SLOPE STABILITY ANALLYSIS OF FLY ASH CONTAINMENT DYKE

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A THESIS SUBMITTED IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE DEGREE OF BACHELOR OF APPLIED SCIENCE

In

THE FACULTY OF APPLIED SCIENCE

(Geological Engineering)

This thesis conforms to the required standard

Supervisor

THE UNIVERSITY OF BRITISH COLUMBIA

(Vancouver)

APRIL 2010

ABSTACT

This study was conducted in order to assess the existing stability of an impoundment dyke and provide analysis for long term stability, following construction of a buttress. Engineering analysis were conducted using three computer modeling programs: SEEP/ W, SLIOPE/ W and Settle 3D. Initially the study consisted of modeling seepage and induced pore water conditions through the Dyke using SEEP/ W and Settle 3D. The analysis results were then incorporated into a SLOPE/ W model, which was used to analyze the slope stability at various stages of Buttress Construction. The results of this study concluded that the construction sequence required for the Buttress structure would temporarily lower the stability of the Dyke, but that the Dyke would remain structurally sound. Furthermore, the construction of the Buttress would significantly increase the, post construction, long-term stability of the Dyke.

TABLE OF CONTENTS

LIST OF FIGURES

1. INTRODUCTION

A containment dyke was built in the 1960s for the purpose of retaining Gypsum and Fly Ash waste material from a coal powered electrical plant. The dyke separates two containment ponds. It maintains the level of the Gypsum waste slurry at an elevation of 275m above sea level (ASL) and a fly ash pond level of 268.8 m ASL on the opposite side. Recently a dyke of similar construction and built for the same purpose as the study dyke breached, resulting in millions of cubic meters of waste material being deposited in the surrounding areas and watercourses. It is this failure which prompted the stability analysis of the study dyke to ensure a similar catastrophic failure would not occur at this site.

The purpose of this study is to examine and evaluate the different aspects of stability for a containment dyke. It will describe the current condition of the dyke in terms of stability and the factors affecting stability. It will then examine the stability of the dyke when remedial steps have been taken. In order to apply the remedial measures, construction conditions could cause further instability. Analysis during these processes will therefore also be explored.

The stability analysis was conducted using Geostudio software; specifically SEEP/ W and SLOPE/ W. In addition, the Rocscience software SETTLE 3D was used. These computer models were used to assess the stability of the study dyke in terms of a factor of safety (FOS). The first analysis was conducted in order to determine the FOS of the existing structure. The resulting analysis showed a FOS well below the acceptable level of 1.5. Therefore a buttress was designed which would bring the stability of the dyke up to an acceptable level, i.e. 1.5 or higher.

The slope stability of the resulting dyke (with buttress) was then analysed to determine the long term FOS of the structure. The construction of the buttress could create conditions which would destabilize the slope and cause it to fail. Therefore construction sequence conditions were analysed, and a minimum FOS was determined for the period of buttress construction.

2. SITE DESCRIPTION

The study area is roughly 50 hectares and consists of a containment dyke (study dyke) which impounds a Gypsum disposal area within a larger Fly Ash pond. These disposal areas were constructed in the early to mid 60s in order to contain the by-products of a coal burning electrical plant. The plant produces the Gypsum and Fly Ash through the burning of roughly 6,600 tons of coal per day in order to produce, on average, 889 mega watts of electricity.

Fly ash is collected from the chimneys of the coal burning plants and mixed with water to reduce dust and form slurry which can be pumped to the nearby Fly Ash pond. Gypsum is the by-product of flue gas desulfurization, also known as scrubbing. This process involves the removal of sulphur oxides by mixing the gasses with finely-ground sorbents, ether limestone or lime, to produce synthetic gypsum. The synthetic gypsum is then transported to the Gypsum disposal area. Both containment areas can be seen on Fig.2.1. The nature of the containment dyke is such that it is has undergone periodic expansion since its initial construction in the 1960s. This is because the material which the dyke is impounding is continually being produced by the associated coal powered electrical facility.

The cross section of the dyke which underwent stability analysis was section L-L’ shown in Fig.2.1.

The climate of the region is temperate with an average monthly rainfall of 100mm and an annual average temperature of 17 degrees Celsius.

Local rivers are present within the study area making it an environmentally sensitive zone and a high priority for stability analysis.

2.1 Subsurface Conditions

The hydraulically placed ash exhibits thin horizontal bedding planes and is underlain by soft to loose alluvial deposits of lean clay. The dyke itself is constructed out of mechanically worked fly ash material. Relative thicknesses of these materials can be seen in Fig.4.1.1.

The ground water hydrology in the study area is the result of the man made containment structures, which hold back saturated material that is completely submerged in water. The resulting ground water regime exhibits seepage (through the dyke) southwards, from the higher elevation Gypsum disposal pond to the lower elevation Fly Ash containment pond.

3. METHODOLOGY DESCRIPTION

The analysis performed for the determination of a slope stability factor of safety for a containment dyke is multifaceted, and requires the integration of several theoretical concepts with empirical relationships and computational modeling. This section will discuss the various theories and empirical relationships which are crucial for conducting the type of analysis described in **Section 4**. Much of the theory utilized in the analysis is contained in the three computer modeling software programs: SEEP/ W, SLOPE/ W and Settle 3D. This literature review will therefore describe, in brief, the techniques employed by each of these models, as well as the methodology for determination of the physical properties used in each.

3.1 SEEP/ W Analysis

SEEP/ W is a widely used numerical modeling software program. It simulates the ground water regime through a material geometry using boundary conditions and material properties as inputs. SEEP/ W has the ability to model both steady state and transient groundwater conditions.

3.1.1 Boundary Conditions

Solutions to numerical models such as SEEP/W are obtained by applying various input boundary conditions. These conditions specify the total head between two points in the analysis or specify some rate of flow into or out of the system. It is these boundary conditions that drive the calculations through the model domain.

Constant head boundaries are used for submerged surfaces of an earth structure. These surfaces are assigned a total head from hydrostatic conditions that are constant for that entire surface. The magnitude of this total head is equal to the elevation of the water level that the surface is submerged by. Applying boundary conditions of this type, to the upstream and downstream submerged faces of the dyke, allows for calculations within the model domain. Additionally, it is standard practice to apply no flow boundaries to the bottom extent of the model to simulate an impermeable layer below the structure. This is done by specifying a flux of zero along this boundary. The un-submerged portion of the downstream face must also have a boundary condition. For this situation a potential seepage face condition is applied. This special condition allows for ether a specified flux of zero (when seepage is not occurring) or a total head equal to the elevation of the ground surface at that point (when seepage is occurring).

Another type of boundary condition, which is applicable for this study, is the total head transient boundary condition. This condition allows the user to model a change in the boundary condition with time. This is a useful tool when modeling drawdown of the water level on either side of the dyke. The transient analysis allows time steps to be added in, so that drawdown can be specified at a certain rate. The transient option also allows for the excess pore water pressure dissipation to be modeled, for selected time steps, following the completion of drawdown. The pore water pressures for any given time step can be imported into a SLOPE/ W analysis to investigate slope stability at that time.

3.1.2 Material Properties

In order to construct an accurate SEEP/W analysis there are certain material properties that need to be determined. The hydraulic conductivity (k) function of each material is the key property that needs identifying. This function dictates the materials ability to conduct water in saturated and unsaturated conditions. When the soil is saturated, it is able to conduct water most efficiently, because all of the pathways between soil grains are filled with water. When the soil begins to become unsaturated, the larger intergranular void spaces begin to drain, since the surface tension of the water is not strong enough to keep these larger voids saturated. Any water being conducted through this unsaturated material must then travel through only the smaller, still saturated voids. This increases the tortuosity of the flow path and thus decreases the conductivity. As the soil drains the hydraulic conductivity continues to drop as a function of the water content of the soil. Even in the vadose zone the soil is able to conduct water, however since this conductivity is so low, for the purposes of this study, only saturated flow is considered.

Measurement of hydraulic conductivities, for the formulation of a hydraulic conductivity function, through laboratory or in situ testing is an expensive and time consuming process. Fortunately, SEEP/ W has approximation methods built in, which use empirical relationships to estimate a reasonable hydraulic conductivity function from saturated hydraulic conductivity (ksat) and grain size distribution data. For this study, each material type underwent laboratory testing in order to determine the saturated hydraulic conductivity. The volumetric water content function was then approximated by using a typical function for that material type. This was deemed acceptable, since the hydraulic conductivity function is less crucial in the saturated zone, and this study was considering only this zone.

Another aspect of hydraulic conductivity which needed to be considered, is the ratio of vertical hydraulic conductivity (kv) to horizontal hydraulic conductivity (kh). This ratio ranges from one (for a perfectly isotropic material) to near zero (for highly anisotropic material), and therefore quantifies the anisotropy of the material. This ratio can be quite significant in determining the ground water regime in this study area. Horizontal conductivity values are typically much higher than those in the vertical direction; this is due to the orientation and geometry of soil strata which typically provide easier flow paths in the horizontal direction (Coduto 1998 p228-229). With stratified platy material grains, this ratio can become much smaller and the difference between conductivities in each direction much larger. This ratio is the main material property which is adjusted, when attempting to match the model phreatic surface with the observed.

Fine tuning of the SEEP/W model is achieved by adjusting the properties of each material within a reasonable range. When this fine tuning is complete, the SEEP/W model should reflect the observed ground water regime as closely as possible. Once the model reflects observed conditions as closely as possible, the pore pressures and hydraulic gradients from the SEEP/ W model are used in slope stability and piping analysis. FOS for slope stability is calculated using the pore water pressures from the SEEP/W analysis, as input parameters for a SLOPE/ W analysis with identical geometry and material properties. This analysis will be discussed in **Section 3.2**. To calculate a FOS against piping the vertical hydraulic gradients are used.

3.1.3 Piping

Piping is a major concern when dealing with water retaining earth structures; it causes roughly 20% of all failures in this type of structure (probeinternational.org/leading-causes-dam-failure). Piping occurs when there is a “quick” condition at the surface. This is when the upward seepage forces are equal to the weight of the material.

The seepage force is attributed to the viscous frictional drag, on the solid soil, by the flowing water in the direction of flow (Craig 1992). This energy transfer from the water to the solid particles decreases the total head as energy is transferred, this is the seepage force. If the seepage force exceeds the soil weight, then the soil will be displaced and piping will occur. Piping progressively erodes and washes out the interior of the dyke until the entire structure is washed away. The gradient at which the seepage force overcomes the weight of the material is called the critical gradient(ic) and is calculated by:

$$ic = (Gs-1) / (1+e) \quad \text{(Craig 199)}$$

Where Gs is the specific gravity of the material, and e is the void ratio.

Piping is such a dangerous condition because it is self propagating. Since the difference in total head between the ground surface and just below the surface is constant, when material is removed from the surface the distance between the two total heads is decreased, thereby increasing the hydraulic gradient and therefore the seepage force. This accelerates the piping process, which continues until the entire structure is washed out. This critical gradient must occur at the ground surface in order to cause displacement. A large seepage force at the centre of the dyke will not cause migration of material, by virtue of the entire soil weight above this point. This is the strategy for low permeability cores in the centre of water retaining earth structures. By having the large head drop over the short core length occur in the centre of the structure the gradients on the downstream face will be reduced and critical gradients will not be reached at the surface.

3.2 SLOPE/ W Analysis

SLOPE/ W is a numerical modeling software program, which utilizes the method of slices to descretize a failure mass along its slip surface. Each of these individual slices is then analyzed in terms of equilibrium. There are a variety of methods used to accomplish these equilibrium calculations, the most comprehensive, and the one used in this study, is the one developed by Morgenstern and Price. This method satisfies moment and force equilibrium for each individual slice. The power of SLOPE/ W is its ability to rapidly calculate many slip surfaces and to optimize the critical slip surface. In order to delineate the extent of the study slope and to save computing time, the radius and tangent slip surface grid of the computed slip surfaces are specified. As with SEEP/ W modeling, material properties play a key role in developing a realistic and accurate model.

3.2.1 Material Properties

The majority of the technical research done for this study, was to ensure that strength parameters for the materials were established at a reasonable value. There are two situations which were considered, the static long term loading of the material, or drained condition, and the dynamic short term loading, or undrained condition. When a soil is loaded the soil grains are forced closer together, this reduces the pore space between the grains. In unsaturated material this reduction of pore space will occur within the air filled voids first as air is easily forced out of the voids. When all of the air filled voids have been closed so that only water filled voids remain, the soil is saturated. In our study we considered only the saturated condition, since our water table was close enough to the surface that the failure surfaces would shear almost entirely through only saturated soils.

For saturated soils the water in the void space is considered incompressible. Loading of saturated soils therefore causes an immediate increase in the pore pressures, as the whole load is being resisted by the incompressible water.

(Coduto 1998). For coarse grained material, the individual voids are much larger, which makes for easier drainage paths for the pore water and therefore higher hydraulic conductivities. These types of soils dissipate this water so quickly in fact that the condition is always assumed to be drained. Fine grained soils however, have much smaller individual voids and much lower hydraulic conductivities. It therefore takes much longer for pore pressures in these types of soils to dissipate, resulting in undrained conditions for a significant length of time, and drained conditions only after long periods of static loading.

The difference between drained and undrained strength parameters is an important one and is essential to developing a representative model. Drained strength parameters, also known as effective strength parameters, are obtained from Consolidated Drained (CD) triaxial tests as described by Coduto 1998. These tests provide values for effective strength parameters: cohession (c') and friction angle (ϕ'). These parameters relate to the shear strength of that soil by the following equation:

$$S_d = c' + \sigma' \tan \phi'$$

Eqn3.2.1: (Terzaghi 1943)

Where σ' is the effective stress, Sd is the effective shear strength and ϕ' is the slope of the failure envelope (c'= 0).

The magnitude of the effective stress is determined by the weight of soil on top of the depth in question and a hydrostatic or seepage based pore water condition.

The c’, or cohesion, for coarse grained material is equal to zero since cementation and particle bonding is minimal for these types of soils and cannot be relied upon. (Craig 1992)

In fine grained materials, the drained or undrained strength parameters are used depending upon the condition being analyzed. For short term stability analysis, ie immediately after loading or unloading, the undrained conditions must be used, since there is not enough time for pore pressures to dissipate. For static loading over long periods of time, drained conditions can be used. This is because hydrostatic or seepage conditions are able to establish equilibrium pore water conditions over long periods of time. Any subsequent loading or unloading, will however, induce excess pore water pressures and undrained conditions will again need to be used.

Undrained behaviour in soils can be theoretically described in the same way as drained soils (Eqn 3.2.1). However, drained (effective stress) conditions are not used for short term loading. This is because the effective stress on a fine grained soil is difficult to determine, due to the inability to accurately predict the pore water pressures induced during the shearing process. (Kulhaway 1992). For long term conditions, when pore pressures are allowed to come to equilibrium, drained conditions are suitable. Drained shear strength can be determined from a Consolidated Drained triaxial test, which is conducted slowly enough for pore pressures to dissipate. Alternatively, Consolidated Undrained tests, where the pore water pressures are measured, can be performed (Craig 1992).

For static short term conditions in fine grained soils, the undrained shear strength is used in analysis.

The undrained shear strength (Su) of a soil, is based on several factors including:

- the anisotropy of the soil
- the mode of shearing
- the stress history of the soil
- the water content, void ratio and degree of saturation
- sampling disturbance of test samples
- strain rate

Therefore, for undrained conditions, other relationships for shear strength are used.

If a saturated soil does not experience increased shear strength, due to increased normal stress, the slope of the failure envelope must be horizontal (ϕ =0) and the strength of the soil is completely due to cohesion. Unconsolidated Undrained tests, from various samples within the same thin soil layer, will yield a single value for shear strength for that soil. This is represented by a ϕ =0 and c=Su and a horizontal shear strength envelope (Coduto 1998). Since the samples are not consolidated, the confining pressure on that soil is applied only to the pore water and does not increase the shear strength. For thicker soil deposits, UU tests can be done for samples at various depths and a plot of Su as a function of depth can be obtained (Coduto 1998). Alternatively, cone penetration tests can be conducted, which give Su values along the depth profile. This data can be used to develop a ratio of Su/ σ'. This is effectively a ratio of undrained shear strength with depth, and is often used in Geotechnical practice (Bowels 1984). This technique was used in the development of representative Su values for modelling of the study dyke.

3.3 Settle 3D Analysis

Following the slope stability analysis for the existing condition of the study dyke, it was determined that remedial measures would have to take place in order to increase the factor of safety of the dyke slope. The chosen method of stabilization for the site, was to load the toe of the slope with large Rip Rap material or buttress, in order to resist circular slip surfaces that would exit through the toe of the slope. This is a common stabilization technique used in Geotechnical Engineering; however care must be taken when placing any material near a slope to ensure that a failure scenario is not induced by the added weight, or by the induced pore water pressures below the newly applied load. It is the latter of these two which concerns the study dyke.

In order to determine the magnitude of induced pore pressures, due to the buttress, the computer program Settle 3D was used to model the conditions beneath the new load. Settle 3D is a computer modeling program, which enables the user to quickly perform settlement calculations for given soil and load conditions. The settlement calculated by the program is due to immediate or initial settlement, settlement due to consolidation and secondary settlement (creep). Immediate settlement is that settlement which occurs immediately after the load is applied and is linear elastic. Secondary settlement can occur at a constant effective stress and therefore is independent of pore water dissipation. It is the mode of settlement due to consolidation, that we are interested in for the purpose of this study. This type of settlement is due to the expulsion of water from the pore space of the material, due to an increase in total stress.

As discussed earlier, when a load is first applied, the entire load is taken up by the incompressible water in the saturated material. This increases the pore water pressure by a magnitude equal to that of the increased load. In coarse grained material, this water is quickly forced out of the pores until the pore pressure reaches its hydrostatic equilibrium. At this point the entire increase in total stress has now been transferred to the soil skeleton and therefore the increase in effective stress is equal to the increase in total stress, and the soil particles are able to consolidate accordingly.

In fine grained soils, where the hydraulic conductivity is much lower, the excess pore pressures cannot dissipate as quickly and it takes much longer for the applied total stress to be transferred from the excess pore water pressure to effective stress on the soil skeleton. Therefore it takes much longer for fine grained soils to consolidate. It is important, for slope stability analysis, to determine the correct pore water conditions at depth in a material, so that the effective stress can be used for shear strength determination. It is this distribution of pore water pressures that is most important, for determining the stability of the slope during buttress construction.

Settle 3D utilizes Terzaghi's 1D consolidation equation:

$$\frac{\partial u}{\partial t} = c_v \frac{\partial^2 u_e}{\partial z^2}$$

Eqn3.3: Terzaghi 1943.

Where Ue is the excess pore water pressure, cv is the coefficient of consolidation and z is the vertical distance below the ground surface. The excess pore water pressure, at any time, can thus be calculated and the effective stress at this time determined. The excess pore water pressure (Ue), just after time zero, when the buttress load is applied, is equal to the applied load at the ground surface. However, this applied load influences the lower soil layers to a lesser extent as expressed in stress bulb theory by Coduto 1998.

Therefore, the excess pore water pressures will be the greatest just below the applied load and will diminish as depth increases. Settle 3D accounts for this attenuation of stress with depth and computes the corresponding Ue accordingly. The rate of consolidation is dictated by the rate of pore water pressure dissipation. This is accounted for in settle 3D through the use of Terzaghi's equation and the specification of time steps within the computer model. This is an important principal, when looking to predict settlements over time in soil layers. However for the purposes of this study the important relationship is that of the pore water pressure with time.

For stability analysis, the total settlement of the soil layer is of little importance, it is rather the excess pore water pressure which dictates the stability of the soil at that depth. It is therefore useful, to determine at what time and at what depth the maximum pore water pressures are experienced and also the time in which all excess pore water has been dissipated. By developing a timeline for pore pressure dissipation, a construction sequence can be established whereby subsequent lifts of the buttress material will be postponed until the pore pressures induced by the first lift have been adequately dissipated. In this way, the pore pressures can be kept at a safe level during construction of the buttress, to avoid instability of the soil below.

Modeling of the excess pore water dissipation, with time, is dependent upon the accurate determination of the coefficient of consolidation (Cv), which is dependent upon the permeability (k) of the material and the material stiffness. Materials which have higher permeability dissipate pore water faster and consolidate quicker than material with low permeability. In this study the permeability of the material is essential to accurately model the stress conditions at various times.

4. ANALYSIS

The existing stability analysis was run using SEEP/W and SLOPE/ W. With steady state ground water conditions and material properties obtained from laboratory testing of materials from the failure site. These conditions and material properties were entered into the SEEP/W model in order to estimate a ground water regime. Monitoring wells and water table data, from exploration boreholes along the study dyke, were then used to modify the SEEP/ W model phreatic surface to more accurately reflect observed conditions. The SEEP/ W analysis was then used as the parent analysis for the SLOPE/ W model. The combination of model types, allows pore water conditions from the SEEP/W analysis to be used for effective stress analysis in the SLOPE/ W model. The SLOPE/ W analysis was then refined to reflect existing conditions at the site. It was found that the FOS for the existing structure was well below 1.5.

In order to increase the FOS of the study dyke, an addition to the earth dyke in the form of a buttress along the toe of the dyke was needed. A buttress would increase the FOS of the dyke by resisting rotational failure of the dyke by virtue of increased weight at the toe. In order to construct this buttress, the water level in the Fly Ash pond would need to be lowered. The lowering of the water level on one side of the dyke alters the ground water regime. A new stability analysis was therefore needed, in order to determine the FOS during and after the drawdown of the Fly Ash pond. Following the drawdown of the Fly Ash pond, construction of the buttress would commence.

The buttress was to be constructed of large rip rap type material. The placement of this material on top of the existing dyke would induce excess pore water pressures in the underlying soil. These excess pore water pressures could destabilize the slope by reducing the effective stress, of the soil. In order to estimate the magnitude of these pore pressures, a Settle 3D model was developed, which analyzed the induced pore water pressures with time, due to the placement of the buttress material. The induced pore pressures were then incorporated into the SLOPE/ W analysis in order to estimate the FOS during construction. Eventually, these induced pore pressures would dissipate and the dyke would become more stable. Stability of the slope, after the pore pressures dissipate, must have a FOS of 1.5 or greater. The FOS, during construction, must be maintained at 1.1 or higher to ensure that failure of the dyke does not occur during construction of the buttress.

The FOS against piping through the dyke was also assessed, since this is another possible failure mechanism. This analysis was conducted at the same time as the above mentioned SEEP/W analysis and is based on vertical hydraulic gradients at the surface of the down slope face of the dyke. If these gradients become too large the seepage forces cause migration of the earth material along these gradients, which leads to internal erosion of the dyke.

This section is intended to describe, in detail, the steps that were taken in the slope stability analysis of the study dyke. It will include descriptions of the settings used in each of the applied computer models, as well as justification for the physical material properties used in each. The results of the methods described here will be presented in **Section 5.**

There was no consideration in this study of seismic stability, alteration of the geometry of the existing dyke or alteration of the water levels on either side of the dyke, apart from those mentioned.

For both SEEP/ W and SLOPE/ W, the delineation of the geometry and stratigraphy of the earth structure is the first step in developing the model. This was accomplished through the use of topographic survey data and an extensive drilling investigation program. This program was necessary because no "as built" drawings of the current dyke configuration were available. The surface elevations were obtained from the topographic survey and the delineation of the layers, within the structure, were interpolated from bore hole logs obtained from the drilling investigation. This data was then incorporated into the SEEP/ W program. The SLOPE/ W model was then produced by modifying the SEEP/ W model. This allows the geometry and pore water conditions to be directly applied to the SLOPE/ W model for stability analysis.

4.1 SEEP/ W

Once the geometric configuration of the earth structure was imported into SEEP/W, the boundaries of the individual material types needed to be specified. In SEEP/ W, this is done through the "Draw Regions" command. Using this function, the material types are separated into regions. Each region has its own physical properties which are applied to that entire region. As shown in Fig 4.1.1

the SEEP/W model consisted of three distinct regions. These were the Ash Dyke, the Sluiced Fly Ash and the Native Lean Clay. For SEEP W analysis, the only physical property that needs to be assigned to the regions is the hydraulic conductivity. As discussed in **Section3,** the hydraulic conductivity function is determined through the saturated hydraulic conductivity of the material and an empirical relationship between volumetric water content and non saturated hydraulic conductivity.

Fig 4.1.2 shows the graph of the matric suction vs. volumetric water content. This graph is developed through the SEEP/ W program by importing grain size distribution data. This relationship then dictates the hydraulic conductivity of the material for different matric suction (or pore water pressures). Fig 4.1.3 shows the graph of Matric suction vs X Conductivity, i.e. conductivity in the horizontal direction. As shown in the figure, the hydraulic conductivity for the Ash Dyke material is relatively stable for matric suctions up to about 300psf, at which point the air entry value is reached. Here the pore water surface tension is no longer high enough to keep the pores saturated and the pores begin to rapidly drain, reducing the volumetric water content and hydraulic conductivity. The graphs for the other materials look similar, with only the horizontal line for low matric suction at different levels, corresponding to different horizontal, saturated hydraulic conductivities. These graphs become more important when dealing with seepage through the vadose zone. For the purposes of our study, the defining characteristic of the material regions is the saturated hydraulic conductivity, or the y intercept on these graphs. The saturated hydraulic conductivities were determined through third party laboratory testing and yielded values as shown in Fig 4.1.4 for vertical hydraulic conductivity but with a Kv/Kh ratio of 1.

In order to make the finite element calculations more reliable, the element mesh was smoothed in order to keep the individual elements as equilateral triangles or squares, as suggested by the SEEP/ W theory manual.

In order to solve a seepage problem, boundary conditions must be applied. The left (upstream) face of the dyke, had a total head boundary condition of 275 m, this corresponds to the elevation of the pond along the upstream face of the dyke. The right (downstream) face of the dyke, had a total head boundary condition of 268 m, this corresponds to the level of the pond along the downstream face of the dyke. These boundary conditions quantify the difference in head between the two faces of the dyke and establish the hydraulic gradient, which drives the model. In addition, the downstream face of the dyke, which was above 268 m elevation, was assigned a potential seepage face boundary condition. This would allow the phreatic surface to intersect the ground surface if necessary. Also, an impermeable boundary was established along the base of the native lean clay in order to delineate the problem domain, a no flow boundary was specified for this.

The model was then run and a phreatic surface was produced. However, the phreatic surface, determined by this first model run, was inconsistent with the observed conditions seen in the borehole information and at the earth structures surface. The ratio of Kv/Kh was then fine tuned in order to match, as closely as possible, the modeled water table with the observed. When this was achieved, the phreatic surface (contour = 0) was determined throughout the earth material and contours were developed, which identify the pressure head at any location within the dyke.

The SEEP/ W program also has the ability to calculate the hydraulic gradient at any location within the earth structure, by dividing the difference in head between two points by the distance between these points. By using this option, vertical gradient contours were produced throughout the dyke. These contours, which were close to the seepage face, indicated the potential for piping. As discussed in **Section 3.1,** the critical vertical gradient for piping to occur is calculated by icr = (Gs-1) / (e+1) and for the material through which the seepage is observed (ie the sluiced fly ash) Gs = 2.45 and e = 0.65, the critical vertical gradient is therefore 1.06.

The SEEP/ W model at this point represented the existing condition of the containment dyke, this condition was evaluated using SLOPE/ W and was determined to have a FOS which was too low. The buttress material was considered the best option for increasing this FOS. However, in order to construct the buttress, the water level in the downstream Fly Ash pond would need to be lowered by 2.2 m to 267 m elevation. This posed a potential risk for the stability of the slope, as hydrostatic pore water pressures, established within the earth structure before the start of drawdown, would dissipate slowly following drawdown. This would therefore reduce the effective stress on the material. In order to model the stability of the slope, due to the drawdown, the pore water pressures within the slope would need to be determined. This is achieved through the transient seepage option within SEEP/ W.

Modeling the drawdown of the Fly Ash pond was conducted by selecting the transient option in the analysis dialogue. The number of time steps was then set to 30, so that the condition during and after drawdown could be observed at 30 different intervals. The right side boundary condition was then set to a function which varied total head with time. The total head was then reduced by 1 m per week (as per the operational procedures at the containment site) until the pond reached 267 m elevation. The total head then remained constant. In this way the pore water conditions, during and after drawdown, could be determined at various stages. The slope stability analysis could then be run for all of the time steps.

4.2 SLOPE/ W

The Geostudio software suite, allows the user to combine two or more analysis types in order to transfer valuable information between two or more models. For the purposes of our study, the SEEP/ W analysis described in **Section 4.1** was used as a “parent” analysis for the subsequent SLOPE/ W analysis. This allows the SLOP/ W model to use the region geometry and the pore water conditions from the SEEP/ W analysis in its slope stability calculations.

The analysis type was chosen to be the Morgenstern Price analysis and the circular failure slip surface option was chosen. The slip surface was specified, using radius and tangent slip surface grid specifications, which were selected to evaluate the slope of interest, ie the lower downstream slope. Throughout the analysis process certain failure mechanisms which presented lower factors of safety were rejected, this was due to the nature of the slip surface that was investigated by the SLOPE/ W program. Typically in each analysis there would be failure surfaces which were very shallow, ie less than 1 m thick, these were disregarded as unrealistic because surface factors such as root cohesion and matric suction which were not inputted into the model would resist this mechanism of failure. Therefore, only deep seated failures were investigated.

In addition to the information obtained from the parent analysis, material properties which pertain to the slope stability calculations must be entered into the model. Specifically, the unit weights and shear strengths of each material must be specified.

The material properties for the Ash Dyke were determined through density testing and triaxial tests of similar material from a nearby site, as well as empirical relationships with SPT and CPT data. The triaxial tests indicated a Su/ σ' ratio of 0.5 for the Ash Dyke material. This is much higher than the ratios of 0.12-0.25 which are typically observed for normally consolidated, saturated cohesive materials. The Ash Dyke was considered to be normally consolidated since it was a construction material, and has not been subjected to any higher stress conditions in the past (such as glaciations). CPT data correlations, as shown in Fig.4.2.1, were used to calculate a representative Su/ σ' ratio as follows:

Su(kPa)	Su(psf)	Depth(m)	Depth(ft)	σ'	Su/ σ'
5	104.4	5	16.5	702.9	0.148528
8	167.04	5.5	18.15	773.19	0.21604
14	292.32	3	9.9	421.74	0.693128
					0.352565

Table 4.2.1: Calculation of Su/ σ' ratio.

Through back analyzing the existing conditions at the site, it was judged that a Su/ σ' ratio of 0.4 and a minimum Su of 410 psf would be representative of the material. This was done by altering the minimum shear strength value within a reasonable range so that the model represented the existing conditions at the site, in which the slope had not failed and therefore had a FOS greater than 1. The minimum Su is indicated for layers that are close to the surface which have relatively low effective stress associated with them but still have significant strength due to surface processes, such as root cohesion from vegetation, or desiccation of the surface, which produce a crust which is stronger than a Su/ σ' ratio indicates.

The Sluiced Fly Ash material was investigated using an SPT test, as shown in Fig.4.2.2, and low blow count values (1>blow/ft) were observed. This suggests low values of Su/ σ' and a ratio of 0.1 was deemed representative, with a minimum Su (again determined through back analysis) of 350 psf.

The Native Lean Clay was tested using undrained triaxial testing, as shown in Fig.4.2.3, where the angle 22.5 corresponds to a Su/ σ' ratio of 0.4. The Native Lean Clay material was therefore considered to have similar strength properties as the Ash Dyke material, with a Su/ σ' ratio of 0.4 and a minimum Su of 400 psf, as seen in the lower section of Fig.4.2.1 (20kPa = 400 psf).

These material properties were applied to their respective regions, through the key-in tool, and the analysis for each time step was conducted. This now yielded a FOS for the lower slope on the downstream side of the dyke for the drawdown period and for several days thereafter. It was now time to evaluate the stability of the slope during buttress construction.

The slope stability analysis was to utilize another key feature in the SLOPE/ W program, which allows for pore water pressure spatial functions to be applied. The determination of the appropriate pore water pressures to be entered was conducted using Settle 3D software and will be discussed in **Section 4.3.** The spatial function allows the user to enter in any pore water pressure, at any node, within the finite element mesh. When the appropriate values were obtained from the settle 3D analysis, the pressures within the strata below the applied buttress were entered in. The program then interpolates the pressure head contours so that a realistic, smooth contour is established. This condition was then evaluated, using SLOPE/ W, and a FOS during buttress construction was obtained. The long term stability of the buttress could be determined without the application of the excess pore water pressures since, over a long period of time, these pressures would dissipate and hydrostatic conditions would establish.

4.3 Settle 3D

The Settle 3D software was used in order to determine the appropriate excess pore water pressures, within the earth dyke, which were induced due to the construction of the buttress. The actual settlement, due to the placement of the buttress, was of little interest because it is so small that it does not affect the stability of the slope. The model was first set up by defining the soil layers beneath the proposed buttress location. The area underneath the proposed buttress consisted of just two layers, the Sluiced Fly Ash and the Alluvial Clay. The thicknesses of these layers were determined by the region geometry and were 4.5 m and 6 m respectfully. The material properties which needed to be assigned where:

- unit weight
- saturated unit weight
- hydraulic conductivity (k)

The values for these properties were chosen, based off of tables of typical values for these material types, and are shown in the following table:

Layer	Unit Weight(tons/ft^3)	Sat. Unit Weight (tons/ft^3)	k(ft/s)
Fly Ash	0.0525	0.067	3.20E-07
Native Lean Clay	0.0615	0.08	2.30E-06

Table 4.3.1: Settle 3D Material Properties

The buttress was to be placed along the length of the downstream face, and was to reach elevation 269 m, assuming no settlement. The buttress was to be 5 m tall at its thickest section, and was to slope with a 3:1 horizontal to vertical ratio on its downstream side. The long term stability of the buttress was anticipated to be higher than the short term, due to the dissipation of pore pressures, so only the short term stability required a pore pressure spatial function.

Since settle 3D does not allow for the construction of sloped layers, the downstream face of the dyke was simulated using a load with the same dimensions as the dyke slope. The buttress was simulated using 11 separate loads with varying heights depending on the thickness of the buttress at that section. In order to simulate the staged construction sequence, the time steps for the model were set at 11 stages and half of the buttress height was applied at time 100 years. This was done to ensure that the induced pore pressures from the dyke slope load would be fully dissipated by the time the buttress was applied. The induced pore water pressures, from this first lift, were then plotted against depth for each time step to illustrate the extent and duration of the excess pressure. This plot (Fig 4.3.1) was then used to determine the length of time before the second lift should be applied. The second lift was simulated by adding a height to each of the existing loads, which would bring the buttress to its final thickness. The excess pore water pressures vs. depth were plotted at three locations along the buttress, immediately after the application of each buttress load. It was these plots which were used to obtain excess pore water pressures, for the spatial functions for the SLOPE/ W analysis.

5. RESULTS

At crucial stages in the analysis, involving the SEEP/ W and Settle 3D models, information from each was applied to a SLOPE/ W model, which was able to analyse the slope stability at that stage. This section will present the conditions at these crucial stages, as well as the results of each of the slope stability scenarios.

5.1 Piping

As discussed earlier, piping is a major cause of earth containment structure failures. The FOS against this type of failure should therefore be quantified, in order to determine if the conditions are reasonably stable. In **Section 4.1,** it was shown that the critical vertical gradient for piping in the seepage model was 1.06. By investigating the highest near surface vertical gradients, a factor of safety can be determined for resistance of this failure. The maximum vertical gradients often occur near the surface, since the hydraulic head at the surface is the lowest in the entire system. This will typically provide the largest head change and therefore the highest gradients. Fig 5.1.1 shows the vertical gradient contours which result from the existing condition within the earth fill dyke. It can be seen that the maximum vertical gradient in the system occurs near the surface at the lowest slope on the downstream side of the dyke. This gradient has a value of 0.2. The factor of safety against piping in this location is therefore 1.06/0.2 = 5.3. This is a high factor of safety and indicates a very stable condition against piping.

5.2 Slope Stability: During and After Drawdown

As discusses in **Section 4**, in order to place the buttress material, the downstream pond level would need to be lowered by 2.2 m. This required an analysis of the slope stability of the dyke, during and after the drawdown, when pore water pressures within the dyke would be in excess of hydrostatic. This was done using a transient seepage analysis, the results of which were used as the parent analysis for the subsequent slope stability analysis. Fig 5.2.1 shows the seepage condition immediately after the start of drawdown. Fig 5.2.2 shows the results of the SLOPE/ W analysis which corresponds to these pore water conditions. It can be seen that the lowest factor of safety against slope failure is 1.2. This indicates that the slope will not fail during this time. Fig 5.2.3 shows the seepage condition at time = 47 days, which corresponds to the end of drawdown, as the pond is lowered 0.3 m every week. Slope stability analysis were run for various time steps during the drawdown and the lowest FOS occurred just after the completion of the drawdown (time=47days). Fig 5.2.4 shows the corresponding SLOPE/ W analysis with a minimum factor of safety against slope failure of 1.1. This is a low factor of safety, but still indicates a stable condition for this time and as the excess pore pressures dissipate, the slope becomes more stable.

5.3 Slope Stability: Buttress Construction

Following completion of drawdown of the Fly Ash pond, buttress construction was to commence. The least stable condition during this scenario would be if the construction began immediately following the drawdown, when the pore pressures within the dyke were still elevated. This was therefore the condition which was investigated. Fig 5.3.1 shows the pore water conditions which were present at this time. The elevated pressures, beneath the first buttress lift, were determined through the settle 3D analysis and incorporated into the spatial function for the SLOPE W analysis. Fig.5.3.2 shows the corresponding SLOPE/W analysis results, which indicate a minimum factor of safety of 1.3 for slope failure during this time. This FOS was higher than that of the initial condition following drawdown because, despite the elevated pore pressures due to the buttress, the buttress itself provided increased resistance to the circular slip surface by increasing the load at the toe of the slope. The same procedure was followed for the analysis of the second buttress lift and the seepage conditions during this time are shown in Fig 5.3.3. An important factor, in the analysis of the second buttress lift, was selection of the time in which this second lift was to be placed.

If this second lit was placed too quickly, it could elevate the pore pressures beneath the buttress to an unstable level and failure would occur. Plots of pore pressure vs. depth, for various times, were therefore investigated in order to see when the pore water pressures had dissipated to a reasonable level. Fig 5.3.4 shows the dissipation of these pressures over time. It was determined that 18 days would be enough time, to allow for more than half of the induced pressures to dissipate. This is illustrated by the pink line in Fig5.3.4, as the first buttress lift was placed at time =100 years, and more than half of the pressure has dissipated by time = 100.05 years (or 18 days after the first lift placement). The pore pressures at this time were incorporated into the spatial function for the SLOPE/W analysis, shown in Fig.5.3.5. A minimum FOS for this condition was shown to be 1.5. The added toe load of the second lift accounts for the increase in the FOS, despite the induced pore pressures beneath the buttress.

6. Conclusions

A summary of the results of the analysis described in this study are as follows:

Condition	**Factor of Safety**
Before start of drawdown	5.3 against piping
Immediately following start of drawdown	1.2
At the end of drawdown	1.1
After placement of first buttress lift	1.3
After placement of second buttress lift	1.5

Table 6.1: Summery of results.

The factors of safety presented here were the basis for the quantitative analysis of the remediation strategy.

6.1 Qualitative Analysis

Based off of the analysis outlined in **Section 4**, and the results presented in **Section 5**, the following procedure for buttress construction was deemed acceptable:

- The water level in the downstream pond shall be lowered in accordance with the transient seepage analysis results at a rate of 0.3 m per week.
- Following completion of drawdown, the first lift of buttress material shall be placed along the entire length of the dyke, to elevation 267 m, excluding settlements. This should require material thickness of no greater than 3 m.
- The first Buttress lift shall be allowed to sit for 18 days, to allow for induced pore water pressures beneath the buttress to dissipate to an acceptable level.
- Following the 18 day settlement period of the first buttress lift, the second buttress lift shall be placed to final elevation of 269 m, excluding settlements. This should require material thicknesses of no greater than 3m.
- Following placement of the second buttress lift, it is anticipated that the new factor of safety, against slope failure, for the study dyke shall be 1.5 or greater. After this time the downstream pond elevation may be returned to its original level.

6.2 Problems and limitations

The construction of computer based models of real world conditions are inherently permeated with problems. It is therefore the goal, when developing these models, to employ enough empirical evidence and engineering judgement to justify the results obtained from these models. It is the contention of this study that all such considerations have been made and models, which most closely represent the real world conditions, using the available data have been developed. However there is doubtless inacuracys associated with nearly every aspect of the model. The following limitations and uncertainties have been identified while constructing these models.

- The geometry of the surface of the earth dyke can be determined to a high degree of reliability through topographic surveys, the region geometry however requires the interpolation from bore hole data and is implicit.
- The laboratory testing data, for determining physical properties of the materials present, were not sampled from the study dyke location but rather from a nearby dyke with similar construction.
- Physical properties for the undrained condition for soil shear strength were based on empirical relationships i.e. the ratio of Su/σ'.

7. RECOMENDATIONS FOR FUTURE WORK

It is recommended that a similar analysis to the one described in this study should be conducted which incorporates a more detailed investigation program. Specifically laboratory testing of samples from the study site should be conducted. Applying physical properties for analysis based off of samples from a different site can result in an unrealistic model and one which yields a factor of safety which is higher or lower than actual.

The Geostudio suite of software programs includes SIGMA/ W, which can perform the same task as the Settle 3D analysis in this study. Completion of a SIGMA/ W analysis and incorporation of the results into a SLOPE/ W analysis could be done more efficiently than using Settle 3D.

An alternative approach to the construction of a buttress would be a re-grading of the dyke in order to flatten the downstream slope. This technique would however require more material than the construction of a buttress and a new analysis would need to be conducted.

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APPENDIX A: Figures

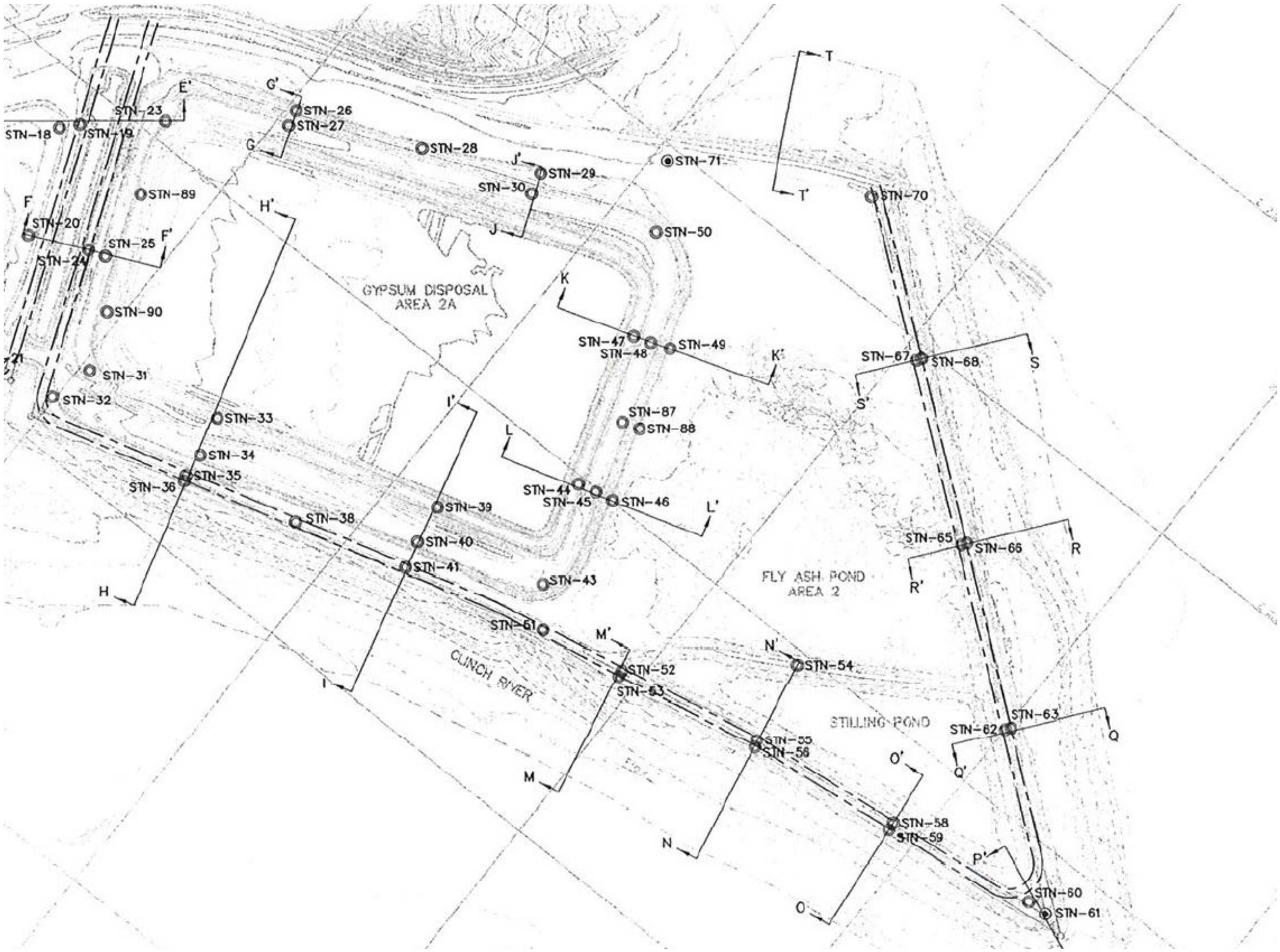

Fig.2.1: Plan view of Study Site with station locations

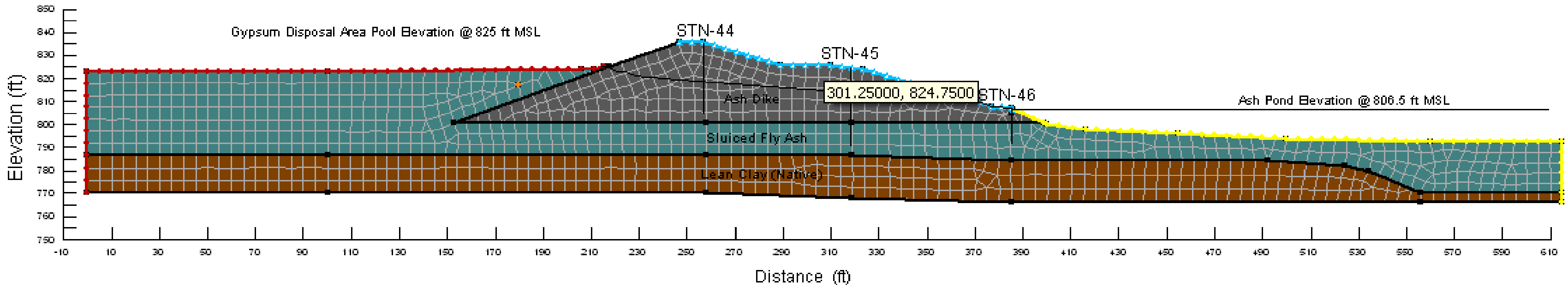

Fig.4.1.1: Model Regions with Finite Element Mesh.

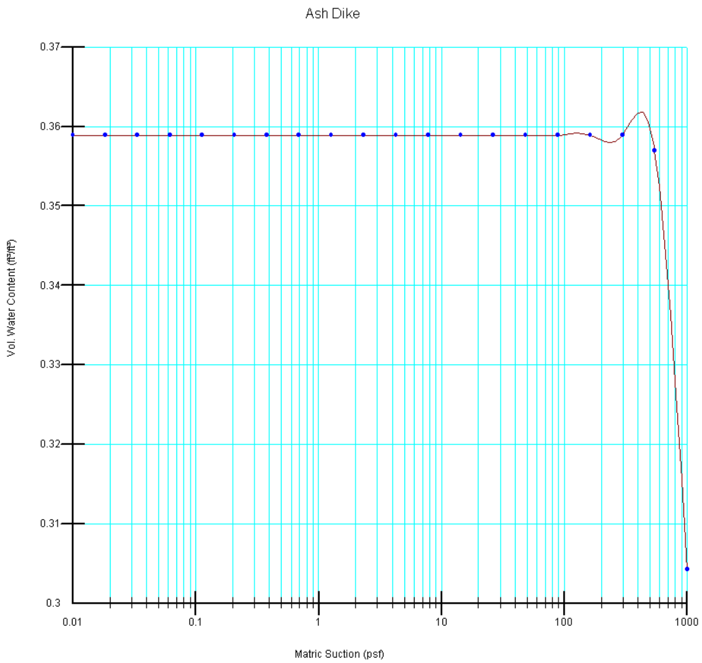

Fig.4.1.2: Matric Suction vs. Volumetric Water Content for Ash Dyke Material.

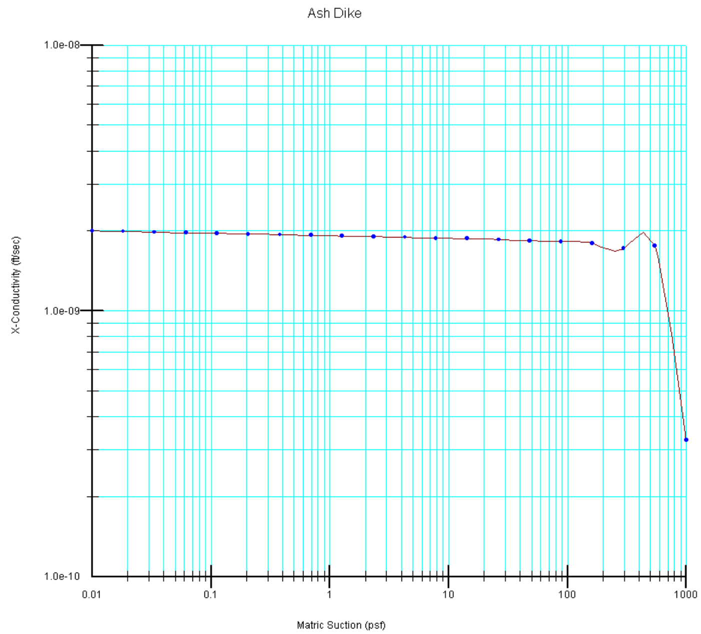

Fig.4.1.3: Matric Suction vs. X-Conductivity for Ash Dyke Material

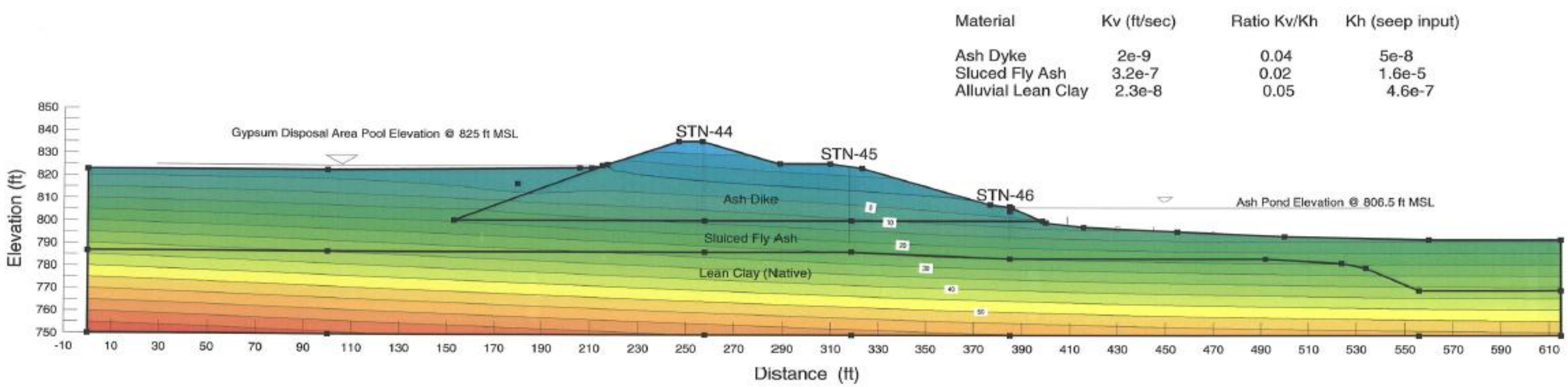

Fig.4.1.4: SEEP W Model Showing Material Properties.

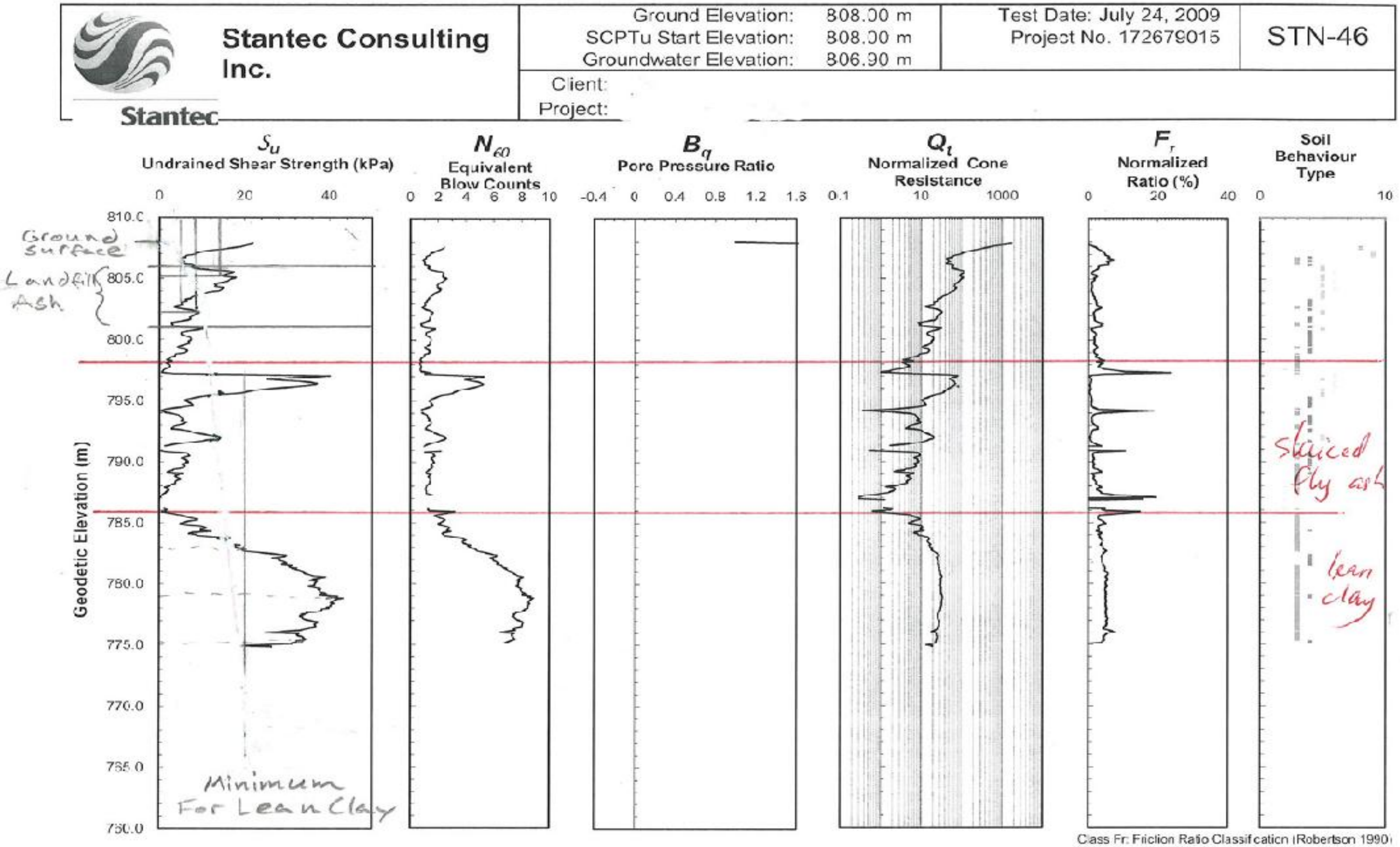

Fig.4.2.1 CPT Data from Station 46.

Project No.	172679015	Location	N 591420.20, E 2546001.83 (NAD27)
Project Name		Boring No.	STN-46 Total Depth 42.0 ft
Location		Surface Elevation	808.2 ft. (NGVD29)
Project Type	Geotechnical Exploration	Date Started	7/27/09 Completed 7/27/09
Supervisor	Ben Halada Driller Kent Clements	Depth to Water	2.0 ft Date/Time 7/27/09
Logged By	Ben Halada	Automatic Hammer ☒	Safety Hammer ☐ Other ☐

Lithology			Overburden	Sample #	Depth	Rec. Ft.	Blows	Mois.Cont. %	
Elevation	Depth	Description	Rock Core	RQD	Run	Rec. Ft.	Rec. %	Run Depth	Remarks
808.2'	0.0'	Top of Hole							
		SOIL 4: FILL - LEAN CLAY, light brown, saturated, soft		SPT-1	0.0 - 1.5	0.2	1-1-1	36	Boring advanced using 3 1/4" Hollow Stem Augers
806.2'	2.0'	SOIL 2: FILL - BOTTOM ASH/FLY ASH MIXTURE, gray, saturated, loose		SPT-2	1.5 - 3.0	0.1	1-1-2	30	
				SPT-3	3.0 - 4.5	0.5	1-1-1	41	
				SPT-4	4.5 - 6.0	0.1	WOH-1-1	49	SPT-3: organics in sample
800.7'	7.5'			SPT-5	6.0 - 7.5	1.0	WOH-WOH-1	47	Slotted screen piezometer installed. Tip depth at 10.0 ft. See piezometer installation log for details.
		SOIL 3: SLUICED FLY ASH, gray, saturated, very soft, with varying percentages of bottom ash		SPT-6	7.5 - 9.0	1.3	WOR-WOH-1	41	
				SPT-7	9.0 - 10.5	1.1	WOR-WOR-WOH	49	
				SPT-8	10.5 - 12.0	1.0	WOR-WOR-1	41	
				SPT-9	12.0 - 13.5	1.3	1-2-2	42	
				SPT-10	13.5 - 15.0	1.0	WOH-1-1	45	
				SPT-11	15.0 - 16.5	0.7	WOH-WOH-1	40	
				SPT-12	16.5 - 18.0	0.7	WOH-WOH-1	44	
				SPT-13	18.0 - 19.5	1.2	WOR-WOH-WOH	52	
				SPT-14	19.5 - 21.0	1.4	WOH	49	
				SPT-15	21.0 - 22.5	0.1	WOR	37	
784.2'	24.0'			SPT-16	22.5 - 24.0	1.0	WOR	31	SPT-16: silty clay in tip of spoon
		SOIL 6: LEAN CLAY, light brown to dark brown, moist, soft to stiff, some fine sand		SPT-17	24.0 - 25.5	1.0	1-1-3	28	
				SPT-18	25.5 - 27.0	1.0	2-4-7	29	
				SPT-19	27.0 - 28.5	1.0	7-7-9	30	
				SPT-20	28.5 - 30.0	1.1	7-9-11	30	
				SPT-21	30.0 - 31.5	1.2	6-7-11	28	
				SPT-22	31.5 - 33.0	1.1	6-8-11	26	
				SPT-23	33.0 - 34.5	1.4	8-9-12	25	
				SPT-24	34.5 - 36.0	1.0	2-3-5	25	
				SPT-25	36.0 - 37.5	1.1	2-3-6	24	
				SPT-26	37.5 - 39.0	1.3	2-3-5	24	
				SPT-27	39.0 - 40.5	1.2	1-1-3	27	SPT-40: limestone in tip of spoon
766.4'	41.8'			SPT-28	40.5 - 41.9	1.3	3-9-50/0.4	26	
766.2'	42.0'	Limestone (Augered)							
		Auger Refusal /							

2/5/10

Fig.2.2.2: Bore Hole Log for Station 46.

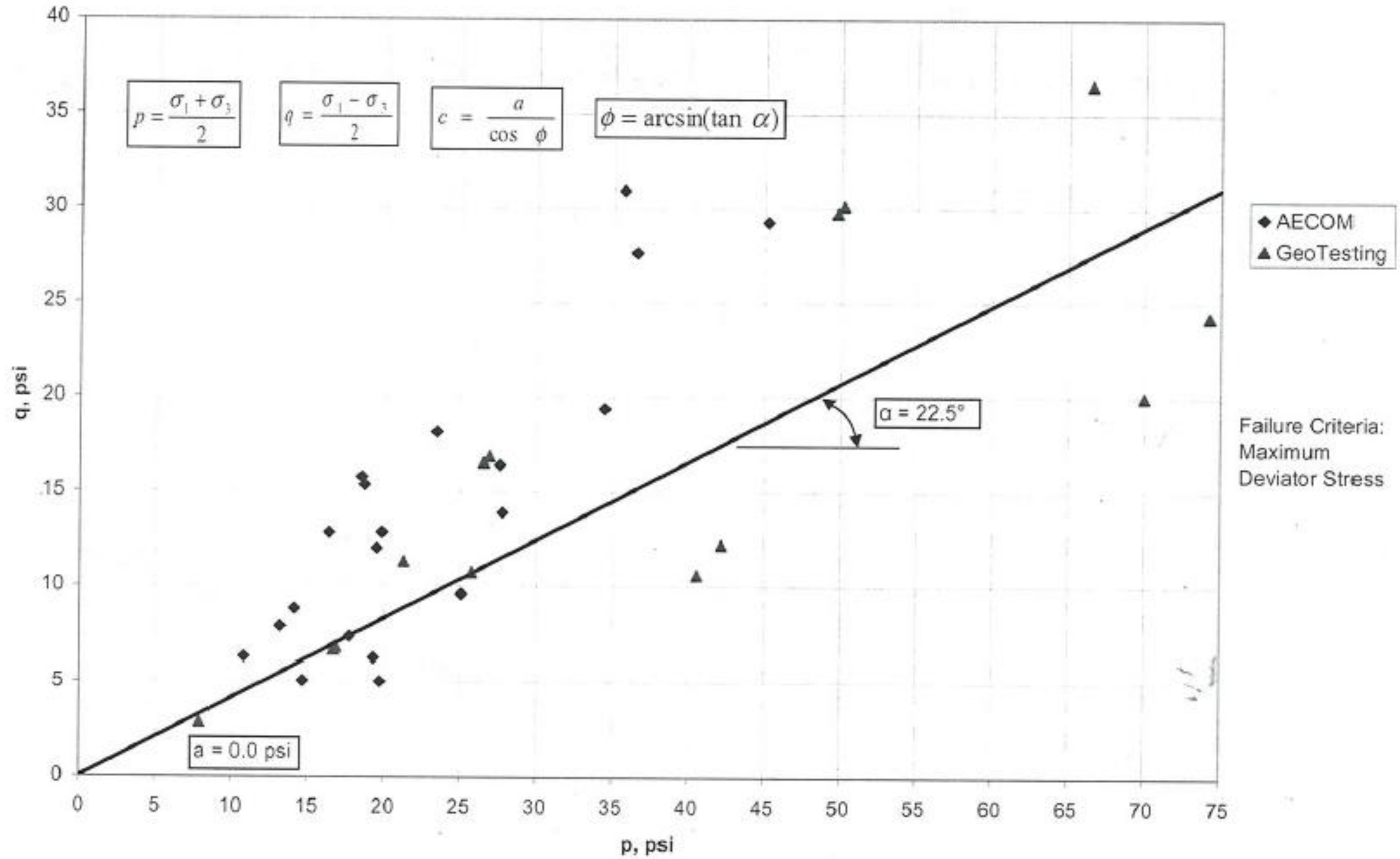

Fig.2.2.3 Triaxial Testing for Native Lean Clay Material.

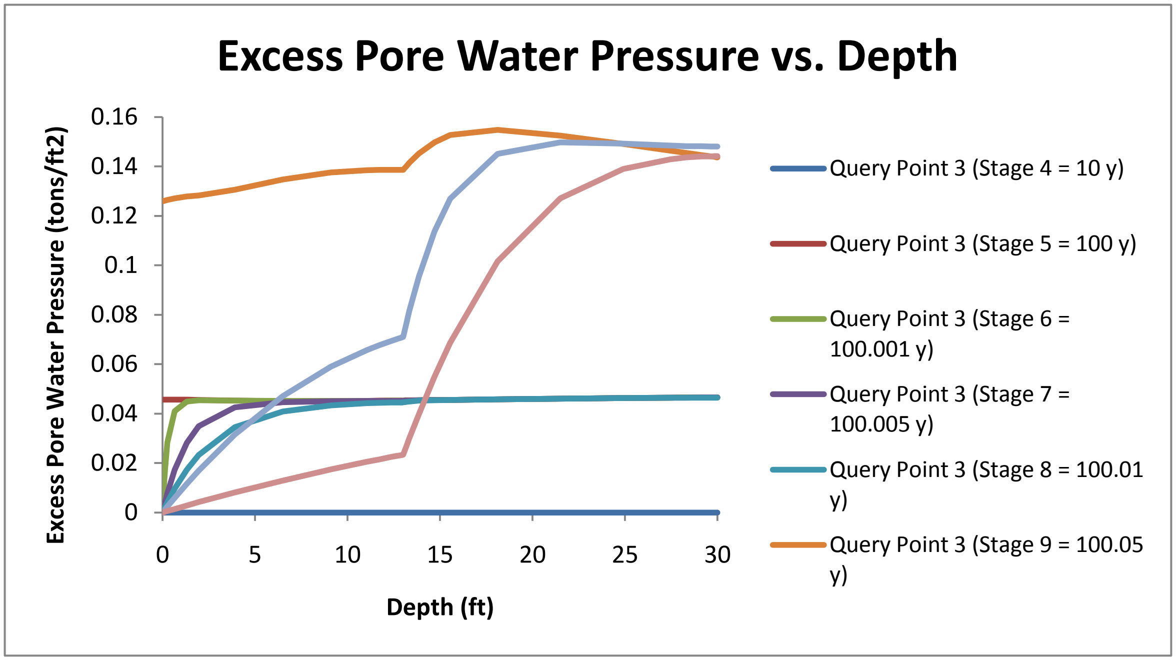

Fig.4.3.1: Settle 3D Data Showing Pore Pressure at Depth for Various Time Steps.

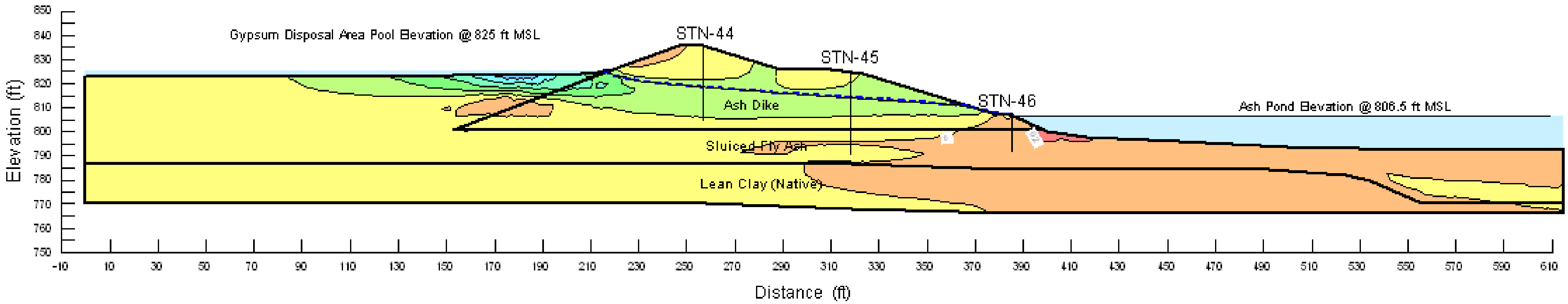

Fig.5.1.1: Steady State Seepage Model with Vertical Hydraulic Gradient Contours

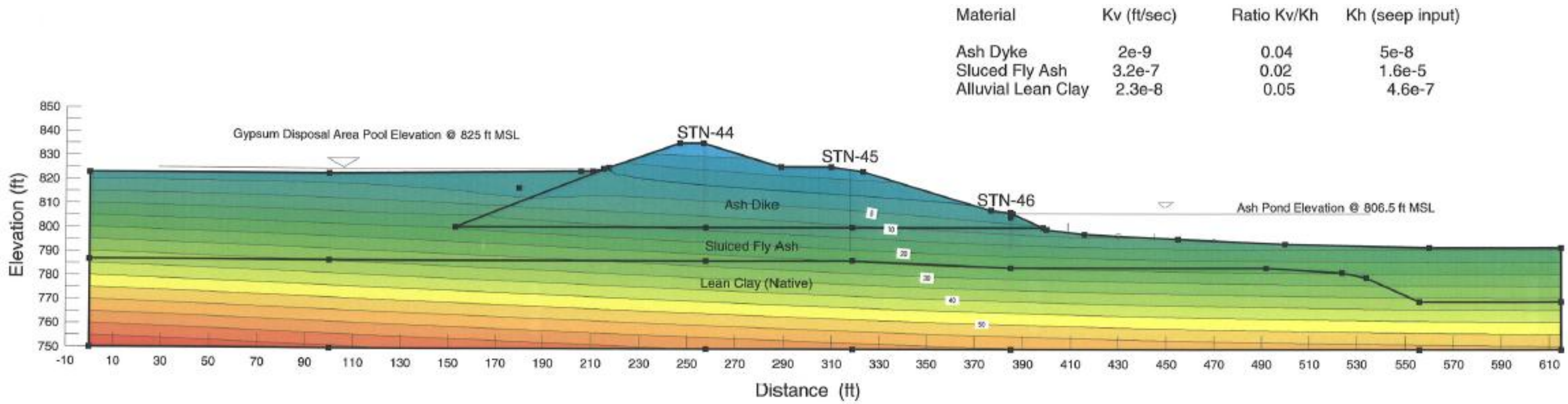

Fig.5.2.1: Transient Seepage Model at time = 0 (just after start of drawdown). Contours are in ft of H2O.

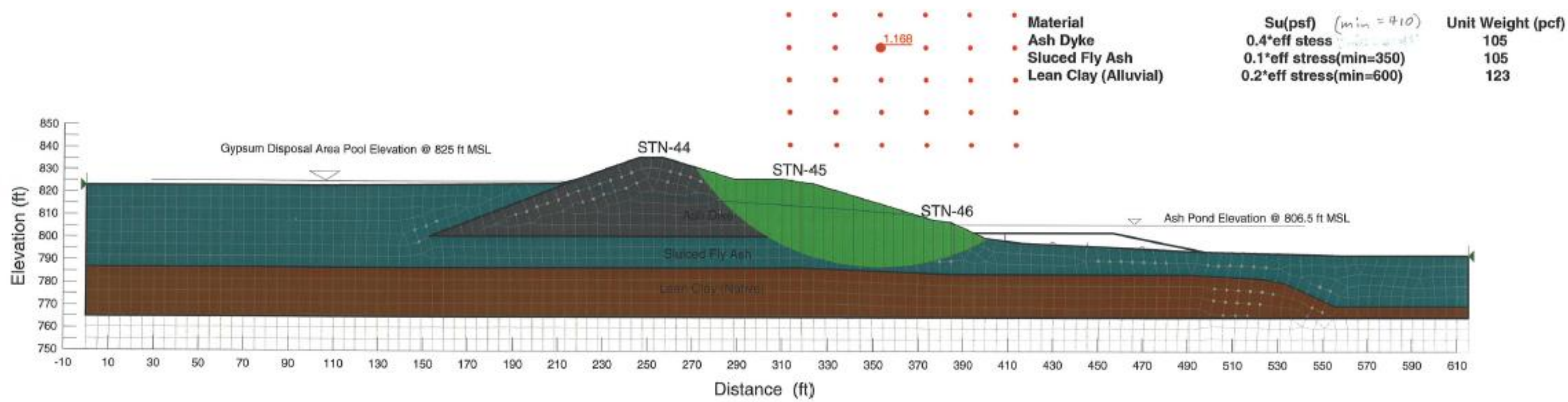

Fig.5.2.2: Slope Analysis at time = 0 (just after start of drawdown).

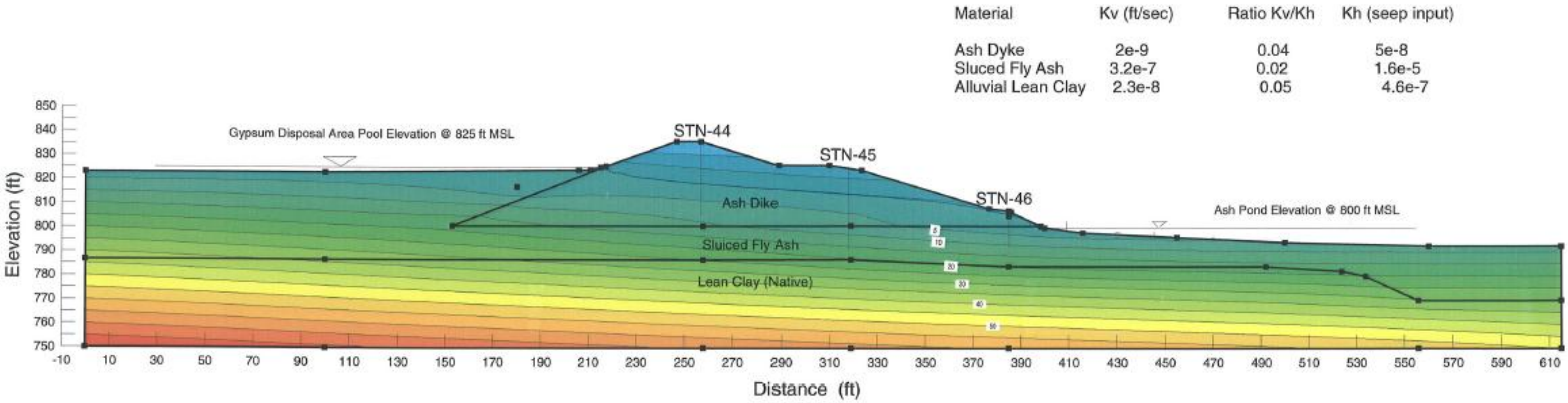

Fig.5.2.3: Seepage Model at Time = 47 days (at the end of drawdown). Contours are in ft of H2O

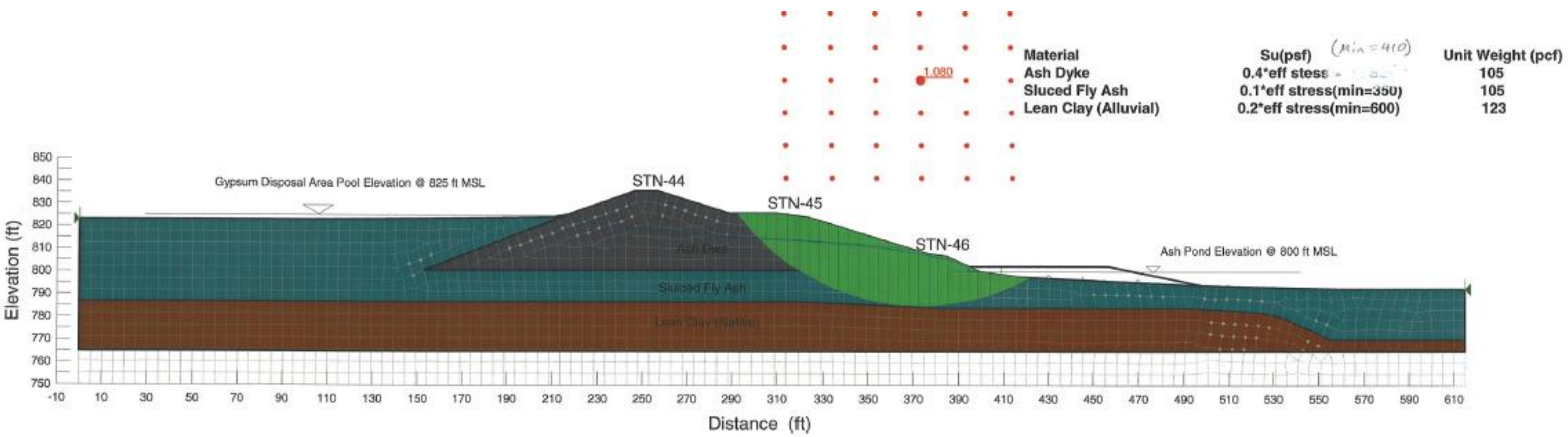

Fig.5.2.4: Slope Analysis at Time = 47 days (end of drawdown).

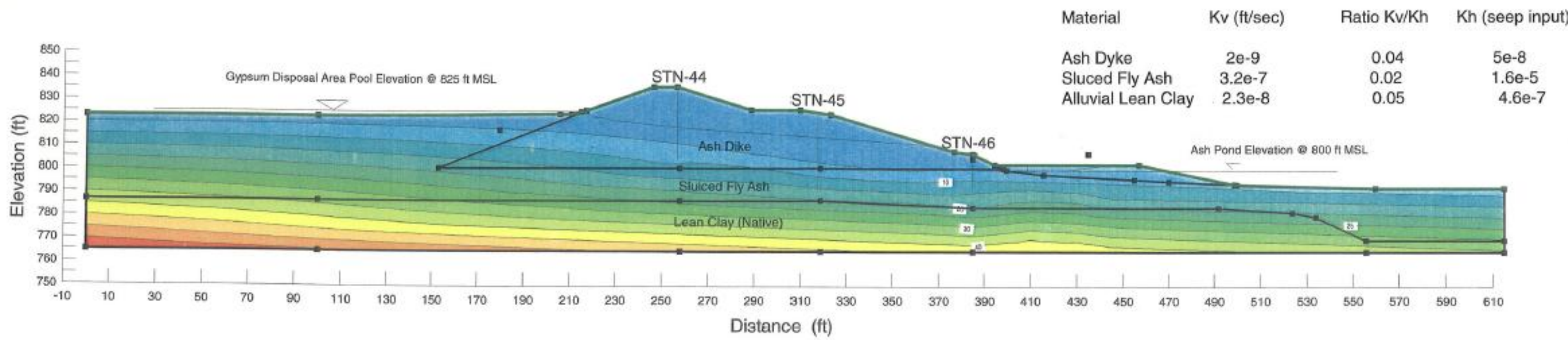

Fig.5.3.1: Pore Water Conditions at time = 47 days (just after end of drawdown) with Induced Pressures from First Buttress Lift. Contours are in ft of H2O.

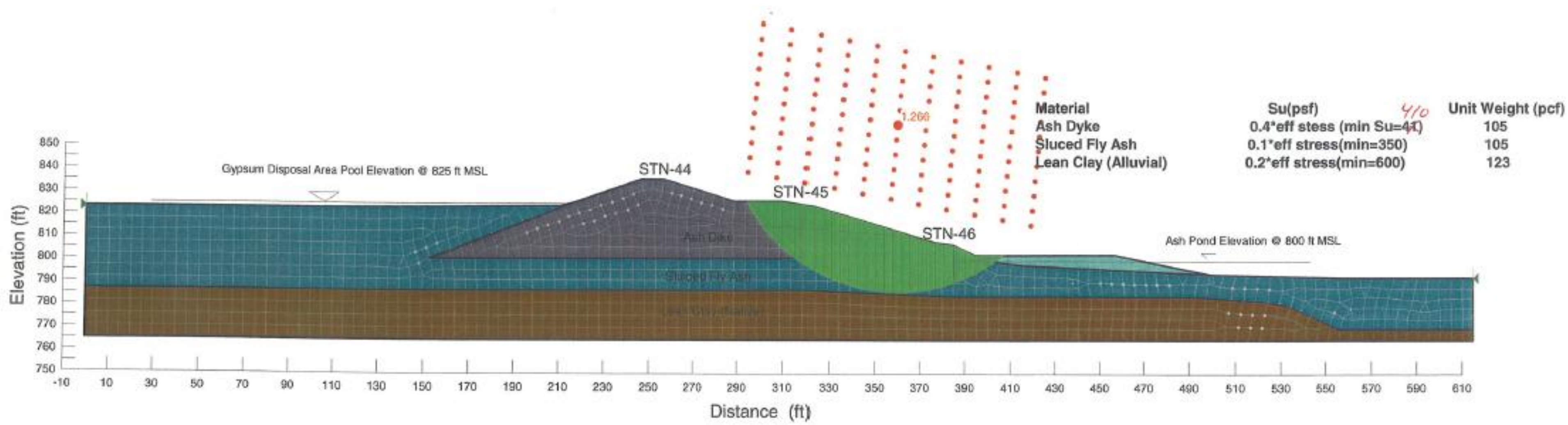

Fig.5.3.2: Slope Analysis at time = 47 days (just after end of drawdown) with Induced Pressures below First Buttress Lift.

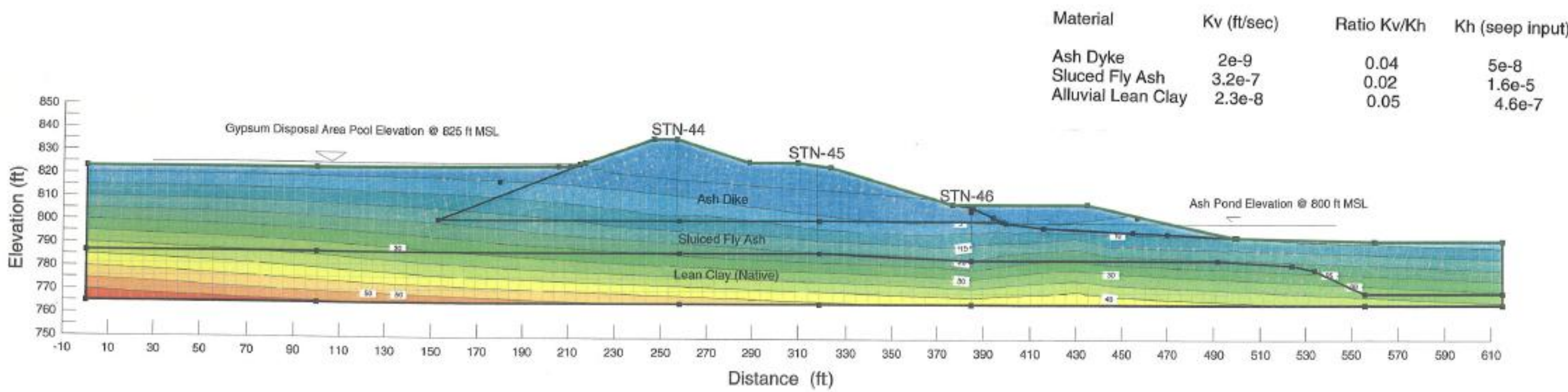

Fig.5.3.3: Pore Water Conditions at Time = 65 days (18 days after completion of drawdown) with Induced Pressures from Second Buttress Lift. Contours are in ft of H2O

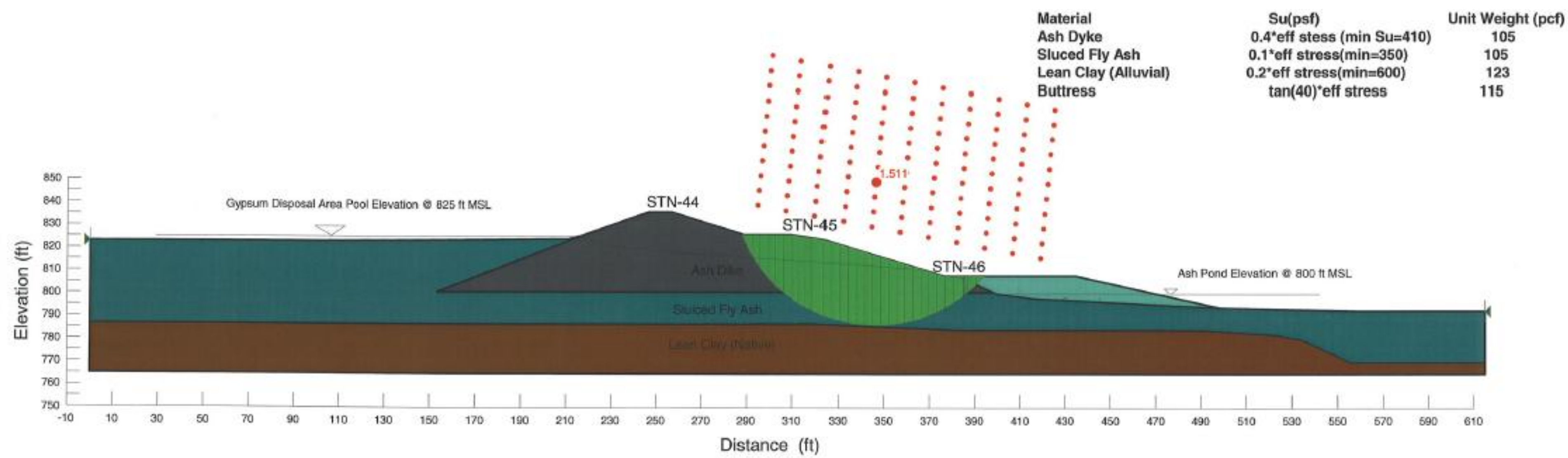

Fig.5.3.4: Slope Analysis at time = 65 days (18 days after completion of drawdown) with Induced Pore Pressures from Second Buttress Lift.