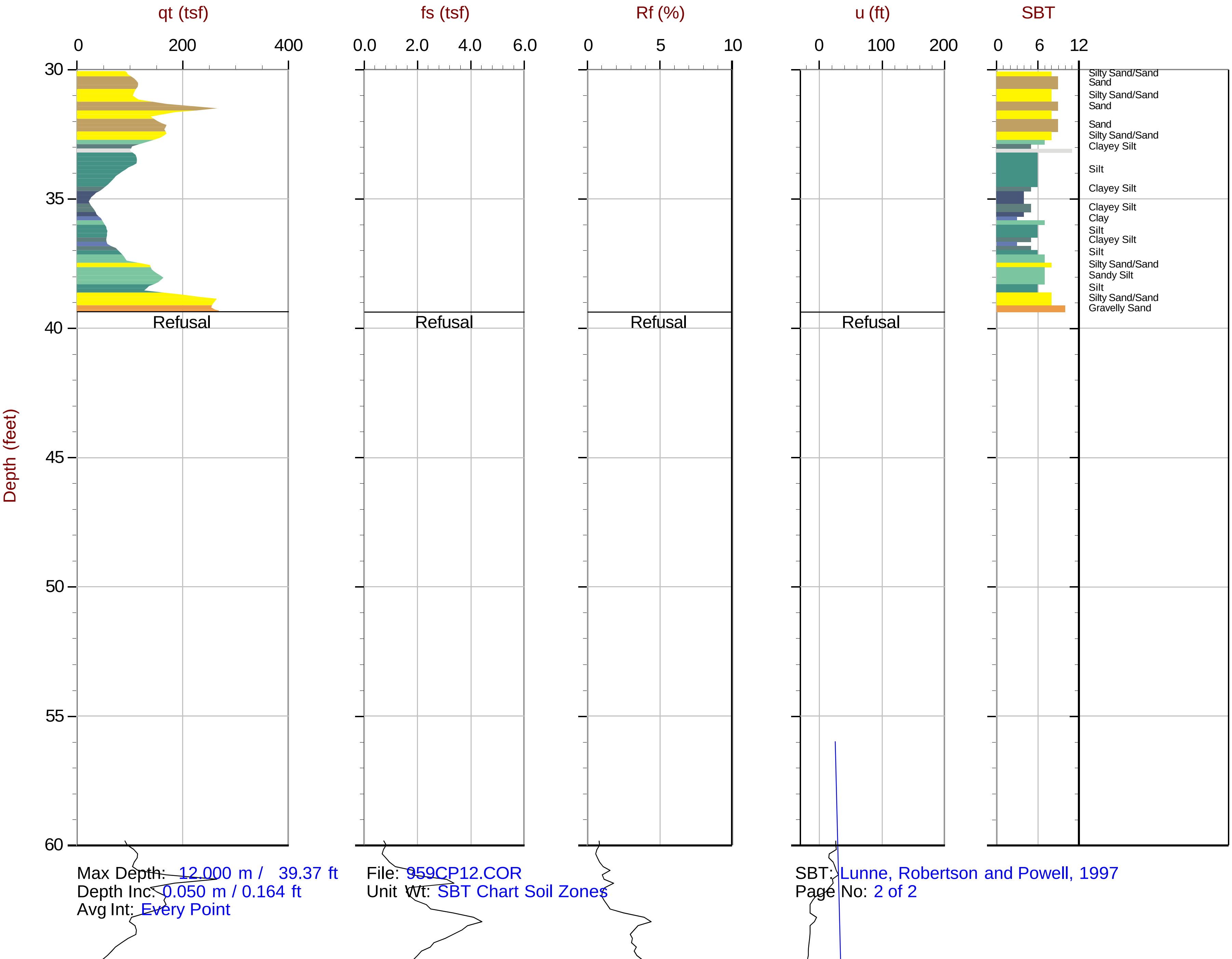

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Avg Int: Every Point

File: 959CP12.COR
Unit Wt: SBT Chart Soil Zones

SBT: Lunne, Robertson and Powell, 1997

ATTACHMENT B – DRAIN CLOSURE REPORT

TECHNOLOGY ADVANCEMENTS

REMEDIATION OF THE KINGSTON FOSSIL PLANT KlF006
ANOXJC LIMESTONE DRAJN

PROJECT CLOSURE REPORT
Project RG228

September 1998

PROJECT CONTACTS

Organization	***Contact***
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Submitted By: ______________________ / 9/9/98
Technology Advancements Project Manager — Date

Approved By: ______________________ / 9/9/98
Manager, Environmental Energy Technologies — Date

Customer: Concurrence ______________________ ____________
F&HP. APT&RI — Due

Remediation of the Kingston Fossil Plant KIF006 Anoxic Limestone Drain

1.0 Introduction: This report describes the remediation of the anoxic limestone drain at the Kingston Fossil Plant KIF006 constructed wetland for treating acid ash pond leachate. The report also describes historic activities at the site which have contributed to the success of the overall passive treatment system at KIF006.

From 1984-1997, TVA's Fossil and Hydro Power (Fuels, Advanced Production Technology & Regulatory Integration, Fossil Operations), and Customer Services and Marketing (Technology Advancements) invested over $3 million into demonstrating and applying constructed wetlands-based, passive treatment technologies for managing various coal-related aqueous acid discharges. Passive treatment systems (PTS) are cost-effective alternatives compared to conventional treatment methods such as pumping and chemically treating. Through the constructed wetlands program, TVA pioneered and advanced important new technologies, such as the aerobic constructed wetland and the anoxic limestone drain, both of which revolutionized passive treatment of acid drainage and other toxic, metal-bearing aqueous discharges associated with the coal and electric utility industries.

In 1987, TVA constructed an aerobic wetland to treat acid water at the Kingston Fossil Plant in Roane County, Tennessee. The prototype, aerobic wetlands design was inadequate to treat the strong acid drainage and required additional components to improve the treatment efficiency of the system. The chief new addition was an anoxic limestone drain (ALD). The ALD is a passive pretreatment stage consisting of a buried trench backfilled with limestone ($CaCO_3$), through which acidic, anoxic groundwater (e.g., ash leachate) is routed (Figure 1).[(1)] The acidic water slowly dissolves the limestone which neutralizes acidity in the acid drainage.

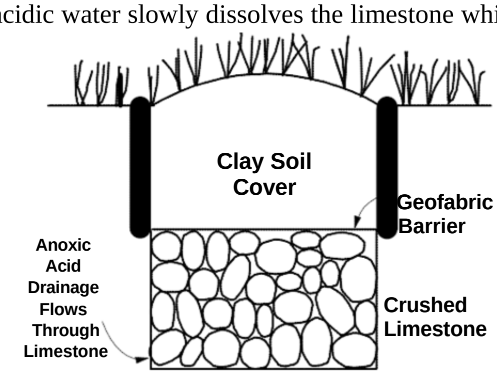

Figure 1. Cross-section of Simplified Anoxic Limestone Drain

1.1 Background/Project Need: Figure 2 is a plan of the KIF006 passive treatment system (also showing recent modifications). In October 1984, an Environmental Protection Agency compliance evaluation inspection identified seepage (i.e., "red" water) from an ash embankment adjacent to the west bank of the intake channel at the Kingston Fossil Plant. TVA, in order to comply with EPA water discharge regulations, constructed a 430 m collection trench along the ash embankment toe and installed a temporary facility to pump the seepage back to the top of the embankment to the bottom ash sluice channel (Figure 2). Additionally, crushed limestone was placed in the collection channel in an attempt to raise the pH of the seepage; this action failed to provide any treatment and was nevertheless not a long-term, cost-effective option for managing the seepage. In 1987, TVA constructed a wetland (KIF006) to treat the coal ash drainage. KIF006 was a three cell, aerobic system totaling 9300 m^2 preceded by the 430 m seepage collection trench. Final discharge was pumped to the ash basin for treatment because it did not meet anticipated discharge permit limits (pH = 6-9, Fe<3 mg/l) (see Table 1).

In 1988 Cell 3 was converted to a compost-type marsh (0.5 m spent mushroom compost placed over 0.3 m of crushed limestone and planted in cattail). There were no significant results from this action.[(5)] In 1991, 4 ha within the source groundwater recharge basin were treated with a bactericide (B.F. Goodrich *ProMac®*) and reclaimed (shown as the "Reclaimed Ash Disposal Area" on Figure 2), a 3300 metric ton anoxic limestone drain (ALD) was installed, and Cell 1 was converted to an oxidation basin. Water quality improved but not to compliance levels and, of greater consequence, wetlands inflow (ALD outflow) decreased to 70 l/min.

Investigations in 1993 revealed a constriction in the ALD [(2)] which required remediation in order for the ALD and the overall passive treatment system to function at peak efficiency. The constriction investigation and repair were proposed by TA under a FY 1995 project which was completed in April-September 1994.

1.2 Project Goals: The project goal was to investigate the cause of reduced flow and successfully remediate a suspected constriction in the existing ALD and make any other changes to the existing KIF006 passive treatment system in order to achieve peak acid water treatment. Further goals included transferring the lessons learned from this investigation to other TVA and non- TVA customer sites. These goals would benefit Fossil and Hydro Power Operations and other customers by, 1) reducing water pump and treat activities and associated costs, 2) improving designs of ALDs and other passive treatment technology components, thereby enabling TVA F & HP engineers, plant operators, and managers a means to make informed decisions on acid drainage treatment options.

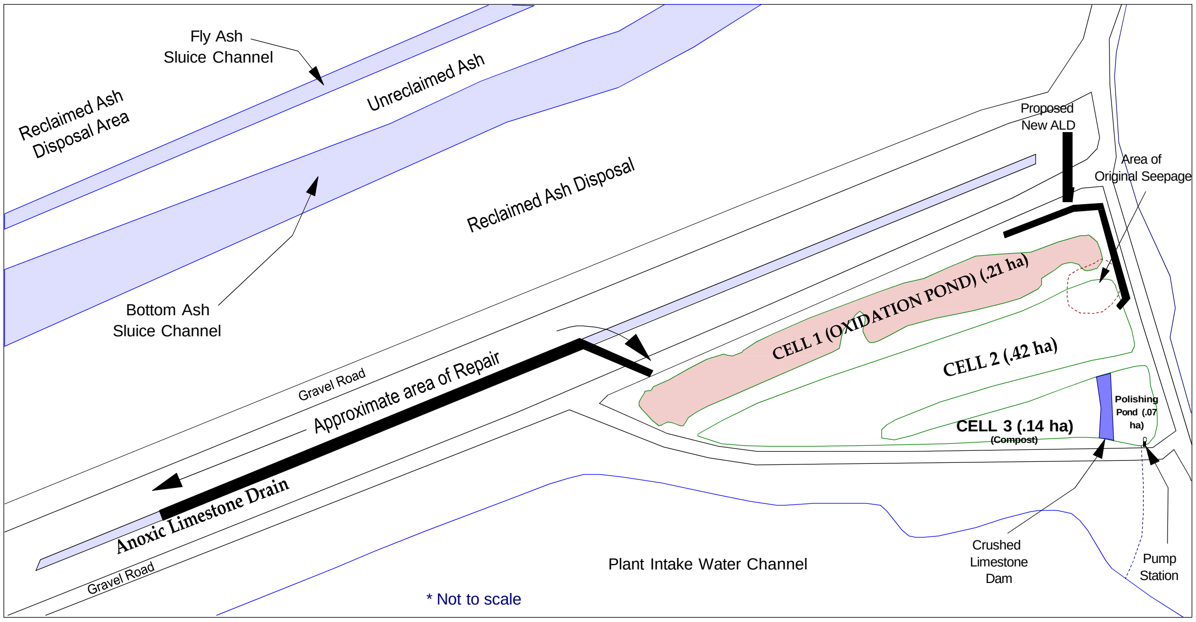

Figure 2. Plan of KIF006 Passive Treatment System

1.3 Scope of Work: This project involved investigating causes for reduced flow in the existing ALD and identifying areas in the existing ALD that were constricted or otherwise hydraulically incapacitated and resulting in inefficient treatment of the acid drainage.(1) Once the causes were identified, those portions of the ALD that required repair were repaired using updated standards for the technology. Approximately 120 m of the original 430 m ALD was rebuilt. The system was then monitored for 3 years (October 1994-September 1997) for performance. A technical paper will be prepared on the entire passive treatment system.(3)

1.4 Deliverables:

1.4.1 Completed, state-of-the-art anoxic limestone drain at the KIF006 constructed wetland site.
1.4.2 Reduced costs associated with O&M of the passive treatment system and with the pumping facility.
1.4.3 Improved ability to negotiate favorable NPDES permit requirements for KIF006.
1.4.4 Closure report outlining the major results of the project.
1.4.5 Technical report for publication (in progress).
1.4.6 Demonstration of ability of a constructed wetlands system to passively treat highly acidic ash pond leachate.
1.4.7 Increased aesthetic and wildlife values of the wetlands due to improved pH and increased productivity.
1.4.8 Demonstration of the use of a power auger for investigating anoxic limestone drains.
1.4.9 Verification of the results of the 1993 hydrologic investigation.
1.4.10 Data base for improving guidelines for constructing anoxic limestone drains.
1.4.11 Demonstration of TVA Environmental Responsibility.

1.5 Project Organization

Technology Advancements - Greg Brodie, Project Manager
F&HP Fossil Operations, Kingston Fossil Plant - R. L. Pope, Environmental Engineer
Engineering Services, Norris Engineering Laboratory - Tony Rizk, Civil Engineer
Engineering Services, Central Region - H. Nick Taylor, Environmental Engineer
Engineering Services, Central Region - Tina M. Tomaszewski, Environmental Engineer
Engineering Services, Central Region - Tony Knight, Environmental Engineer
F&HP Advanced Production Technology and Regulatory Integration - Larry Wolfe, Sr. Regulatory Specialist

2.0 Project History

2.1 Passive Treatment System History: In October 1984, the EPA inspection identified the seepage and TVA installed the collection trench and pumping facility. In 1984-85, alternatives to permanently manage the acid discharge were evaluated, including installation of a permanent station to pump the seepage to the ash pond for treatment; elimination of the unlined bottom ash sluice channel and reclamation of the infiltration source area; chemical treatment and discharge; bactericides; constructed wetlands; and other engineering solutions. Constructed wetlands was selected based on cost (wetlands was least cost), high probability of success at effectively treating the discharge, and TVA's desire to advance an environmentally preferred technology for treating a pervasive problem (i.e., ash pond seepage).

In October 1987, TVA constructed the KIF006 wetland to treat the acid drainage. KIF006 was constructed as a three cell, aerobic system totaling 9300 m^2 preceded by the 430 m seepage collection trench. Unlined cells were constructed by excavating into unclassified fill which was used for dike construction. Water depths were 0.15-m to 1.0-m, averaging 0.4-m. Cells were planted in cattail (*Typha sp.*) and bulrush (*Scirpus sp.*). Initial inflow averaged 1574 1/min with a maximum of 2271 1/min emanating from the toe of the coal ash disposal embankment. Hydraulic loading was 0.24 1/day/m^2 of wetland. Seepage was characterized by pH = 5.5, Fe = 170 mg/l, Mn = 4.4 mg/l, acidity > 600 mg/l, alkalinity =40 mg/l, and Al < 0.05 mg/l. Outflow was characterized by pH = 2.8, Fe = 82 mg/l, Mn = 11 mg/l and averaged 100 1/min greater than inflow due to seepage upwelling in Cell 1. Average chemical loading was 41.4 g Fe/day/m^2 of wetland. Water monitoring was conducted at four locations.[3] Final discharge was pumped to the active ash disposal basin for treatment because it did not meet limits which would be imposed by inclusion of the discharge in the Kingston Fossil Plant National Pollutant Discharge Elimination System (NPDES) Permit.

In 1988 KIF006 Cell 3 was modified by conversion to a compost-type wetland.[4] The upper half of Cell 3 was backfilled with 0.5 m spent mushroom compost overlain by 0.3 m crushed (2-4 cm) limestone; a crushed limestone berm was placed transverse to the cell to support the limestone and compost. No significant improvements resulted.

In 1990, investigations were conducted toward designing an ALD to pretreat the acid drainage before it flowed into the wetland.[5] In August 1990, construction began on the ALD. Weather delays resulted in construction deferrals until October 1990, when TVA budgeting issues caused an indefinite deferral of the project to August 1991. The ALD was completed in September 1991. The ALD was designed at 1.6 m wide, .9 m deep, and 430 m long and located in the existing seepage collection channel. Geofabric lined the bottom of the trench which was filled with 3300 metric tons of 4 cm crushed limestone. The limestone was covered with filter fabric and capped with .6 m - 2 m of compacted local clay loam. The ash embankment and ALD were sloped approximately 3:1. The first cell of the KIF 006 wetlands was converted to an oxidation-precipitation pond. It was dredged to a depth of about 2 m and a shallow subsurface rise was installed near the cell midpoint to allow for reaeration of the water. Water quality was improved but not to the degree necessary for permitting the discharge (i.e., pH > 6, Fe < 6 mg/1). After installing the ALD, wetlands inflow decreased from 1574 1/min to less than 70 1/min. Alkalinity in the reduced flow was approximately 250 mg/l, thus the ALD was functioning properly from a geochemical standpoint. However, it was suspected that something was causing a restriction in the observed flow. Such a constriction prevented adequate flow of alkaline water into the wetlands, which received an additional approximate flow of 100 1/min acid water via a seep into Cell 2; this additional non-ALD-treated seepage contributed significantly to causing water quality in the wetland to degrade to sub-discharge standards.

In April 1991, a 10-acre, wet, bottom ash disposal area upgradient from the KIF006 wetlands was treated with ProMac® bactericide[6] and reclaimed with a mixture of lime-amended soil and grass. The area, which contained pooled water from the plant's bottom ash sluice canal, was presumed to be a significant contributor to the KIF006 drainage. There were no significant water quality improvements attributed to this effort. Immediately after the bottom ash area remediation and ALD installation, the wetland inflow (i.e., the ALD outflow) decreased to an estimated 450 l/min (expected design flow was 750 l/min). The flow continued to decrease and in 1993 was averaging about 70 l/min. A team was formed to identify probable causes and recommend remediation procedures.

O&M activities from 1987-1991 were associated with spillway and dike repair, muskrat and Canada Goose impacts, pump station operation, and replanting aquatic vegetation.

2.2 Technology Advancements Project: A field inspection was performed in April 1993, when a 2.5 m by 2 m pit was excavated near the current ALD outlet to expose a small section of the ALD. The bottom liner and the top filter fabric had been reversed during installation, but neither appeared to be clogged. Although the designed depth of the ALD was one meter, the actual depth appeared to be less than 0.6 m. Minor flow was observed in this section of the ALD and the limestone did not appear to be armored. Consequently, a gravel outlet with two 8-inch slotted ADS N-12 pipes was installed along an excavated trench between the pit and the wetland to provide additional drainage. The second outlet increased the discharge into the wetland by about 1-2 liters/sec.[2] Since flow was observed in the ALD, the possibility of a constricted drain causing the low-level flow was examined next.

Hydrologic testing was performed to determine locations and extent of significant flow zones. Results indicated that the ALD geofabric and filter fabric were permeable and allowed some hydraulic communication between the ALD and surrounding clay. It appeared that the ALD was constricted between two test wells used in the investigation.(2) This constriction caused the phreatic surface in the ALD and its surrounding clay to rise on the south side (i.e., the upgradient half of the ALD), creating nearly artesian conditions in the upgradient portion of the ALD.

Several remediation alternatives were evaluated:

1. Install a new ALD outlet from somewhere before the ALD constriction to the wetland. The exact connection area could be determined by power auger.

2. Install a new wetland along the length of the ALD extending from the existing wetland to south of the constriction and connect the ALD to the wetland via several small outlets. A weir would control the discharge rate from the ALD and the new wetland into the existing wetland.

3. Replace the section of the constricted ALD.

4. Install a new wetland on top of the existing ALD. The new wetland would discharge into the existing wetland. A weir would control the discharge rate from the ALD and the new wetland into the existing wetland.

Two hypotheses were formulated to explain the constriction in the ALD:

1. The thickness of the limestone in the ALD was inadequate. The ALD was designed with a 1 m minimum thickness of 4 cm crushed limestone. There was evidence from geologic maps that a shale bedrock knob passed through or near the area containing the constriction. If this bedrock was not excavated to accommodate the 1 m layer of stone, a hydraulic constriction could have been present in the ALD. This scenario would have required replacing a 50 m section of the ALD where the constriction existed.

1. During construction of the ALD, there were several periods of bad weather and other delays which precluded construction work. If during these delays, the sides of the excavation caved into the open trench where limestone was to be placed, and the caved area was not adequately cleaned out, there could exist a small, but significant physical constriction in the ALD. This type of modification would require repairing the area of the constriction.

Two other possibilities existed but were deemed unlikely based on observations from monitoring wells and excavations of the ALD during the summer of 1993. Chemical clogging of the wetlands by ferric hydroxide, gypsum, aluminum, or other materials could cause clogging. There was no evidence of these types of materials within the ALD or at the outlet. There is also the possibility that the flow was reduced or altered by the reclamation of the upgradient ash disposal area. This idea was not supported by the 1993 hydrologic investigation of the ALD.

In order to accurately determine the location and nature of the ALD constriction, a power auger was used to investigate the ALD characteristics (water levels, limestone thickness, sub-ALD geologic conditions). Six-inch holes were drilled to refusal (presumably shale bedrock) or to 2 m below the bottom of the ALD. Drill cuttings were logged geologically and thickness of ALD limestone estimated using field logging techniques. Augering results were used to determine the extent and nature of the constriction, which was determined to be primarily a rise in the shale bedrock that was not excavated during construction and resulted in inadequate depth of the ALD.(2)

The ALD was repaired in 1994 and is the primary subject of this closure report.(7) Approximately 120 m of the existing ALD were replaced to bypass the constriction (Figure 2). The upper 75% of the ALD was backfilled with 1.2 m of crushed limestone (5-7.5 cm) from the Franklin Limestone Quarry, Crab Orchard, TN; the limestone gradually graded upward to 15 cm stone in the lower gradient portion of the new ALD. E. H. Reclamation Services performed the construction work.

3.0 Project Results: Monitoring was conducted for flow, pH, Fe, Mn, acidity, and alkalinity at the ALD outlet and at the discharges from Cells 1, 2, and 3. Effluent samples were obtained during daylight hours on a monthly basis for 4 months and thereafter periodically. All samples were collected and analyzed according to standard methods.[8] Total metals samples were collected in 500 ml acid-rinsed polyethylene bottles, preserved with $HN0_3$ to a pH of greater than 2.0 by tilting the bottle gently into the effluent stream. Samples were placed on ice and transported to the Chattanooga Environmental Chemistry Laboratory for analyses. Samples were digested with concentrated, redistilled $HN0_3$ and HC1, reduced to 20 ml, diluted back to volume, centrifuged or filtered depending on solids, and then analyzed by atomic emission or atomic adsorption. Dissolved oxygen, conductivity, oxidation-reduction potential, and pH were measured in the field with a Surveyor 2 Hydrolab .

In 1995-97, inflow to the wetland from the ALD averaged 265 l/min with a maximum of 454 l/min; hydraulic loading was 0.04 l/day/m^2. Flow reduction from design flow of 750 l/min was attributed to reclamation of the bottom ash area which significantly reduced the recharge area for the seepage. Table 1 summarizes the water quality at the discharge of the wetlands after the various modifications were completed. The latest data collected at the KIF006 site are shown in Table 2. Average Fe loading since the ALD modification is 13.6 g/day/m^2 of wetland, significantly lower than that measured in the 1987-91 period; this loading factor is far more favorable for peak passive treatment system operation.

	Original Seepage (1984-97)	Aerobic Wetland and Compost Modification (1987-91)	ALD, Oxidation Pond, ProMac, Ash Area Reclamation (1991-94)	ALD Modification (1994-97)
pH	5.5	2.9	3.3	6.2
Total Fe, mg/l	170	83	27	4.0
Total Mn, mg/l	4.4	11	9.3	2.4
TSS, mg/l	40	<5.0	<5	15
Alkalinity, mg/l	40	0	0	10
Acidity, mg/l	350	230	100	40
Flow, l/min	1574	1674	220	365

Table 1. Water Quality of from KIF006 Discharge After Various Modifications Were Installed

	ALD Discharge	Cell 1 (Oxidation Pond) Discharge	Cell 2 Discharge	Cell 3 Discharge
pH	6.8	6.6	6.6	6.6
Total/Dissolved Fe, mg/l	75/62	6.9/2.7	3.1/1.5	2.7/1.4
Total/Dissolved Mn, mg/l	1.5/1.4	1.6/1.5	1.6/1.5	1.6/1.5
Sulfate	580	630	670	670
Total Alkalinity, mg/l	134	40	42	42
Total Acidity, mg/l	146	20	16	14
Estimated Flow, l/min	356	-	-	-

Table 2. Water Quality of from KIF006 Passive Treatment System in August 1997

Clearly, the modification to the ALD consisted not of removing the constriction, but by installing a new section of ALD, bypassing the constricted section. This action resulted in improved water quality in the wetland. However, the project was unsuccessful in lowering the phreatic surface in the south end of the ALD. The existing groundwater monitoring Well 10 is still artesian in wet weather and a significant seep has broken out in the area south of the repair. The reason for the seep and the artesian conditions in Well 10 are unknown, but are probably related to the hydraulic regime created by the ALD.

Additionally, samples upstream vs. downstream of the new drain indicate loss of iron, raising questions about the ability of the ALD to maintain iron in a reduced state.

3.1 Costs: The Technology Advancements project (ALD modification) cost $47,000. Capital cost of the aerobic wetlands was $131,700 (1987) and annual operating costs from 1987 to 1997 were about $500,000 due to pumping and monitoring.[9] The compost modification cost $12,000 (1988). Capital costs for the anoxic limestone drain were $167,523 (1991). ProMac and reclamation of the bottom ash area cost $22,255 (1991). Thus, the total cost of the KIF006 system from 1987-1997 was $833,478. Conventional chemical treatment over this period would have been over $950,000.

3.2 Project Benefits: The cost-benefit analysis for the passive treatment system is attached as Appendix 1. The attached cost-benefit analysis shows a project Net Present Value of $446,000. Benefits derive primarily from eliminated costs for operation of the pumping facility and avoiding the cost of another, conventional treatment system. The benefits to costs ratio is 7.4. Other benefits from the project include:

3.2.1 Complete or improved treatment of acid drainage at the KIF006 constructed wetlands.
3.2.2 Potential elimination of operation and maintenance costs and activities associated with the pumping facility.
3.2.3 Improved compliance with future NPDES permits.
3.2.4 Demonstration of improved ability of ALD technology to passively treat highly acidic ash pond leachate.
3.2.5 Increased aesthetic and wildlife values of the constructed wetlands.
3.2.6 Improved ease of further modifications to improve water quality at this facility.
3.2.7 Improved guidelines for constructing anoxic limestone drains.
3.2.8 Improved public relations.
3.2.9 Improved ability to provide expert technical assistance to non-TVA customers on outside business.

3.3 Lessons Learned: This project disclosed several lessons. One of the root causes of the reduction in flow from the ALD was nonconformance of the construction crew to design specifications (i.e., maintaining a uniform designed depth of the ALD). It is imperative that the site construction engineer adhere to design specifications unless changes are discussed with and approved by the design engineer. Additionally, this project advanced ALD technology by suggesting that larger gradation (>4 cm) crushed limestone be used to avoid clogging by Fe precipitates and silt. The project also demonstrated the possibility to repair an ALD by rebuilding a new one adjacent to the defective ALD rather than removing and renovating the defective ALD.

4.0 Project Recommendations:

4.1 Project Summary: A passive treatment system (KIF006) built in 1987, along with an anoxic limestone drain built in 1991, were inadequately treating acid ash pond leachate at the Kingston Fossil Plant. A modification to the constricted existing anoxic limestone drain was completed in 1994. The modification resulted in improvements to flow through the ALD and in increased generation of alkalinity which allowed the downstream constructed wetland to operate at significantly increased efficiency. The pH of the discharge in the wetland increased from 3.9 to > 6.0, which will allow Kingston Fossil Plant to permit discharge from the constructed wetlands and cease the current pump and treat scenario. The project cost $61,000 and resulted in quantified benefits of $446,000 along with several unquantified benefits.

4.2 Recommendations:

4.2.1 ALD technology, combined with passive treatment systems such as constructed wetlands, should be considered and used where found practicable for treating acid drainage from coal-related sources such as ash ponds, coal piles, and other facilities at TVA fossil plants.
4.2.2 The KIF006 passive treatment system should be sampled at least quarterly for pH (and total Fe if possible) from the ALD and at the final discharge. Other analytes should be considered for periodic analyses if resources are available (total Mn, sulfate, dissolved Fe, alkalinity, and acidity). Biweekly inspections of the system should be conducted to note any significant changes in the nature of the system (such as increased flow, turbidity, drastic water color changes, beaver or other pest activity, vandalism, erosion, etc.). If major discrepancies from the present water chemistry or other adverse conditions are noted, appropriate technical staff (Environmental Research Center - Environmental Engineering; Technology Advancements; or F&HP Environmental Affairs) should be contacted for assistance.
4.2.3 TVA should market its expertise and provide outside customers technical assistance on ALD and passive treatment technology through TVA Resource Management or other organizations.
4.2.4 A professional technical paper should be prepared and published on the project in an appropriate journal.
4.2.5 The passive treatment system is operating marginally, i.e., it is very close to consistently treating water to compliance levels. In the event that Kingston Fossil Plant pursues permitting the KIF006 outfall under the NPDES permit, several remedial provisions should be installed:

4.2.5.1 A beaver-resistant spillway should be installed at the final outfall. This would minimize potential noncompliances and O&M due to beaver dam building.
4.2.5.2 Another small ALD should be installed to pretreat seepage upwelling into Cell 2. This seep is of similar quality to that entering the existing ALD and constitutes about 30% of the total flow in the wetland. If left untreated, it will continue to stress the ability of the wetland to adequately and consistently treat the acid drainage, especially during periods of low flow from the existing ALD.
4.2.5.3 A simple means of chemical treatment should be made available in the lower part of Cell 3, such as soda ash briquettes or caustic soda, to ensure that pH excursions are avoided.

4.2.5.4 Total Fe (i.e., suspended Fe) may be a problem in the discharge if the limit is established at or below 3 mg/l. Permit limits for total Fe should be negotiated with the State at the highest possible level (e.g., > 3.5 mg/l if possible) to ensure consistent compliance. Alternatively, suspended Fe removal in the wetland can be improved by: 1) Properly maintaining Cell 1 by keeping Fe precipitates to less than 60% of the pond capacity; and 2) Ensuring that the aquatic vegetation in the wetland Cells 2 and 3 are properly established and not diminished due to high water levels or pest (muskrats, beavers) activity.

4.2.5.5 The oxidation pond should be periodically cleaned out to remove Fe precipitates. These precipitates, if allowed to accumulate to over about 60% of the pond capacity, will spill over into the downstream wetland cells and impair the wetland's ability to further improve water quality.

4.2.5.6 The seep at the south end of the ALD and the artesian conditions in Well 10 should be monitored. In the event that the seepage and artesian flow need to be managed to prevent discharge into the intake channel, additional ALDs and/or constructed wetlands and diversion channels routing the flow to KIF006 should be considered as a potential solution.

5.0 Appendix:

5.0.1 Cost-benefit spreadsheet

6.0 References:

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6. Rastogi, V. 1990. Quote to the TVA for Treatment of the Bottom Ash Canal Area, Kingston Steam Plant, Kingston, TN. Quote No. 052290-TNTVA1. he BFGoodrich Company. Akron, OH.

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8. American Public Health Association. 1992. Standard Methods for the Examination of Water and Wastewater. Washington, DC.

9. Pope, R. L. 1995. Estimated Operation and Maintenance Costs Associated with the KIF006 Constructed Wetland. Kingston Fossil Plant. Personal Communication. Kingston, TN.

	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R	S	T
1			**Technology Advancements**															rev 4e 6/3/98		
2	**Cluster:**	Environmnental Control							R&D Project		R&D Success		Implementation		Royalty			Expected		
3									Cash Flow (000's)		Cash Flow (000's)		Cash Flow (000's)		Cash Flow (000's)			Cash Flow (000's)		
4	**Project Name:**		Kingston FP Anoxic Limestone Drain Modification					**Year**	Costs	Benefits	Costs	Benefits	Costs	Benefits	Costs	Benefits	Costs	Benefits	Total	
5								1994	(46)	50			$0	$0	$0	$0	($46)	$50	$4	
6	**Project No.**		RG228		**Date:**	**9/8/98**		1995	(15)	52			$0	$0	$0	$0	($15)	$52	$37	
7								1996			(2)	53	$0	$0	$0	$0	($2)	$50	$48	
8	**Short Code**							1997			(2)	55	$0	$0	$0	$0	($2)	$52	$50	
9								1998			(2)	56	$0	$0	$0	$0	($2)	$53	$51	
10	**Project Manager**		G. A. Brodie					1999			(2)	358	$0	$0			($2)	$340	$338	
11								2000			(2)	60	$0	$0			($2)	$57	$55	
12	**Benefit Category:**		Debt		(enter Debt Reduction or Sales Growth)			2001			(2)	62	$0	$0			($2)	$59	$57	
13								2002			(2)	64	$0	$0			($2)	$60	$58	
14								2003			(2)	66	$0	$0			($2)	$62	$60	
15	**Economic Assumptions:**							2004			(3)	68	$0	$0			($2)	$64	$62	
16	**Discount Rate =**		15%		**In Terms of**	**In Terms of**	**In Terms of**	2005			(3)	70	$0	$0			($2)	$66	$64	
17	**Inflation Rate =**		3%		**Prioritization**	**Origination**	**Current**	2006			(3)	72	$0	$0			($3)	$68	$66	
18					**Year:**	**Year:**	**Year:**	2007			(3)	74	$0	$0			($3)	$70	$67	
19			**Enter year here:**		**1999**	**1994**	**1998**	2008			(3)	76	$0	$0			($3)	$72	$69	
20			**PV Costs:**		N/A	($70)	**$ (122)**													
21	**Term=**	15	**PV Benefits:**		N/A	$515	**$ 901**													
22			**NPV:**		$626	$446	**$ 780**													
23			**IRR:**		#NUM!	#DIV/0!														
24	**Annual Equivalent Annuity:**				$101	$72														
25	**Present Value of Perpetuity:**				$673	$479														
26	**Benefit/cost ratio:**				N/A	7.41														
27	**Assumptions**																			
28	1) The Probability of Technical Success (%) =						95%													
29	2) The Probability of TVA's Commercialization/Implementation of this Technology =						100%													
30	3) The Probability of Royalties if TVA Implements this Technology =						0%													
31	4) The Probability of Royalties if TVA Does Not Implement this Technology =						0%													
32	5) Costs and revenues due to R&D, implementation by TVA and royalties																			
33	are shown separately and at the appropriate time of occurance.																			
34	6) No probabilities have been incorporated into the cash flows.																			
35																				
36						**NPV's based on Prioritization Year**			**NPV:**	$0	**NPV:**	$659	**NPV:**	$0	**NPV:**	$0	**Total Prioritization NPV:**		$626	
37	Project Assumptions:					(doesn't consider sunk costs)											**Prioritization IRR:**		#NUM!	
38	1)																			
39	2)																			
40	3)																			

Cell: R1
Note: rev 4c 3/2/98 - made minor changes to cells on annual equivalent annuity calculations so value would always be calculated based on discount rate.

rev 4d 3/23/98 - Corrected annual equivalent annuity calculations; added Present Value of Perpetuity; and used column T to determine appropriate term for annuity calculations.

rev 4e 6/3/98 - Changed years to 1999. Made IRR guess in cell P37 invisible when printed.

Cell: I5
Note: Cost for design and installation of new ALD repairs.

Cell: J5
Note: Eliminated costs for O&M of pump station (new pump impellors, new pump, electricity, labor and supervision).

Cell: I6
Note: Cost for performance monitoring.

Cell: L10
Note: Avoided cost of O&M ($58K) plus avoided cost of design and installation of a conventional treatment system = $300K.

Cell: E22
Note: Value ignores cash flows prior to prioritization year.

Cell: P37
Note: The guess for IRR calculation (required) is stored here. Made invisible so it will not print.

ATTACHMENT C – SITE MONITORING PLAN

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15 May 2012

Mr. Jamey Dotson
Senior Program Manager
Tennessee Valley Authority

Subject: Appendix C - Site Monitoring Plan for East Dike
TVA Kingston Fossil Plant
Kingston, TN

Dear Mr. Dotson:

This document was prepared by Geosyntec Consultants (Geosyntec) under authorization from the Tennessee Valley Authority (TVA) to provide a Site Monitoring Plan for the East Dike (SMP-ED) at the Kingston Fossil Plant (KIF), Kingston, Tennessee. The East Dike comprises an area located adjacent to the Intake Channel on the southwestern portion of the KIF. This SMP-ED was specifically prepared to address seepage and water level monitoring within the compacted earth slopes immediately adjacent to the Intake Channel. This SMP-ED was prepared to be a stand-alone document that is included as Appendix C in the May 2012 report titled *Supplemental Assessment of Seepage and Slope Stability, Kingston Fossil Plant, East Dike,* (East Dike Stability Report). This SMP-ED was prepared by Dr. Robert C. Bachus, P.E. and Will Tanner, P.E., of Geosyntec as authorized by Mr. Vernon J. Dotson, P.E., of TVA.

BACKGROUND

While addressing the rehabilitation of compacted earth dikes at the perimeter of the existing Ash Pond at KIF, TVA was requested by the U.S. Environmental Protection Agency (USEPA) and the U.S. Bureau of Reclamation (USBR) to assess slope stability of the East Dike. Historically, seepage along the toe of the East Dike had been noted during TVA's site inspections of the KIF. USEPA and USBR requested that TVA perform analyses and provide site-specific recommendations for the East Dike. Geosyntec met with TVA personnel on several occasions to understand the construction and work activities related to the East Dike and performed a series of preliminary analyses in both the southern and northern portions of East Dike. Results of these meetings and analyses resulted in: (i) performance of subsurface geotechnical investigations to assess subsurface stratigraphy and engineering characterization; and (ii) installation of a series of piezometers to assess the water levels within the East Dike slopes. Details of the various activities related to the East Dike and results of the analyses are included in the referenced East Dike Stability Report. Important highlights from the East Dike Stability Report that influence recommendations provided in this SMP-ED are discussed in the remainder of this section of Appendix C.

Observed Seepage at the East Dike

The outboard slopes along the toe of the East Dike adjacent to the Intake Channel are extensively vegetated with native species and seepage at the toe of the slope had been noted and monitored by TVA for several years. There have been no indications of piping or internal erosion as a result of the observed seeps.

In addition, high water levels and local seeps have been reported in the slopes above the Perimeter Access Road (i.e., well above the toe of the slope adjacent to the Intake Channel). These seeps are locally collected in a ditch adjacent to the Perimeter Access Road and directed towards the north into the Red Water Pond.

Anoxic Limestone Drain

To provide treatment of low-pH impacted groundwater in the vicinity of the East Dike, TVA installed an Anoxic Limestone Drain (ALD) that acts as a shallow French Drain in the slopes above the Perimeter Access Road. Recent analytical test results indicate that the ALD may not be functioning as intended from a water treatment perspective; therefore TVA is currently assessing potential alternatives to provide the groundwater treatment intended by the ALD. The referenced seepage above the Perimeter Access Road appears to be related to the location and action of the ALD.

Subsurface Stratigraphy and Measured Groundwater Levels

Results of a subsurface exploration program including the advancement of several piezocone penetrometer test (CPTu) soundings were used to confirm information provided in the historic design drawings for the KIF. These drawings show construction details of the earth dike comprising the East Dike. This dike apparently was constructed to provide closure to the entrance of the former Swan Pond Creek and allow water to impound within the vicinity of the current Intake Channel. The results of this exploration program provided site-specific quantitative information that was used for the stability and seepage analyses.

Response of the numerous piezometers installed within the East Dike indicates relatively high piezometric heads in the area behind and within the East Dike. At times, local artesian water pressures were noted in the slopes below the Perimeter Access Road. When coupled with the previously referenced stratigraphy, these high water levels appear to be due to the presence of a granular layer beneath the East Dike.

Source of Groundwater Recharge

Review of the results from Geosyntec's Draft version of the *Kingston Ash Recovery Project – Groundwater Flow and Transport Model Report*, dated 25 July 2011 indicate local recharge effects are primarily from: (i) the Rail Yard on the western side of the Ballfield Site; and (ii) the Sluice Channel on the eastern side of the Ballfield Site. There appears to be limited groundwater recharge from the Ballfield Site itself, the former Rim Ditch located east of the Sluice Channel, and the Dredge Cell located to the north and west of the East Dike. Local groundwater recharge and the measured high water levels in the vicinity of the East Dike are believed to be attributed to the zones of coarse-grained bottom ash near the southern end of the Sluice Channel and the (implied) presence of coarse materials used in the historic closure of the former Swan Pond Creek during construction of the Intake Channel. TVA reports that as part of planned modifications to the KIF, the Sluice Channel will eventually be decommissioned, thus reducing this significant source of recharge. When this occurs, the water levels in the East Dike are anticipated to reduce relative to the current levels.

Slope Stability Calculation Results

As presented in the East Dike Stability Report, results of slope stability calculations performed along two representative cross sections through the East Dike indicate adequate calculated factors of safety (FS) values for both short- and long-term conditions, even in consideration of the relatively high water levels encountered in the vicinity of the East Dike. The slope stability calculations presented in the East Dike Stability Report were performed using the pore water pressures calculated during the seepage finite element analysis. Furthermore, there have been no reported local slope instability issues identified in the vicinity of the East Dike.

Seepage Calculation Results and Effects

Results of seepage analyses for this portion of the KIF indicate that measured high piezometric heads are primarily due to flow through the granular materials below the East Dike. The presence of observed seeps at the toe of the East Dike adjacent to the Intake Channel is believed to be due to the lack of a toe drain during initial/historic East Dike construction and this underlying drainage pathway afforded by the coarse granular layer beneath the East Dike. As presented in the East Dike Stability Report, the calculated seepage velocities are relatively small and are not anticipated to be sufficient to adversely impact the potential for internal piping. These results also indicate that the calculated exit gradients, while in some locations do not meet the guidelines presented in the TVA Programmatic Document, are within the guidelines recommended in the U.S. Army Corps of Engineers (USACE) *Seepage Analysis and Control for Dams* [USACE 1986].

Conclusions and Recommendations

Based on these observations and calculation results, Geosyntec does not believe it necessary to implement any rehabilitation construction measures at the East Dike to increase stability or reduce seepage; particularly in view of the fact that future modifications to the Sluice Channel will likely improve the situation. However, these same observations and calculation results contribute to Geosyntec's recommendation to implement a performance monitoring strategy specific to the East Dike. Recommendations regarding the strategy for local monitoring of conditions along the East Dike, including the location of monitoring points, monitoring frequency, trigger levels, and response actions are presented in the remainder of this document.

MONITORING STRATEGY

The results of the observations and analyses presented in the East Dike Stability Report indicate a low potential for slope instability in the vicinity of the East Dike, even under the current conditions of high piezometric water levels. Therefore, Geosyntec does not believe it necessary to install slope inclinometers or survey monuments to monitor potential slope movements. Furthermore, results of seepage analyses do not indicate a significant potential for piping and the related progressive instability. Nevertheless, Geosyntec acknowledges the locally high piezometric water levels and the local seeps that have been confirmed along the alignment of the East Dike. Based on these conclusions, Geosyntec recommends a "monitoring only" strategy for the East Dike. Geosyntec recommends that the monitoring efforts focus on the high water levels and the observed seeps. Specific recommendations for these monitoring activities follow.

Monitoring Stations

Due to proximity to the Perimeter Access Road and the historical data of the twelve existing standpipe piezometers, Geosyntec recommends monitoring one of the existing piezometers along each of the four analyzed cross-sections (i.e., A through D) on a routine basis. The location of each piezometer is identified on the Monitoring Station Location Plan presented in Figure A-1. These locations are easily accessible and represent areas where the piezometric heads are relatively high. By monitoring the four piezometers, a certain degree of redundancy is afforded. To increase redundancy and to provide data that may be beneficial in the event of an exceedance of a trigger level, it is recommended that all of the current standpipe piezometers be maintained and available for inclusion in the monitoring program as needed.

With regards to the seeps, they can easily be inspected visually and significant changes can be noted. To provide quantitative data that compliments the visual inspection records, Geosyntec recommends the following: (i) installation of a V-notch weir (Weir 1) in the drainage ditch adjacent to the Perimeter Access Road near cross section B; (ii) monitoring of the existing 8" HDPE pipe (Subgrade Pipe) that conveys the water in the drainage ditch underneath the gravel driveway between cross sections B and C; (iii) installation of a second V-notch weir (Weir 2) in the drainage ditch adjacent to the wetlands; and (iv) installation of a third V-notch weir at the toe of the East Dike located in the active seepage area in between cross sections A and B (Weir 3). The locations of these flow measuring devices are shown in Figure A-1. Furthermore, Figure A-1 will be updated to show the location of Weir 3 once it is installed in the field.

Monitoring Frequency

The existing piezometers will be manually recorded and the data entered into a site-specific database. Initially, as a base level monitoring frequency Geosyntec recommends that daily readings be obtained at one piezometer per analyzed cross section (i.e., A through D). In addition it is recommended that readings be obtained at all twelve piezometers once per week. This data will be compiled and presented on a time trend plot to facilitate review. Furthermore, the data from two piezometers from the former ball field site to the west of the East Dike will be included in the East Dike database to determine if a site-wide trend can be established.

Geosyntec recommends that initially, Weirs 1 through 3 and the Subgrade Pipe be monitored on a daily basis. Weirs 1 through 3 and the Subgrage Pipe will be calibrated so that flow rates can be calculated based on the height of the water flowing through the device.

Once a sufficient database of quantitative flows has been developed, it will be possible to reassess the monitoring frequency of Weirs 1 through 3 and the Subgrade Pipe and reduce it if warranted.

Trigger Levels and Response Actions

As stated in the background section of this SMP-ED, neither the current water levels nor the historic seeps in proximity to the East Dike have presented stability or piping issues. Therefore, this monitoring strategy was developed to establish quantitative site-specific information that can be used to better characterize and "calibrate" the visual observations. The best value of these quantitative results will be provided by establishing baseline records and correlating these records to visual observations of performance. Nevertheless, Geosyntec recognizes the value in establishing trigger levels and appropriate responses to observed and/or measured exceedances.

Piezometer Trigger Levels and Responses
Two significant observations are noted when considering trigger levels for the piezometers: (i) for the limited time period when piezometric heads have been measured, the response of each piezometer seems to be within a relatively narrow range; and (ii) overall groundwater levels at the site seem to have a have a “systematic” trend and shift, implying that regional groundwater levels cause a relatively consistent rise in all piezometers. Therefore, trigger levels need to be established to reflect increases that exceed the “systematic” response.

Therefore in order to develop the trigger levels, Geosyntec has performed a series of sensitivity analyses. The analyses demonstrate the effects of an increase in water levels on the global stability factor of safety using the topography, stratigraphy, material strength parameters and software package described in the East Dike Stability Report. The results of these sensitivity analyses are given in Figures A-2 through A-9 and are summarized in Table A-1. The remainder of this section includes: (i) a brief discussion of each cross section; and (ii) the recommended response if a trigger level is exceeded.

For cross section A, the existing condition is shown in Figure A-2. In this condition the groundwater table is found approximately 0.5 ft to 1.0 ft below ground surface, prior to day lighting as seepage near the toe of the slope. Additionally in the existing conditions, piezometer A-3 (screened in the Lower Dike Fill) has historically indicated a maximum water pressure of approximately 1 ft above the ground surface. This elevated piezometric head was modeled with a piezometric line, which was applied to the Lower Dike Fill and all materials beneath this layer. The remaining material layers were modeled with the typical groundwater table. The calculated global stability factor of safety for the existing baseline condition is 1.67. Two sets of sensitivity analyses were modeled for cross section A to demonstrate the sensitivity of the calculated global stability FS with respect to the groundwater pressure increases relative to the existing condition. In the first set the groundwater surface was elevated from existing conditions of approximately 0.5 ft to 1.0 ft below the ground surface to the level of the ground surface, which represents a condition of groundwater seepage occurring from the elevated haul road to the toe of the East Dike. For this condition the global stability failure surface extends through the Upper Dike Fill layer only and the factor of safety was found to be 1.33 as shown in Figure A-3. However, it was found that by introducing a value of only 10 psf for the cohesion of this layer the factor of safety increased to 1.58 as seen in Figure A-4. This amount of cohesion is likely available since the material in the Upper Dike Fill contains a significant amount of fine-grained material. For the second set of sensitivity analyses on cross section A, the water pressure in the Lower Dike Fill was elevated above the level modeled in existing conditions until the critical failure surface passes through the Lower Dike Fill and the predicted factor of safety was recorded below the target value of 1.5. It is noted that the existing groundwater table was considered for this set of sensitivity analyses. The results of this series of sensitivity analyses indicate that if the water pressures measured by piezometer A-3 approach 3 ft above the ground surface, then the global stability factor of safety drops below the long-term target value of 1.5, as provided in Figure A-5. It is noted that historically these levels have not been measured by piezometer A-3.

For cross section B, the existing condition is shown in Figure A-6. In this condition the groundwater table is approximately 1.0 ft to 4.0 ft below ground surface prior to day lighting as seepage near the toe of the slope. Additionally in the existing conditions, piezometer B-3 (screened in the Lower Dike Fill) has historically indicated a maximum water pressure of approximately 1.5 ft above the ground surface. This elevated piezometric head was modeled with a piezometric line, which was applied to the Lower Dike Fill and all materials beneath this layer. The remaining material layers were modeled with the typical groundwater table. The calculated global stability factor of safety for the existing condition is 1.58. Two sets of sensitivity analyses were modeled for cross section B to demonstrate the sensitivity of the calculated global stability FS with respect to the groundwater pressure increases relative to the existing condition. In the first set the groundwater surface was elevated from existing conditions of approximately 1.0 ft to 4.0 ft below the ground surface to the level of the ground surface, which represents a condition of groundwater seepage occurring from the elevated haul road to the toe of the East Dike. For this condition the global stability failure surface extends through the Upper Dike Fill layer only and the factor of safety was found to be 1.42 as shown in Figure A-7. However, it was found that by introducing a value of only 10 psf for the cohesion of this layer the factor of safety increased to 1.58 as seen in Figure A-8. This amount of cohesion is likely available since the material in the Upper Dike Fill contains a significant amount of fine-grained material. For the second set of sensitivity analyses on cross section B, the water pressure in the Lower Dike Fill was elevated above the level modeled in existing conditions until the critical failure surface passes through the Lower Dike Fill and the predicted factor of safety was recorded below the target value of 1.5. It is noted that the existing groundwater table was considered for this set of sensitivity analyses. The results of this series of sensitivity analyses indicate that if the water pressures measured by piezometer B-3 approach 4 ft above the ground surface, then the global stability factor of safety drops below the long-term target value of 1.5, as provided in Figure A-9. It is noted that historically these levels have not been measured by piezometer B-3.

For cross section C, the existing condition is shown in Figure A-10. In this condition the groundwater table is approximately 3.0 to 7.0 ft below the ground surface and becomes gradually shallower to the east until the water surface of the intake channel is reached. The calculated global stability FS for this condition is 2.74. The piezometers located at cross section C have not historically recorded water pressures in excess of hydrostatic pressure. Therefore the only sensitivity analysis performed for this cross section is an increase in the groundwater table to coincide with the ground surface which represents a condition of seepage extending from the wetlands area to the west of the access road down to the toe of the East Dike. If such seepage occurs, the calculated factor of safety for this condition is 2.34 as shown in Figure A-11. Historical data collected at piezometers C-1A, C-1B and C-2 indicate that the groundwater levels at cross section C range from 3.0 ft to 7.0 ft below the groundwater surface.

For cross section D, the existing condition is shown in Figure A-12. In this condition the groundwater table is approximately 2.0 to 7.0 ft below the ground surface and becomes gradually shallower to the east before reaching the intake channel. In addition, piezometer D-1B (screened in the Alluvial Silty Clay – approximately 40 ft below the ground surface) has historically indicated a maximum water pressure of 7.0 ft above hydrostatic pressure. This elevated piezometric head was modeled with a piezometric line, which was applied to the Alluvial Silty Clay. The calculated global stability factor of safety for the existing condition is 3.29. The first sensitivity analysis elevated the groundwater surface from existing conditions to the ground surface to represent a condition of groundwater seepage from the wetlands area to the west of the access road to the toe of the East Dike. If this seepage condition occurs, the calculated global stability factor of safety is 2.76 through the Lower Dike Fill as shown in Figure A-13. Fluctuations in the water pressure in the Alluvial Silty Clay layer were not found to influence the global stability factor of safety since the layer is relatively deep.

The elevation of the water levels measured in the piezometers for cross sections A, B, C and D that represent the trigger levels are summarized in Table A-2.

If the trigger levels summarized in Table A-2 are exceeded for cross sections A and B then the global stability factor of safety is at or slightly below the target required for long-term stability (FS = 1.5). Therefore Geosyntec recommends that if the trigger levels are exceeded, water level measurements for the A-series (A-1 through A-3) and B-series (B-1 through B-3) piezometers should be recorded and reported on a daily basis until water levels return to below the trigger level, at which point the baseline monitoring frequency may resume. Additionally, a thorough visual inspection of the East Dike should be performed near cross sections A and B for signs of increased seepage and/or instability.

If the trigger levels summarized in Table A-2 are exceeded for cross section C and D then the resulting condition will be seepage at the ground surface from the wetlands area to the intake channel. It is noted that the global stability sensitivity analysis does not indicate calculated factors of safety that are below the target value of 1.5 for this case. Rather, if these trigger levels are exceeded then the response should be a visual inspection of the East Dike for signs of turbid or rapid groundwater flow. During these visual observations, the baseline monitoring frequency of piezometers in the C-series (C-1A through C-2) and D-series (D-1A through D-2) should continue.

Seepage Trigger Levels and Responses

Currently, assessment of the seeps is based exclusively on visual observations. It has been noted that indications of internal piping has not been observed. Geosyntec anticipates that with the inclusion of Weirs 1 through 3 and the Subgrade Pipe, TVA will for the first time be able to provide a quantitative assessment of flow to correlate with the visual inspections. Furthermore, it will likely be possible to assess a potential cause/effect relationship between precipitation and/or site activities and seepage flow rate. Therefore, until a baseline condition is established, the proposed trigger and response action should consider both flow rate and turbidity.

If the flow rate visibly increases by more than approximately 25 percent, monitoring should include the length of the East Dike, with particular focus on establishing the cause of the increased seepage flow rate. In addition, if the increased flow causes a noticeable turbidity (or other early indication of internal piping), TVA Engineering should be notified and mitigation measures such as a geotextile filter and gravel blanket, as shown in Figure A-14, should be placed over the seep until further study can be implemented.

If the discharge from the seep is noted to be turbid, this may be an early indication of internal piping. Alternatively, the increased turbidity could be caused by surface erosion. Regardless, Geosyntec recommends that these areas be identified for future rehabilitation that will likely include local construction of a blanket drain and filter, or establishment of surface erosion control features (as appropriate). Regardless, as mentioned above, TVA Engineering should be notified and temporary rehabilitation measures should be implemented immediately.

CLOSURE

This SMP-ED was developed to address the finding from site-specific observations and calculation results along the East Dike of the KIF. Geosyntec believes that a “monitoring only” strategy for this area of the KIF is warranted based on extensive exploration and characterization testing, review of historic site documents and observations, and TVA’s near-term goal of reducing infiltration in the areas immediately west of the East Dike. Once a quantitative baseline condition and time trend assessment is established, Geosyntec believes that it may be possible to adjust the scope and frequency of the monitoring program.

Sincerely,

Will Tanner, P.E.
Project Engineer

Robert C. Bachus

Robert C. Bachus, P.E., Ph.D.
Principal

Attachments: Tables
Figures

TABLES

Table A-1. Summary of Sensitivity Analysis Results

Cross Section	Analysis Condition	Global Stability FS	Figure
A	Existing Conditions	1.67	A-2
A	Full Seepage Slope	1.33	A-3
A	Full Seepage Slope + c = 10 psf(1)	1.58	A-4
A	Elevated Water Pressure in LDF(2)	1.47	A-5
B	Existing Conditions	1.58	A-6
B	Full Seepage Slope	1.42	A-7
B	Full Seepage Slope + c = 10 psf(1)	1.47	A-8
B	Elevated Water Pressure in LDF(2)	1.50	A-9
C	Existing Conditions	2.74	A-10
C	Full Seepage Slope	2.37	A-11
D	Existing Conditions	3.29	A-12
D	Full Seepage Slope	2.76	A-13

Notes:

(1) The failure surface in this case passes through the Upper Dike Fill which consists of a mixture of fine and coarse grained soils as indicated during the laboratory tests performed during the initial South End Study (2010) and North End Study (2010).

(2) Water pressures in the Lower Dike Fill (LDF), as measured by piezometers A-3and B-3 have historically been slightly above hydrostatic pressure.

Table A-2. Summary of Piezometer Trigger Levels

Piezometer	Layer of Screen[1]	Trigger Level[2]	Trigger Level Elevation (ft)
A-1	Ash	Seepage at ground surface[3]	757.00
A-2	Upper Dike Fill	Seepage at ground surface[3]	755.00
A-3	Lower Dike Fill	Pressurized to ground surface + 3ft	750.10
B-1	Ash	Seepage at ground surface[4]	759.45
B-2	Lower Dike Fill	Seepage at ground surface[4]	753.20
B-3	Lower Dike Fill	Pressurized to ground surface + 4ft	752.50
C-1A	Lower Dike Fill	Seepage at ground surface	748.50
C-1B	Alluvial Sandy Silt	Seepage at ground surface	748.50
C-2	Lower Dike Fill	Seepage at ground surface	743.90
D-1A	Lower Dike Fil	Seepage at ground surface	748.70
D-1B	Alluvial Silty Clay	Seepage at ground surface	N/A[5]
D-2	Alluvial Clayey Silt	Seepage at ground surface	743.30

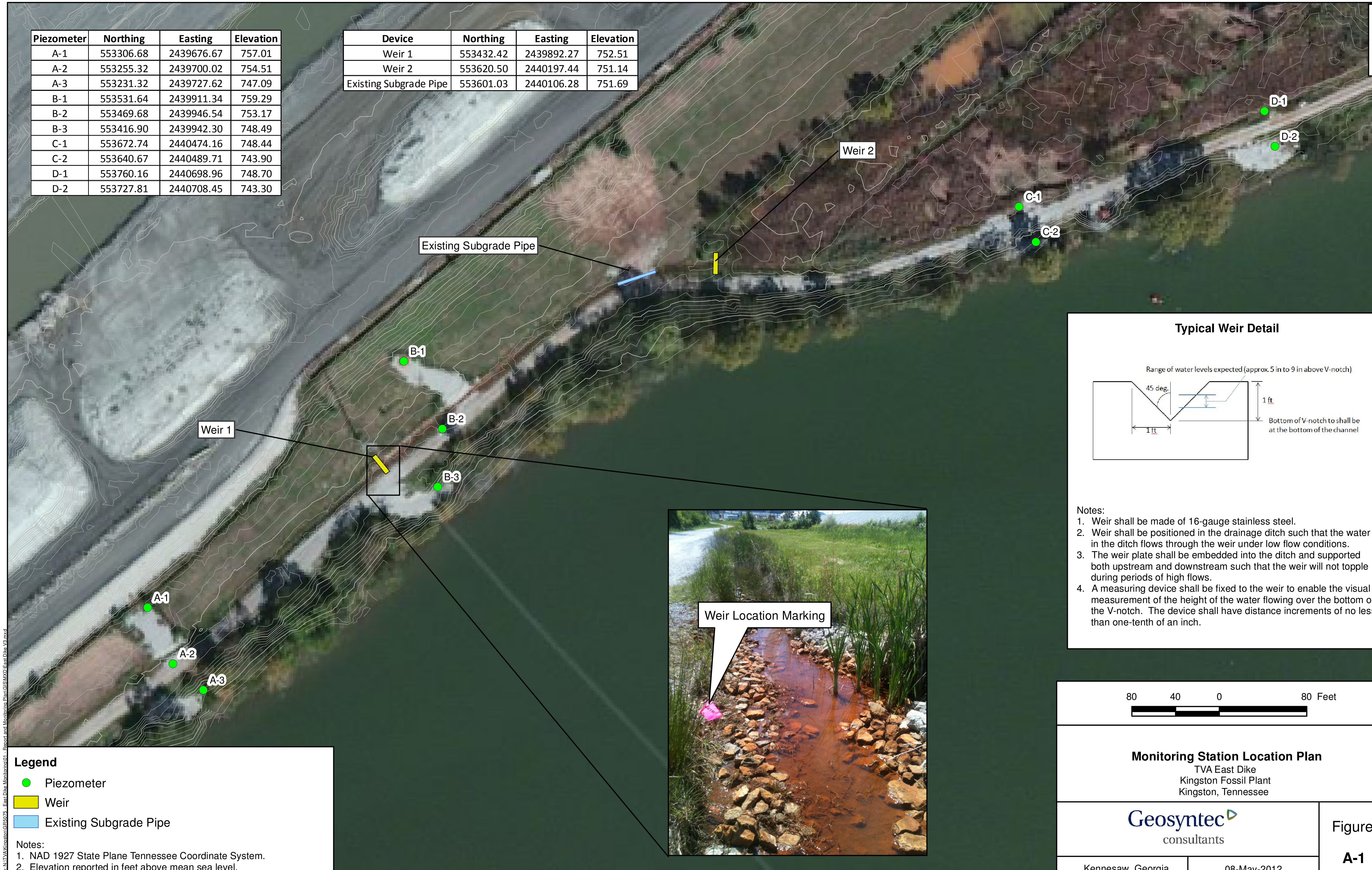
Notes:

(1) The layer of screen indicates the soil layer in which the piezometer is located, therefore the piezometers measure water pressures in the corresponding layer that is indicated.
(2) The trigger levels shown in this Table match the water levels modeled to obtain the global stability factor of safety shown in Table A-1.
(3) For cross section A under existing conditions, seepage is present only at the toe of the East Dike in the area of piezometer A-3.
(4) For cross section B under existing conditions, seepage is present only at the toe of the East Dike in the area of piezometer B-3.
(5) Water levels measured by this piezometer were not found to affect global stability due to the depth of the pressurized layer.

FIGURES

Piezometer	Northing	Easting	Elevation
A-1	553306.68	2439676.67	757.01
A-2	553255.32	2439700.02	754.51
A-3	553231.32	2439727.62	747.09
B-1	553531.64	2439911.34	759.29
B-2	553469.68	2439946.54	753.17
B-3	553416.90	2439942.30	748.49
C-1	553672.74	2440474.16	748.44
C-2	553640.67	2440489.71	743.90
D-1	553760.16	2440698.96	748.70
D-2	553727.81	2440708.45	743.30

Device	Northing	Easting	Elevation
Weir 1	553432.42	2439892.27	752.51
Weir 2	553620.50	2440197.44	751.14
Existing Subgrade Pipe	553601.03	2440106.28	751.69



Typical Weir Detail

Range of water levels expected (approx. 5 in to 9 in above V-notch)

45 deg.

1 ft

1 ft

Bottom of V-notch to shall be at the bottom of the channel

Notes:
1. Weir shall be made of 16-gauge stainless steel.
2. Weir shall be positioned in the drainage ditch such that the water in the ditch flows through the weir under low flow conditions.
3. The weir plate shall be embedded into the ditch and supported both upstream and downstream such that the weir will not topple during periods of high flows.
4. A measuring device shall be fixed to the weir to enable the visual measurement of the height of the water flowing over the bottom of the V-notch. The device shall have distance increments of no less than one-tenth of an inch.

Legend

- Piezometer
- Weir
- Existing Subgrade Pipe

Notes:
1. NAD 1927 State Plane Tennessee Coordinate System.
2. Elevation reported in feet above mean sea level.

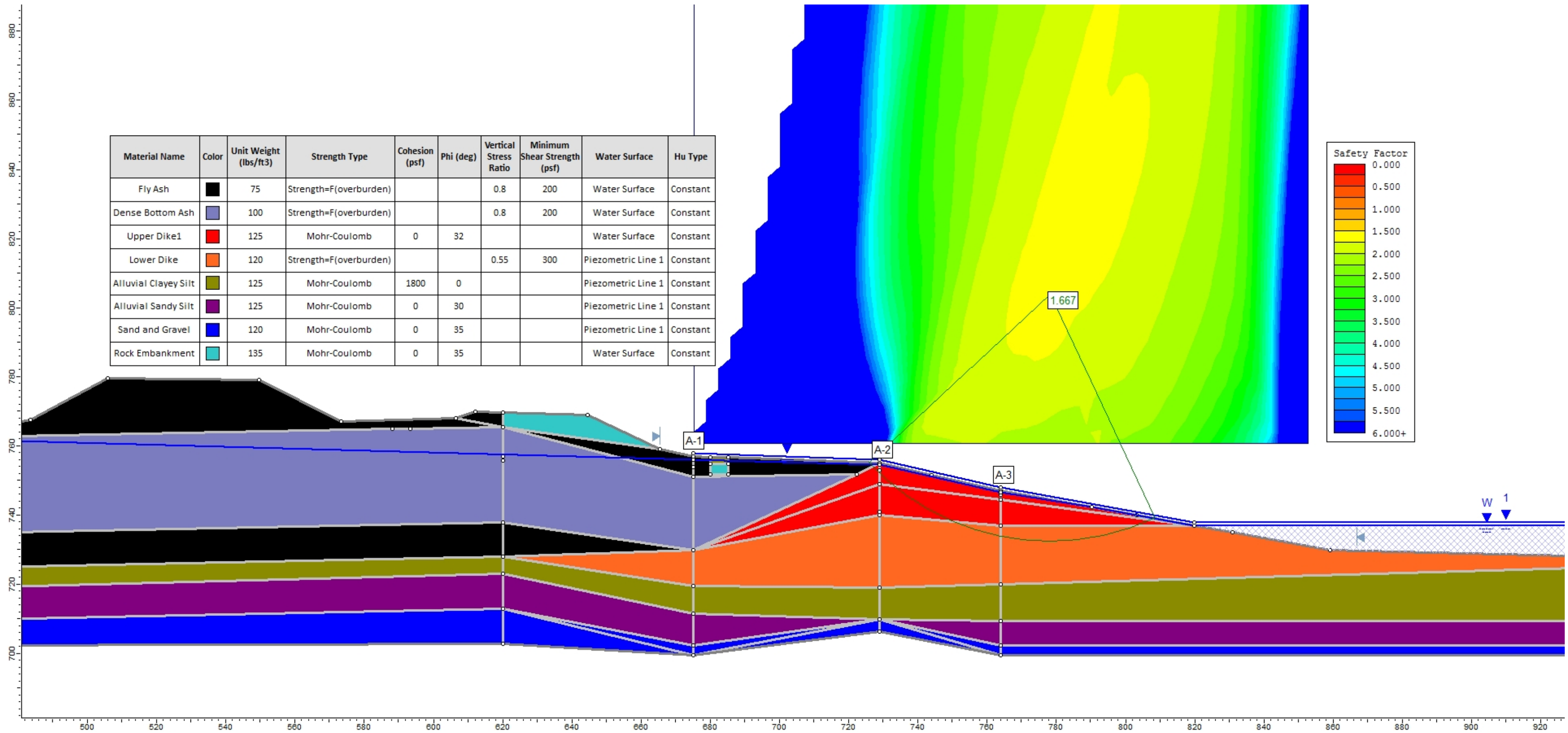
80 40 0 80 Feet

Monitoring Station Location Plan
TVA East Dike
Kingston Fossil Plant
Kingston, Tennessee

Geosyntec consultants

Kennesaw, Georgia | 08-May-2012

Figure A-1



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Vertical Stress Ratio	Minimum Shear Strength (psf)	Water Surface	Hu Type
Fly Ash		75	Strength=F(overburden)			0.8	200	Water Surface	Constant
Dense Bottom Ash		100	Strength=F(overburden)			0.8	200	Water Surface	Constant
Upper Dike1		125	Mohr-Coulomb	0	32			Water Surface	Constant
Lower Dike		120	Strength=F(overburden)			0.55	300	Piezometric Line 1	Constant
Alluvial Clayey Silt		125	Mohr-Coulomb	1800	0			Piezometric Line 1	Constant
Alluvial Sandy Silt		125	Mohr-Coulomb	0	30			Piezometric Line 1	Constant
Sand and Gravel		120	Mohr-Coulomb	0	35			Piezometric Line 1	Constant
Rock Embankment		135	Mohr-Coulomb	0	35			Water Surface	Constant

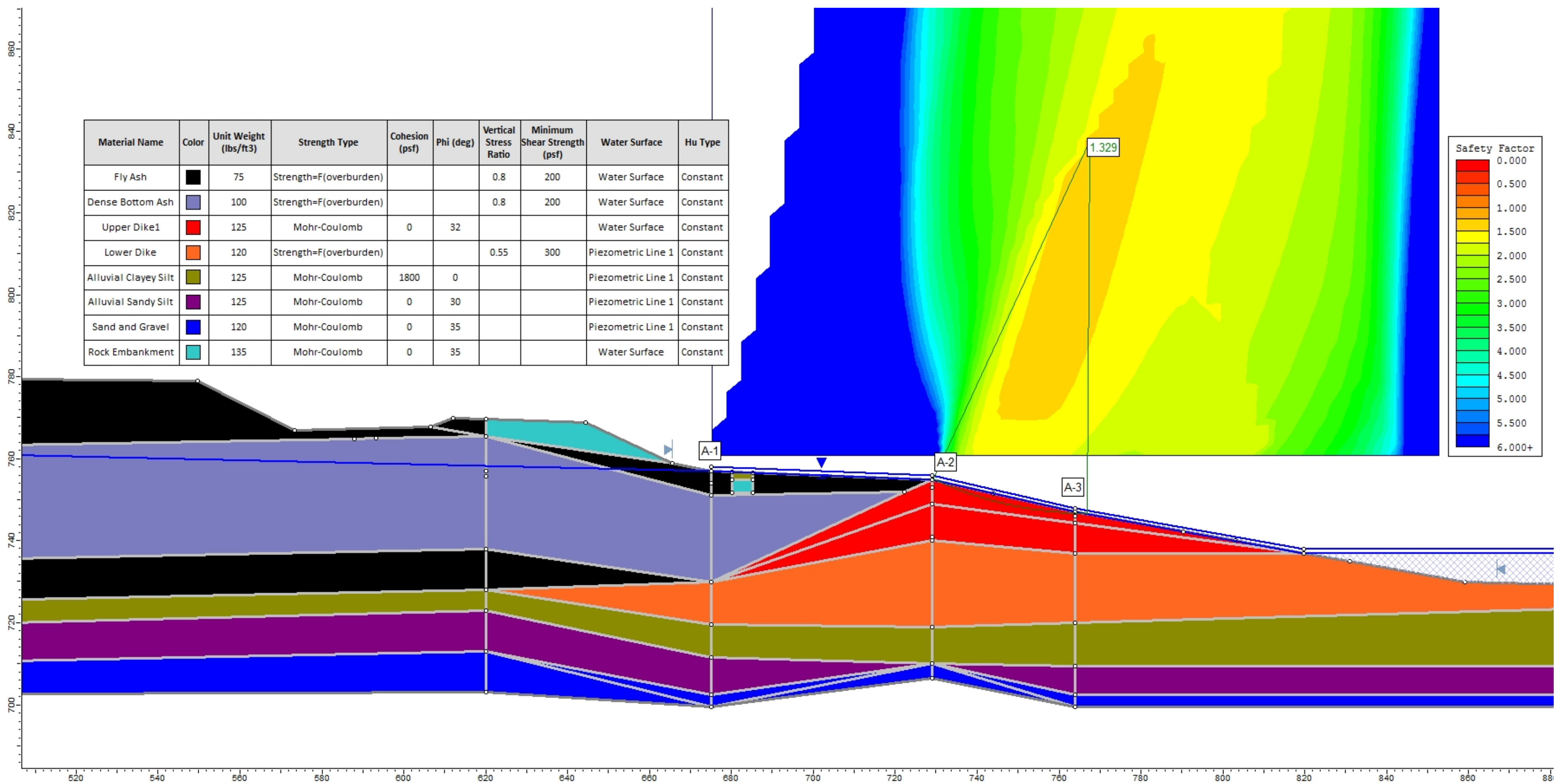
Global Stability Cross Section A - Existing Conditions

TVA East Dike, Kingston Fossil Plant, Kingston Tennessee

Geosyntec consultants

Figure **A-2**

Kennesaw, GA | 7 May 2012



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Vertical Stress Ratio	Minimum Shear Strength (psf)	Water Surface	Hu Type
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Dense Bottom Ash		100	Strength=F(overburden)			0.8	200	Water Surface	Constant
Upper Dike1		125	Mohr-Coulomb	0	32			Water Surface	Constant
Lower Dike		120	Strength=F(overburden)			0.55	300	Piezometric Line 1	Constant
Alluvial Clayey Silt		125	Mohr-Coulomb	1800	0			Piezometric Line 1	Constant
Alluvial Sandy Silt		125	Mohr-Coulomb	0	30			Piezometric Line 1	Constant
Sand and Gravel		120	Mohr-Coulomb	0	35			Piezometric Line 1	Constant
Rock Embankment		135	Mohr-Coulomb	0	35			Water Surface	Constant

Global Stability Cross Section A – Seepage Slope

TVA East Dike, Kingston Fossil Plant, Kingston Tennessee

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Figure **A-3**

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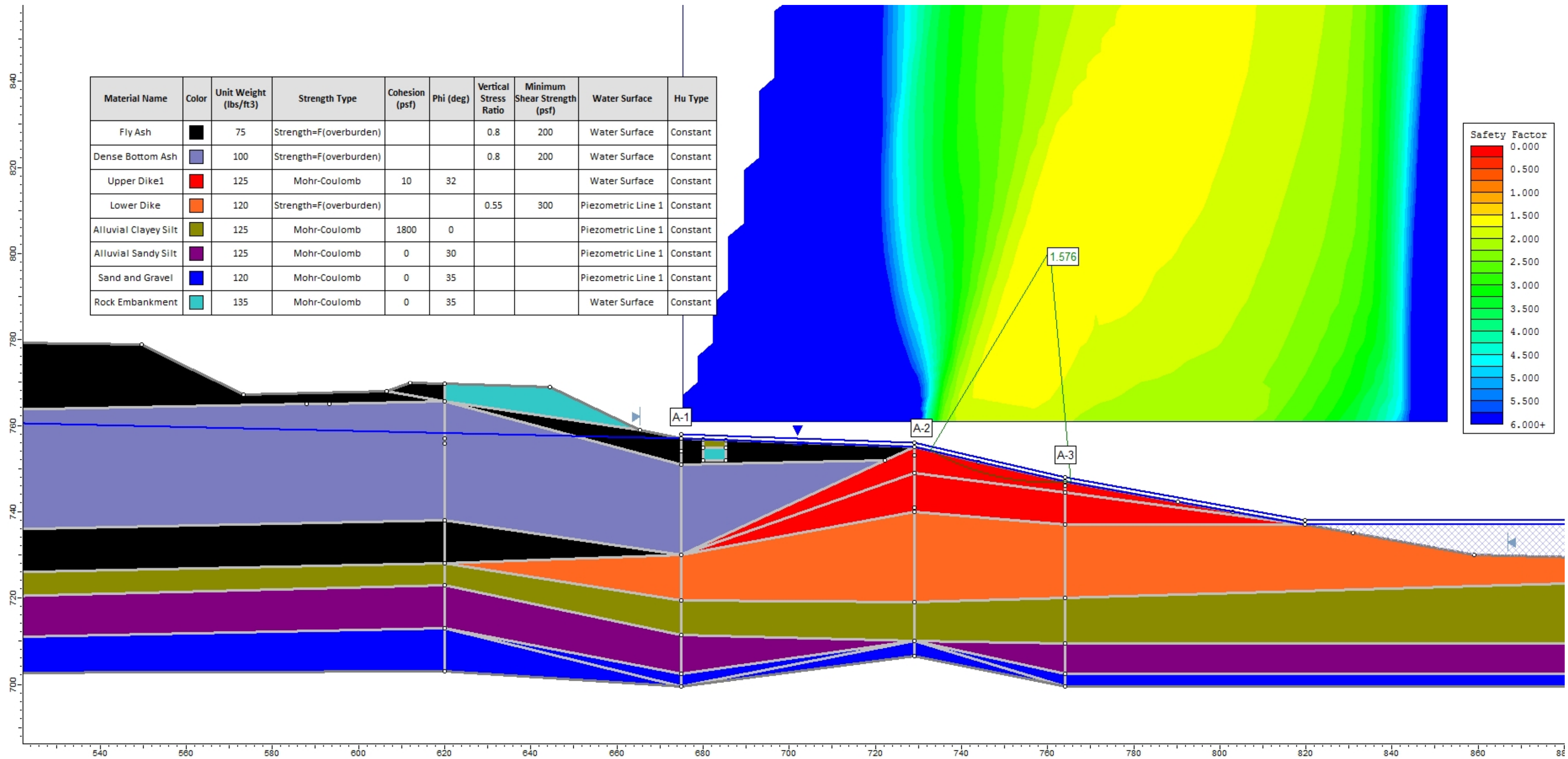
7 May 2012



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Vertical Stress Ratio	Minimum Shear Strength (psf)	Water Surface	Hu Type
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Upper Dike1		125	Mohr-Coulomb	10	32			Water Surface	Constant
Lower Dike		120	Strength=F(overburden)			0.55	300	Piezometric Line 1	Constant
Alluvial Clayey Silt		125	Mohr-Coulomb	1800	0			Piezometric Line 1	Constant
Alluvial Sandy Silt		125	Mohr-Coulomb	0	30			Piezometric Line 1	Constant
Sand and Gravel		120	Mohr-Coulomb	0	35			Piezometric Line 1	Constant
Rock Embankment		135	Mohr-Coulomb	0	35			Water Surface	Constant

Global Stability Cross Section A - Seepage Slope with c = 10 psf

TVA East Dike, Kingston Fossil Plant, Kingston Tennessee

Geosyntec consultants

Figure **A-4**

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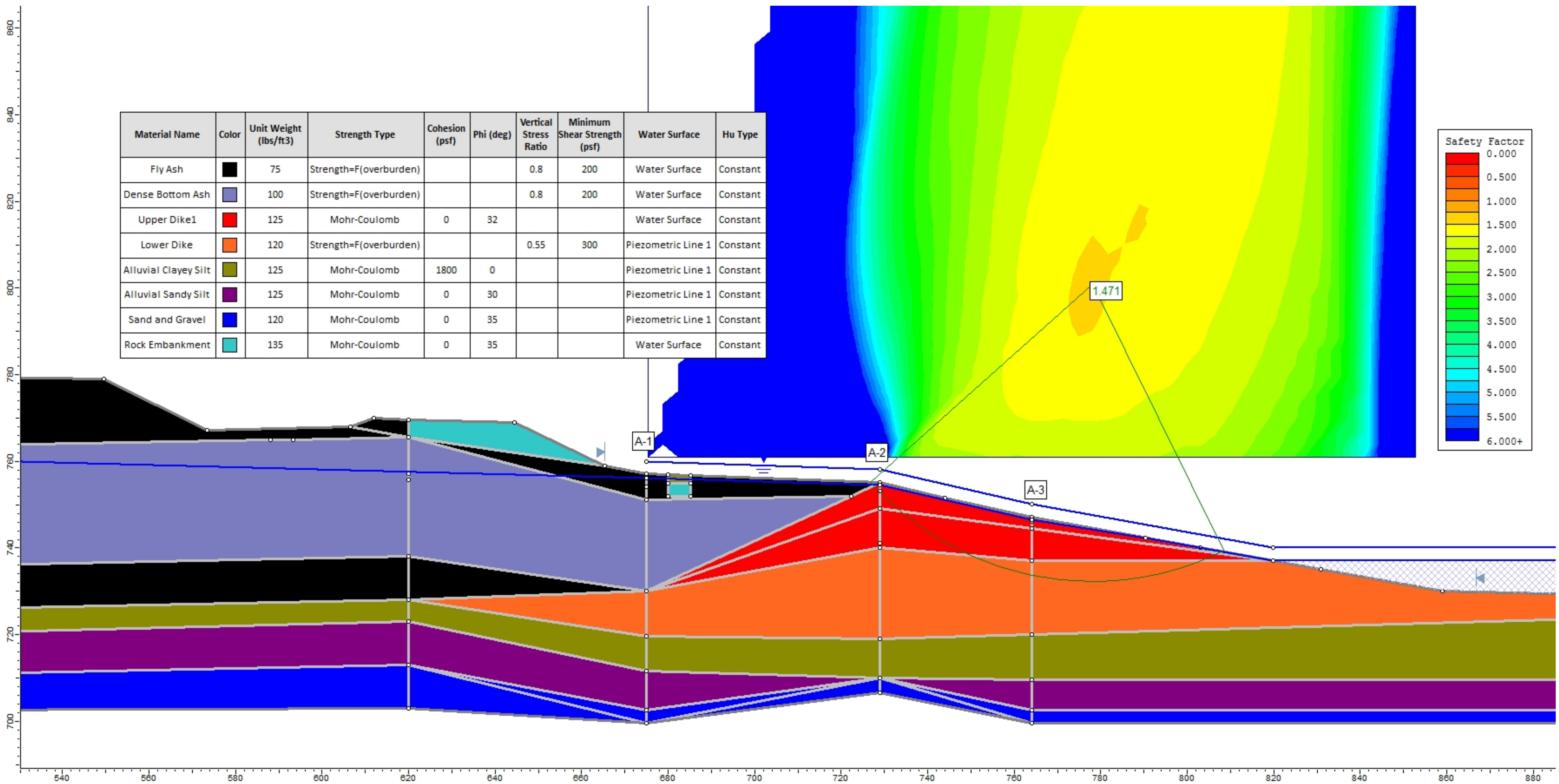

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Vertical Stress Ratio	Minimum Shear Strength (psf)	Water Surface	Hu Type
Fly Ash		75	Strength=F(overburden)			0.8	200	Water Surface	Constant
Dense Bottom Ash		100	Strength=F(overburden)			0.8	200	Water Surface	Constant
Upper Dike1		125	Mohr-Coulomb	0	32			Water Surface	Constant
Lower Dike		120	Strength=F(overburden)			0.55	300	Piezometric Line 1	Constant
Alluvial Clayey Silt		125	Mohr-Coulomb	1800	0			Piezometric Line 1	Constant
Alluvial Sandy Silt		125	Mohr-Coulomb	0	30			Piezometric Line 1	Constant
Sand and Gravel		120	Mohr-Coulomb	0	35			Piezometric Line 1	Constant
Rock Embankment		135	Mohr-Coulomb	0	35			Water Surface	Constant

Global Stability Cross Section A – Piezometric Line Elevated to Ground Surface + 3ft

TVA East Dike, Kingston Fossil Plant, Kingston Tennessee

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Figure

A-5

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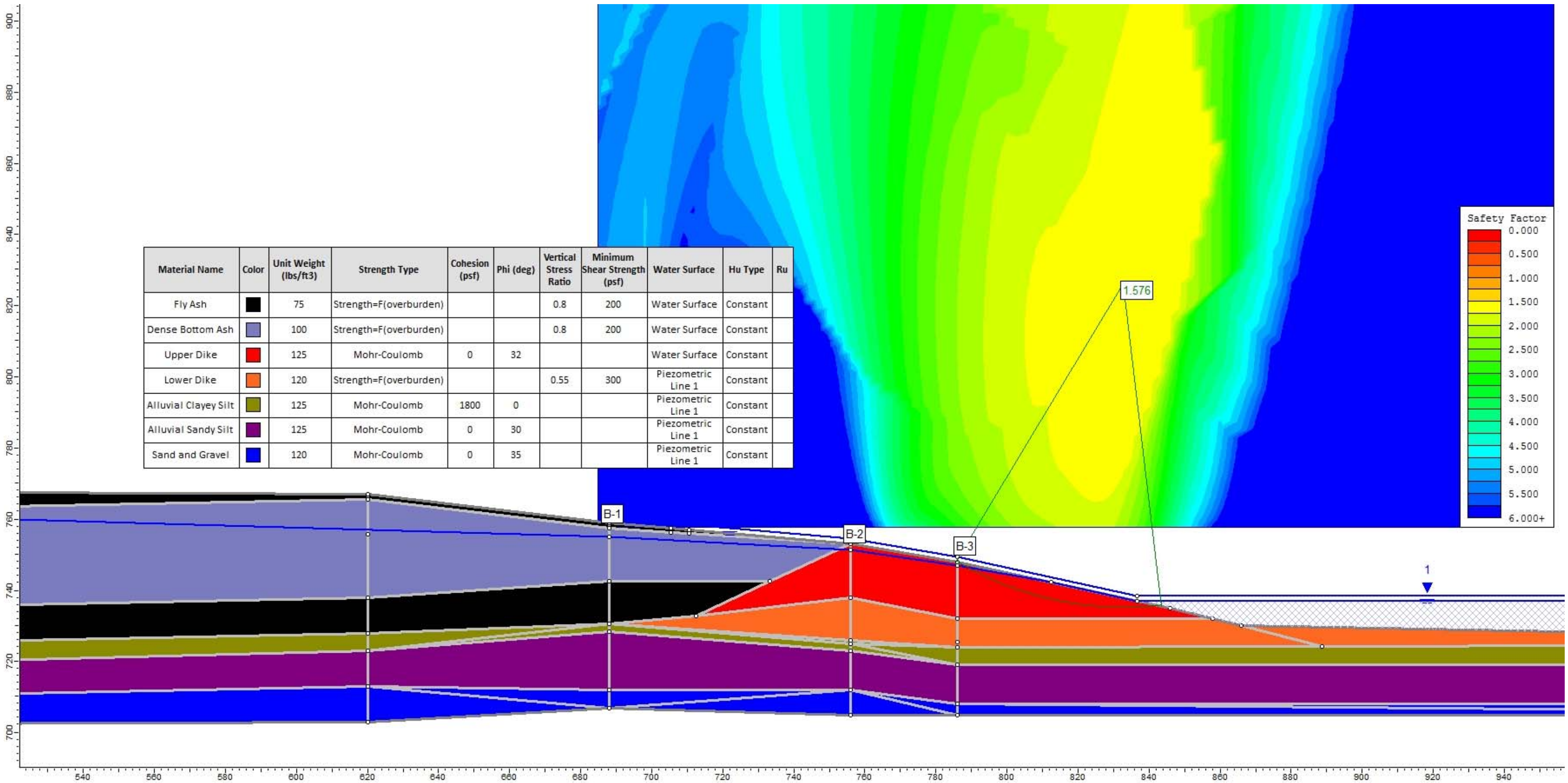

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Vertical Stress Ratio	Minimum Shear Strength (psf)	Water Surface	Hu Type	Ru
Fly Ash		75	Strength=F(overburden)			0.8	200	Water Surface	Constant	
Dense Bottom Ash		100	Strength=F(overburden)			0.8	200	Water Surface	Constant	
Upper Dike		125	Mohr-Coulomb	0	32			Water Surface	Constant	
Lower Dike		120	Strength=F(overburden)			0.55	300	Piezometric Line 1	Constant	
Alluvial Clayey Silt		125	Mohr-Coulomb	1800	0			Piezometric Line 1	Constant	
Alluvial Sandy Silt		125	Mohr-Coulomb	0	30			Piezometric Line 1	Constant	
Sand and Gravel		120	Mohr-Coulomb	0	35			Piezometric Line 1	Constant	

Global Stability Cross Section B – Existing Conditions

TVA East Dike, Kingston Fossil Plant, Kingston Tennessee

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Figure **A-6**

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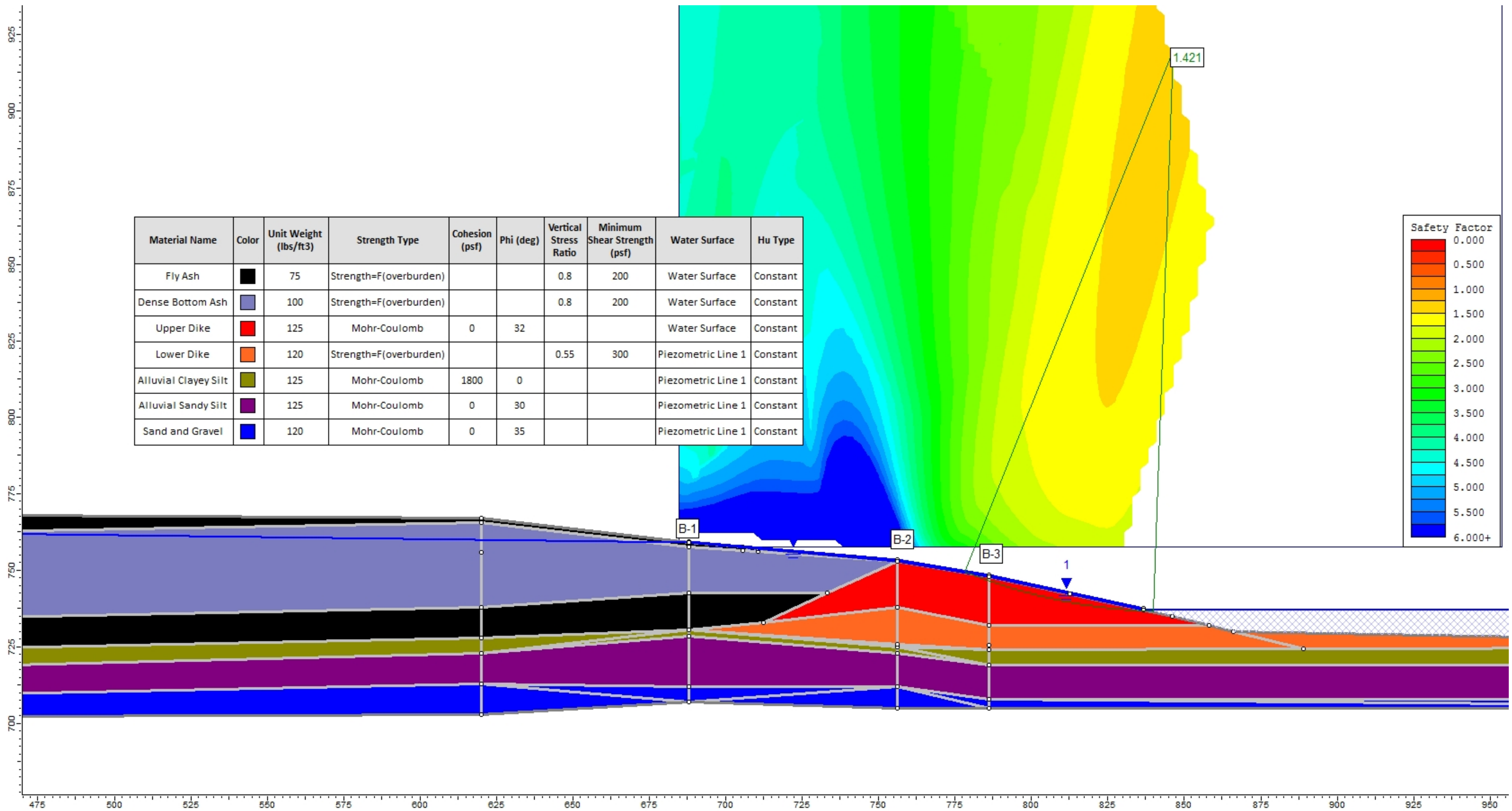

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Vertical Stress Ratio	Minimum Shear Strength (psf)	Water Surface	Hu Type
Fly Ash		75	Strength=F(overburden)			0.8	200	Water Surface	Constant
Dense Bottom Ash		100	Strength=F(overburden)			0.8	200	Water Surface	Constant
Upper Dike		125	Mohr-Coulomb	0	32			Water Surface	Constant
Lower Dike		120	Strength=F(overburden)			0.55	300	Piezometric Line 1	Constant
Alluvial Clayey Silt		125	Mohr-Coulomb	1800	0			Piezometric Line 1	Constant
Alluvial Sandy Silt		125	Mohr-Coulomb	0	30			Piezometric Line 1	Constant
Sand and Gravel		120	Mohr-Coulomb	0	35			Piezometric Line 1	Constant

Global Stability Cross Section B – Seepage Slope

TVA East Dike, Kingston Fossil Plant, Kingston Tennessee

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Figure **A-7**

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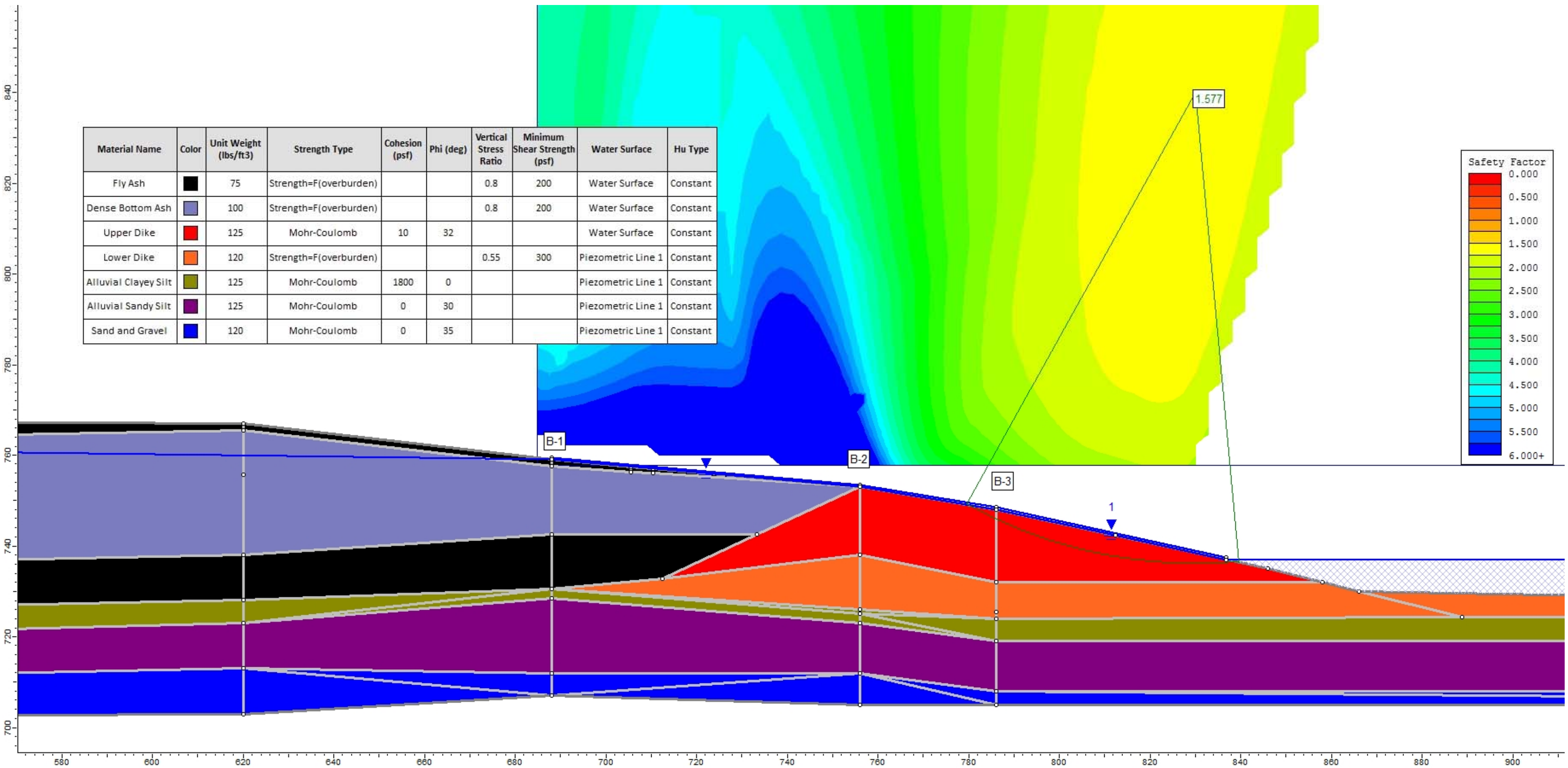

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Vertical Stress Ratio	Minimum Shear Strength (psf)	Water Surface	Hu Type
Fly Ash		75	Strength=F(overburden)			0.8	200	Water Surface	Constant
Dense Bottom Ash		100	Strength=F(overburden)			0.8	200	Water Surface	Constant
Upper Dike		125	Mohr-Coulomb	10	32			Water Surface	Constant
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Alluvial Sandy Silt		125	Mohr-Coulomb	0	30			Piezometric Line 1	Constant
Sand and Gravel		120	Mohr-Coulomb	0	35			Piezometric Line 1	Constant

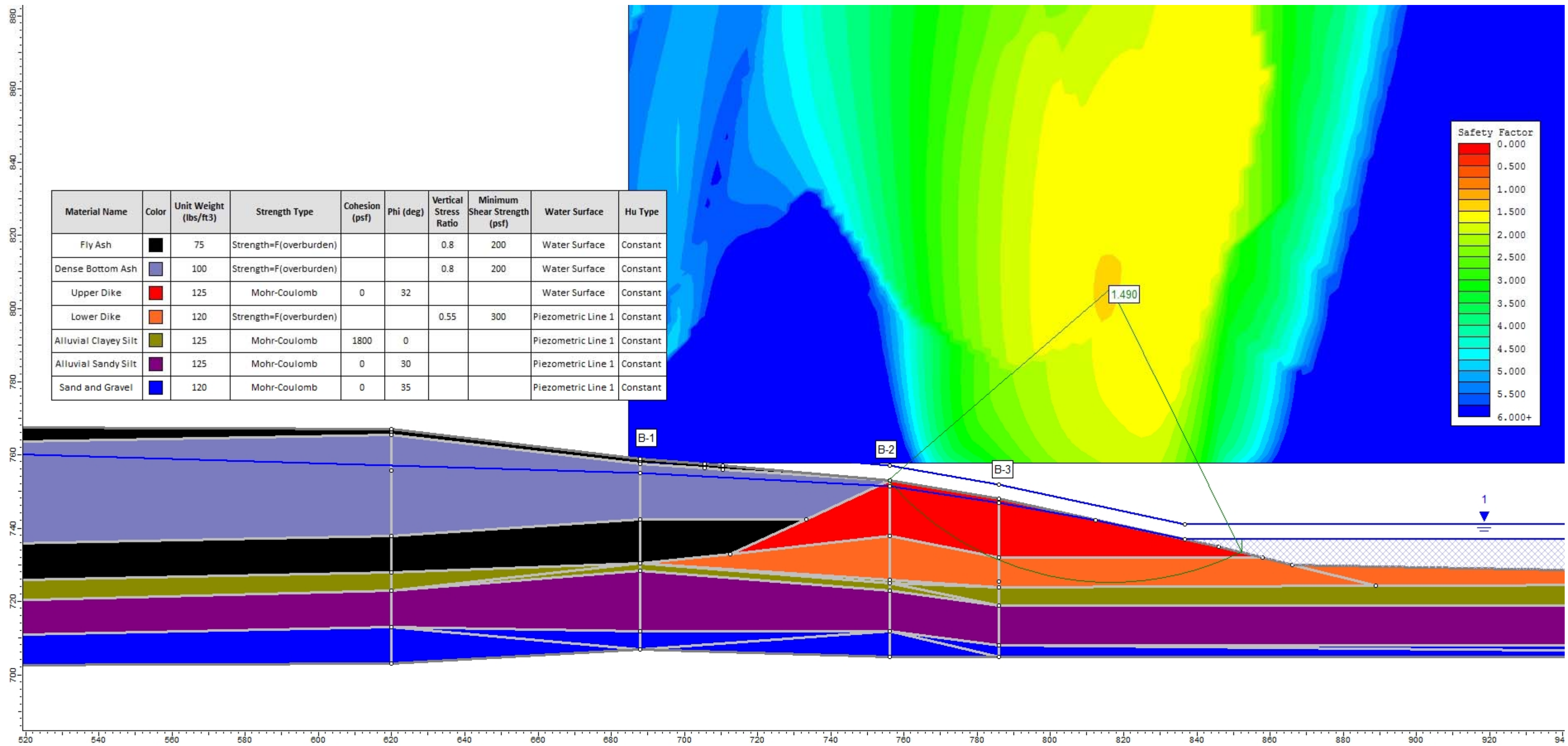
Global Stability Cross Section B – Seepage Slope with c = 10 psf

TVA East Dike, Kingston Fossil Plant, Kingston Tennessee

Geosyntec consultants

Figure **A-8**

Kennesaw, GA	7 May 2012



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Vertical Stress Ratio	Minimum Shear Strength (psf)	Water Surface	Hu Type
Fly Ash		75	Strength=F(overburden)			0.8	200	Water Surface	Constant
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Lower Dike		120	Strength=F(overburden)			0.55	300	Piezometric Line 1	Constant
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Alluvial Sandy Silt		125	Mohr-Coulomb	0	30			Piezometric Line 1	Constant
Sand and Gravel		120	Mohr-Coulomb	0	35			Piezometric Line 1	Constant

Global Stability Cross Section B – Piezometric Line Elevated to Ground Surface + 4ft

TVA East Dike, Kingston Fossil Plant, Kingston Tennessee

Geosyntec consultants

Figure

A-9

Kennesaw, GA | 7 May 2012

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Vertical Stress Ratio	Minimum Shear Strength (psf)	Water Surface	Hu Type
Fly Ash		75	Strength=F(overburden)			0.8	200	Water Surface	Constant
Dense Bottom Ash		100	Strength=F(overburden)			0.8	200	Water Surface	Constant
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Lower Dike		120	Strength=F(overburden)			0.55	300	Water Surface	Constant
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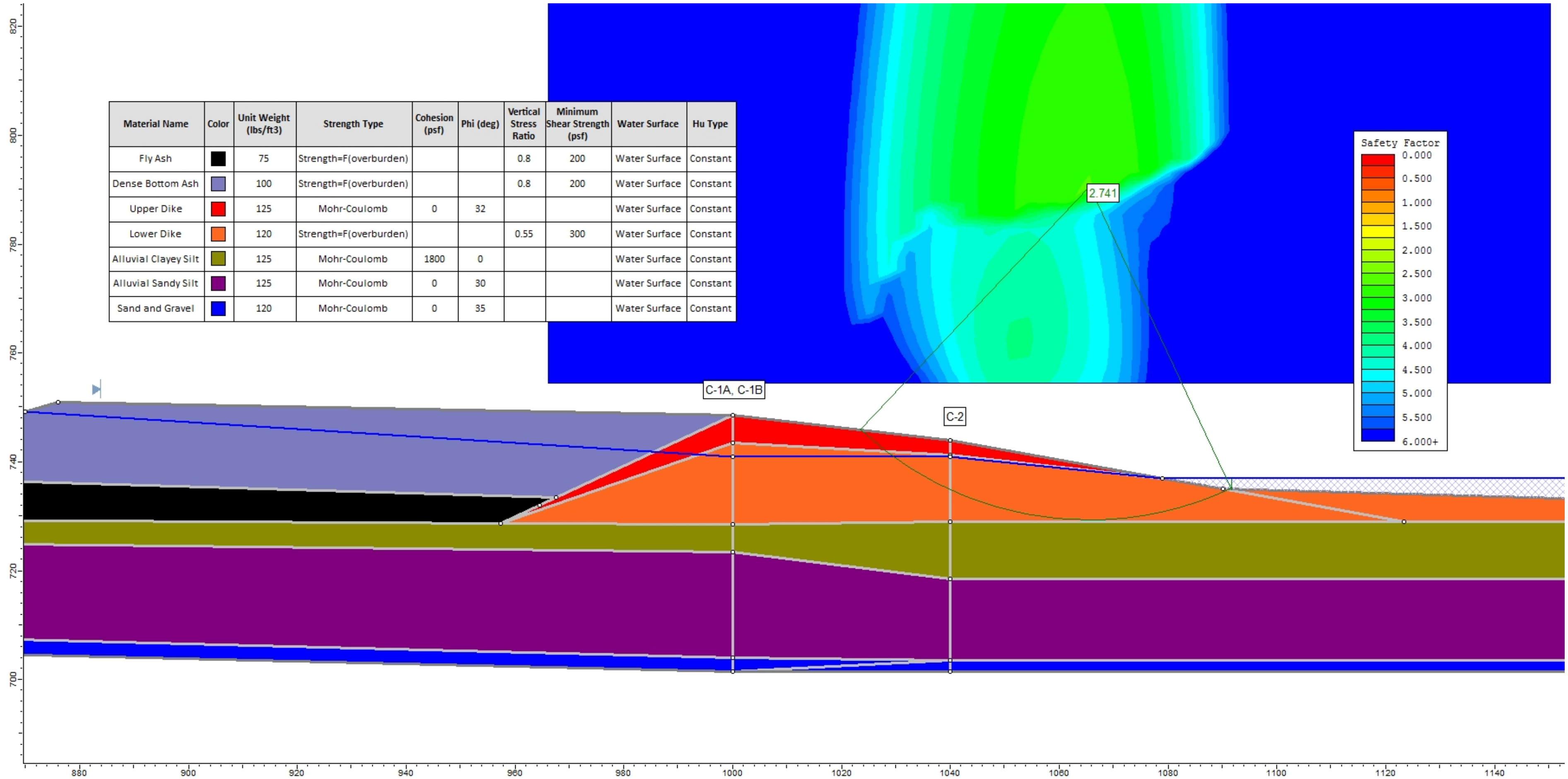
Alluvial Sandy Silt		125	Mohr-Coulomb	0	30			Water Surface	Constant
Sand and Gravel		120	Mohr-Coulomb	0	35			Water Surface	Constant



Global Stability Cross Section C – Existing Conditions

TVA East Dike, Kingston Fossil Plant, Kingston Tennessee

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Figure **A-10**

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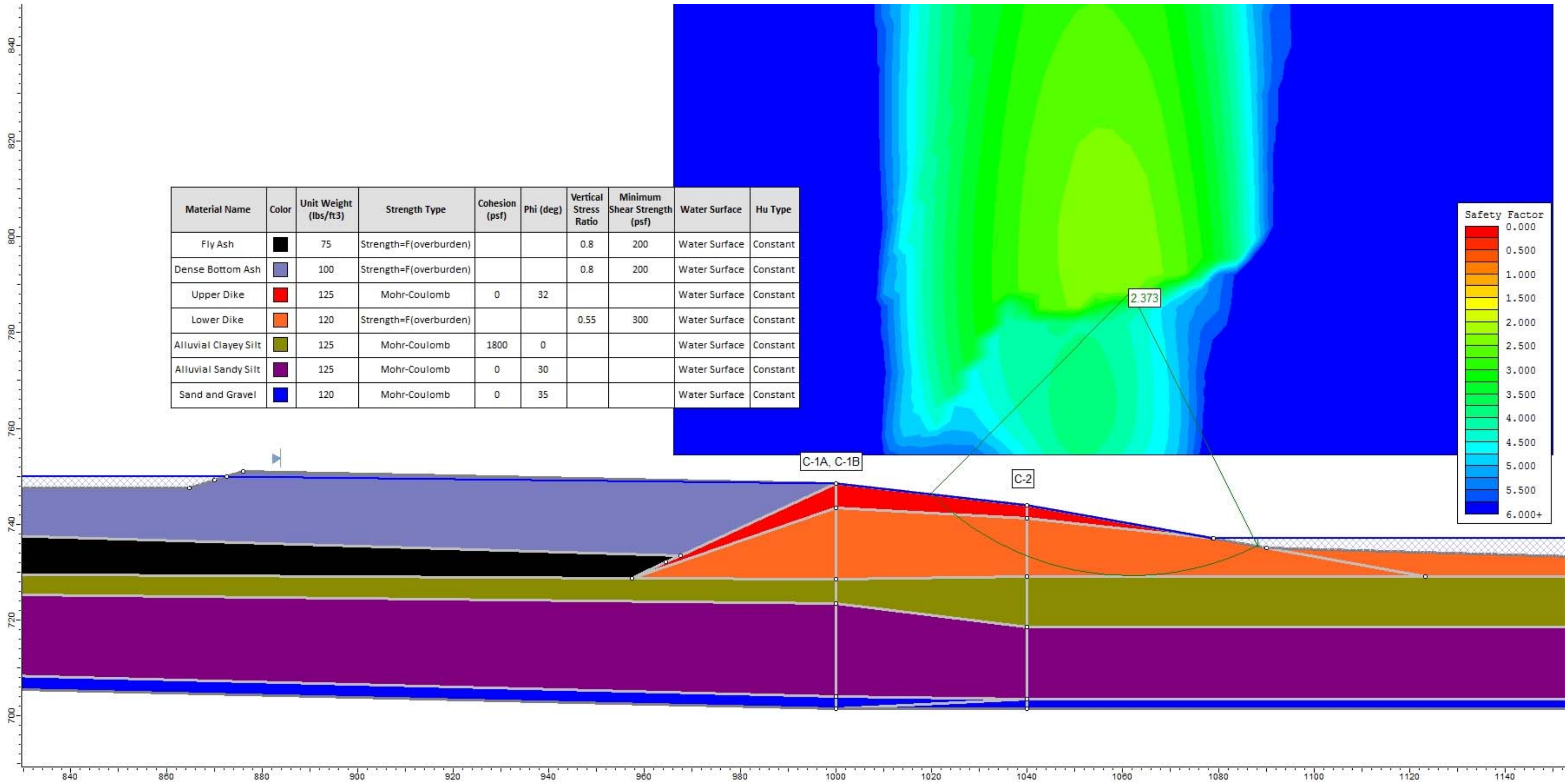

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Vertical Stress Ratio	Minimum Shear Strength (psf)	Water Surface	Hu Type
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Upper Dike		125	Mohr-Coulomb	0	32			Water Surface	Constant
Lower Dike		120	Strength=F(overburden)			0.55	300	Water Surface	Constant
Alluvial Clayey Silt		125	Mohr-Coulomb	1800	0			Water Surface	Constant
Alluvial Sandy Silt		125	Mohr-Coulomb	0	30			Water Surface	Constant
Sand and Gravel		120	Mohr-Coulomb	0	35			Water Surface	Constant

Global Stability Cross Section C – Seepage Slope

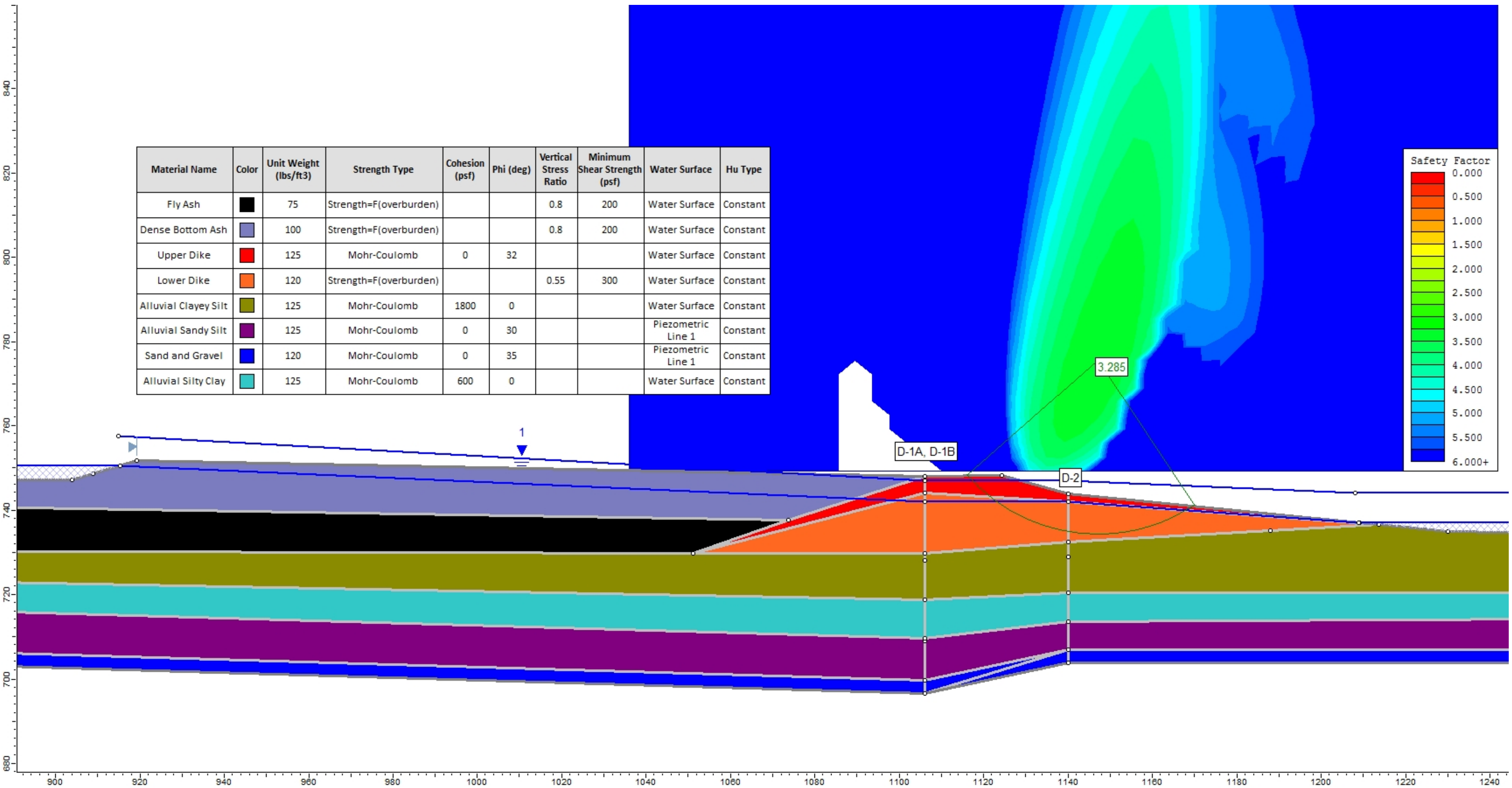
TVA East Dike, Kingston Fossil Plant, Kingston Tennessee

Geosyntec consultants

Figure **A-11**

Kennesaw, GA

7 May 2012



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Vertical Stress Ratio	Minimum Shear Strength (psf)	Water Surface	Hu Type
Fly Ash		75	Strength=F(overburden)			0.8	200	Water Surface	Constant
Dense Bottom Ash		100	Strength=F(overburden)			0.8	200	Water Surface	Constant
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Lower Dike		120	Strength=F(overburden)			0.55	300	Water Surface	Constant
Alluvial Clayey Silt		125	Mohr-Coulomb	1800	0			Water Surface	Constant
Alluvial Sandy Silt		125	Mohr-Coulomb	0	30			Piezometric Line 1	Constant
Sand and Gravel		120	Mohr-Coulomb	0	35			Piezometric Line 1	Constant
Alluvial Silty Clay		125	Mohr-Coulomb	600	0			Water Surface	Constant

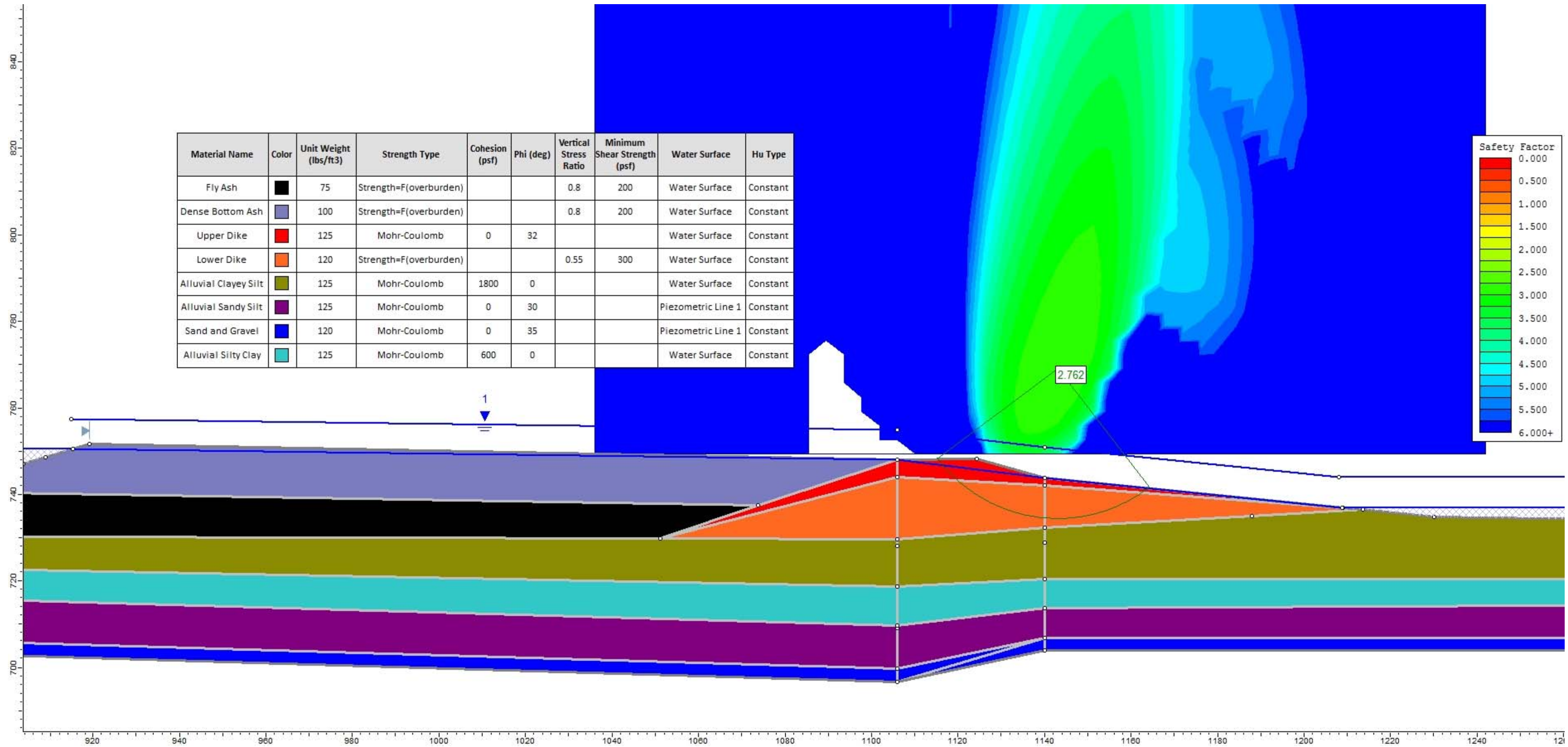
Global Stability Cross Section D – Existing Conditions

TVA East Dike, Kingston Fossil Plant, Kingston Tennessee

Geosyntec consultants

Figure **A-12**

Kennesaw, GA | 7 May 2012



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Vertical Stress Ratio	Minimum Shear Strength (psf)	Water Surface	Hu Type
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Alluvial Sandy Silt		125	Mohr-Coulomb	0	30			Piezometric Line 1	Constant
Sand and Gravel		120	Mohr-Coulomb	0	35			Piezometric Line 1	Constant
Alluvial Silty Clay		125	Mohr-Coulomb	600	0			Water Surface	Constant

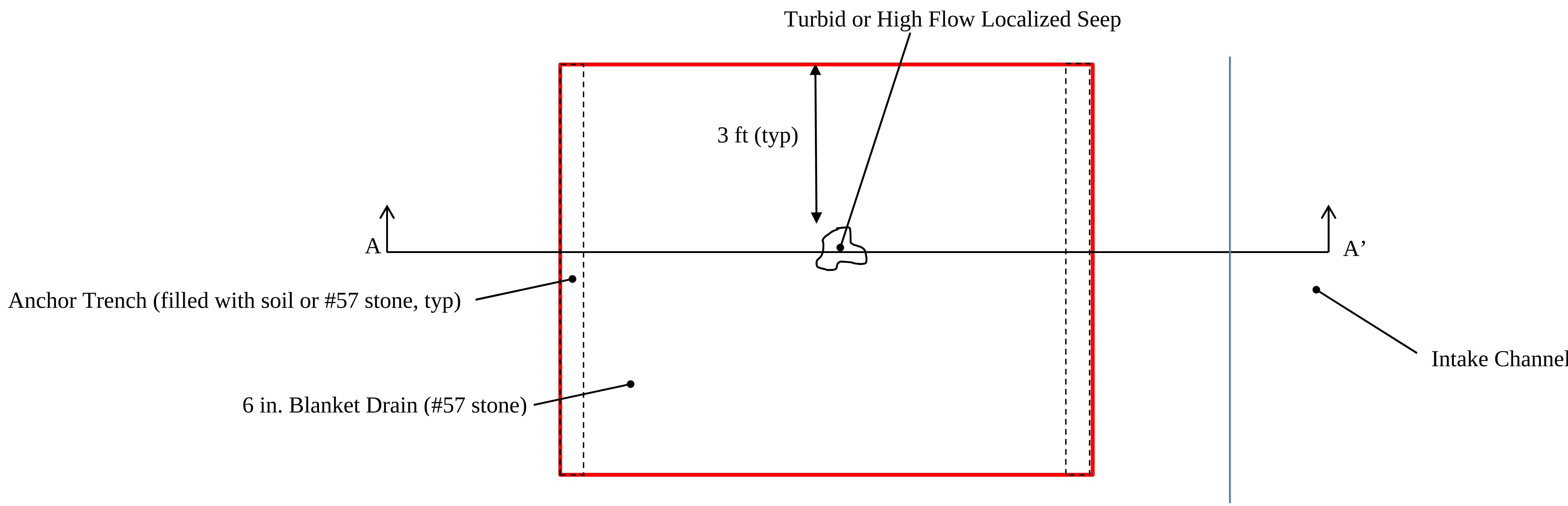
Global Stability Cross Section D – Seepage Slope

TVA East Dike, Kingston Fossil Plant, Kingston Tennessee

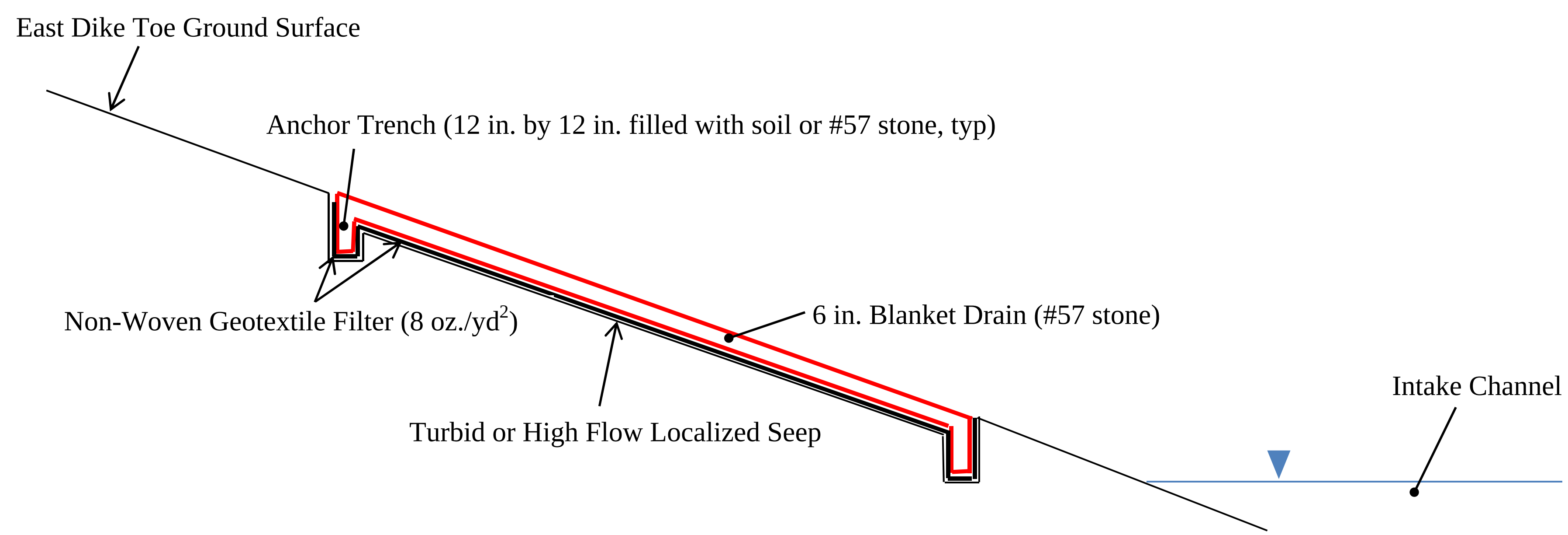
Geosyntec consultants

Figure **A-13**

Kennesaw, GA | 7 May 2012



PLAN



SECTION A-A'

Notes:

1. Not to scale, dimensions as indicated.
2. Excavate around seep area to a depth equal to the Blanket Drain thickness.
3. Blanket Drain shall extend at least 3 feet horizontally past the edge of the Seep (each direction).

Typical Filter and Drainage Blanket Detail

TVA East Dike, Kingston Fossil Plant, Kingston Tennessee

Geosyntec consultants

Kennesaw, GA | 7 May 2012

Figure **A-14**

This page was not included as part of the original document and was added at a later date.

Ref	Northing	Easting	Elevation	Name
1012	553813.28	2440861.85	748.05	CPT-12
2011	553749.47	2440704.21	748.59	CPT-11A
1011	553746.72	2440696.90	748.59	CPT-11
1013	553706.25	2440595.26	749.03	CPT-09B
1014	553703.16	2440588.04	749.11	CPT-09A
1015	553707.03	2440568.11	748.92	CPT-09C
1016	553704.00	2440559.46	748.96	CPT-09
1008	553675.33	2440488.14	747.90	CPT-08
1007	553605.29	2440229.90	753.15	CPT-07
1017	553607.16	2440192.10	752.07	CPT-07A
1018	553607.12	2440178.92	751.91	CPT-07B
1019	553580.50	2440083.94	753.12	CPT-05A
1005	553575.22	2440075.52	753.14	CPT-05
1020	553462.05	2439946.26	752.99	CPT-04A
1004	553455.95	2439938.48	752.93	CPT-04
1021	553359.40	2439825.83	754.19	CPT-03A
1022	553354.22	2439819.85	754.10	CPT-03B
1023	553349.36	2439814.87	754.24	CPT-03C
1003	553345.04	2439810.08	754.17	CPT-03
1024	553253.76	2439701.70	754.59	CPT-02A
1002	553251.87	2439694.53	754.61	CPT-02
1025	553234.85	2439674.54	754.92	CPT-02B
1026	553227.85	2439665.47	755.10	CPT-02C
1027	553221.24	2439656.97	755.20	CPT-02D
1028	553141.60	2439561.57	756.18	CPT-01A
1029	553136.03	2439555.61	756.29	CPT-01B
1001	553130.20	2439549.69	756.40	CPT-01