Settlement Analyses of Grade Supported Tanks Constructed with the Use of Prefabricated Wick Drains and an Earth Preload

Rebecca Elizabeth Scherer

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Settlement Analyses of Grade Supported Tanks
Constructed with the Use of Prefabricated Wick Drains and an Earth Preload

A Thesis

Submitted to the Graduate Faculty of the
University of New Orleans
in partial fulfillment of the
requirements for the degree of

Master of Science
in
Civil Engineering

by

Rebecca Elizabeth Scherer
B.S. University of New Orleans, 2007

May 2011
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ABSTRACT

In the design of tank foundations several design techniques are considered. This study focuses on grade supported tanks constructed under an extensive preload and instrumentation program. Settlement estimation methods were performed and compared to field instrumentation data taken at the project sites. Three project sites were selected for this study. The geotechnical investigations were performed by Eustis Engineering Services, L.L.C. and included both undisturbed soil borings and cone penetrometer tests. Conclusions were made about the accuracy of the calculations and how assumptions affect the settlement computation results.

Keywords: Settlement, Consolidation, Geotechnical, Tank Foundations, Wick Drains, Instrumentation
CHAPTER 1
INTRODUCTION

In the design of tank foundations several design techniques may be implemented. This study focuses on grade supported tanks constructed under an extensive preload and instrumentation program. Settlement estimation methods were performed for comparison purposes. Instrumentation data were available to compare the computed settlement values to the actual settlement that occurred at each site. As part of the literature review, consolidation theory, the determination of the coefficient of consolidation, and procedures for laboratory testing were researched and are discussed in this document. These items are essential in determining the soil parameters that represent the native soil conditions that allow the prediction of settlement estimation.

For this study, data were available for three project sites. All three sites are located in Southeastern Louisiana. The geology of the project sites consist of Holocene deposits that overlie Pleistocene Epoch soils. Each site consisted of several tanks of varying tank diameters. The tanks used for this study were constructed after an earthen preload program and extensive instrumentation program. Prefabricated wick drains were also used beneath the tanks to accelerate the rate of consolidation.

Settlement analyses were performed using stress distribution by Westergaard theory and the rate of consolidation by Terzaghi theory via spreadsheets. Settlement analyses were also performed using Settle3D software by RocScience. The results of these methods are compared to the actual occurring settlement. Discussion is included on the reasons for differences in the computed values and the recorded settlement.
2.1 SETTLEMENT

When soil is loaded by a structure, deformations will occur. Vertical deformation at the existing ground surface resulting from the structure load is termed as settlement. In the design of engineered structures, the amount of settlement and the rate at which the structure will settle are two aspects that are of interest. The total settlement of soil area being loaded has three components. These components are immediate settlement, consolidation settlement, and secondary settlement. The immediate settlement also referred to as distortion settlement is estimated using elastic theory. Consolidation settlement is time dependent and is a process that occurs in saturated fine grained soils with a low coefficient of permeability. The settlement rate is dependent on the rate of drainage of the pore water. Secondary compression occurs at a constant effective stress with no subsequent changes in the pore water pressure.

2.2 CONSOLIDATION THEORY

The amount of settlement the clay soil layers experience is directly related to how much pore water is squeezed out of the clay soil voids. This is a simplistic description of the consolidation process. The amount of water that has been moved out of the soil voids and the change in the void ratio of the clayey soils is then directly proportional to the amount of pore water pressure that has dissipated. Therefore, the settlement rate is directly related to the rate of excess pore water pressure. In order to predict the settlement rate, it is necessary to have a theory that can predict the pore pressure and void ratio at a specific time and space in the clay layer. The most commonly used theory of consolidation in soil mechanics is a one dimensional theory that was first developed by Karl Terzaghi in 1925. The Terzaghi theory is a strain theory where the applied load increments result in only small strains in the soils. This translates to both the coefficient of compressibility and the Darcy coefficient of permeability remain essentially constant during consolidation. Terzaghi one-dimensional consolidation equation is written as
\[ C_v \frac{d^2 u}{dz^2} = \frac{du}{dt} \quad (2-1) \]

where

\[ C_v = \frac{k}{\rho_w g} \frac{1 + e_o}{a_v} \quad (2-2) \]

The coefficient \( C_v \) is termed as the coefficient of consolidation. This coefficient contains the material properties that govern the process of consolidation.

There are a variety of ways to solve the Terzaghi consolidation equation and some are mathematically accurate while others are only approximations. M.E. Harr in 1966 presented an approximate solution by use of the method of finite differences. Following Terzaghi, D.W. Taylor in 1948 gave a rigorous mathematical solution in terms of Fourier series expansions. For the Taylor solution, the initial conditions are the compressible layer has complete drainage at the top and bottom and the initial excess hydrostatic pressure is equal to the applied increment of stress at the boundary. The solution is in terms of the Fourier series expansion as

\[ u = \left( \sigma'_2 - \sigma'_1 \right) \sum_{n=0}^{\infty} f_1(Z) f_2(T) \quad (2-3) \]

where \( Z \) is a geometry parameter equal to \( z/H \) and \( T \) is the time factor. The time factor is related to the coefficient of consolidation by

\[ T = C_v \frac{t}{H_{dr}^2} \quad (2-4) \]

where \( t = \) time and \( H_{dr} = \) the length of the longest drainage path.

The consolidation after some time, \( t \), and at any depth, \( z \), of the consolidating layer is related to the void ratio at that time and final change in void ratio. This is called the consolidation ratio and is expressed as

\[ U_Z = \frac{e_1 - e}{e_1 - e_2} \quad (2-5) \]
Putting the solution in terms of the consolidation ratio, the equation becomes

\[ U_z = 1 - \sum_{n=0}^{\infty} f_1(Z) f_2(T) \]  

(2-6)

This equation's solution is shown graphically on Figure 1. This figure allows the user to avoid the tedious calculations involved in the equation. The consolidation ratio can be determined from the figure by use of the coefficient of consolidation, the layer thickness, and the drainage conditions. Figure 1 also depicts the progress of consolidation. The lines for the time factors are called isochrones and represent the degree of consolidation for a given time factor within the compressible soil layer.

![Figure 1: Consolidation Ratio as a function of any location and time factor (Taylor, 1948)](image)

The coefficient of consolidation, the total thickness of the soil layer, and the drainage conditions can be used to calculate the time factor, \( T \). After the time factor has been computed, the consolidation ratio can be determined from Figure 1.
2.3 DETERMINATION OF THE COEFFICIENT OF CONSOLIDATION

The coefficient of consolidation is an important part of the consolidation equation because it takes into account the soil properties that govern the rate of consolidation. This coefficient generally decreases as the liquid limit of the soil increases. The approximate correlations of the coefficient of consolidation with liquid limit are shown on Figure 2.

![Figure 2: Approximate Correlations of the Coefficient of Consolidation, $c_v$ (Das, 2006)](image)

Casagrande and Taylor developed empirical procedures to approximately fit the observed laboratory data to the Terzaghi theory of consolidation. The curve fitting procedures are used to determine values of the coefficient of consolidation from the laboratory data as well as allow separation the secondary compression from the primary consolidation. These two graphical curve-fitting methods are commonly considered for the determination of the coefficient of consolidation. These methods are Casagrande's Logarithm of Time Fitting Method and Taylor's Square Root of Time Method. The determination of the coefficient of consolidation uses the later part of the consolidation
curve and are typically influenced by secondary compression. Secondary compression is concurrent with primary consolidation and it tends to decrease the value of the coefficient of consolidation. If the early part of the consolidation data is used, the values obtained will be less influenced by secondary compression effects.

In the Casagrande’s Logarithm of Time Fitting Method, the deformation dial readings are plotted versus logarithm of time. Several steps may be followed to interpret the laboratory test data to determine the coefficient of consolidation, \( c_v \). The first step is to extend the straight-line portions of primary and secondary consolidation to intersect at a point labeled “A”. The ordinate of point “A” is represented by the deformation at the end of 100% primary consolidation, \( d_{100} \). Next, the initial curved portion of the plot is approximated to be a parabola on the natural scale. Select two times, \( t_1 \) and \( t_2 \), on the curved portion such that \( t_2 = 4t_1 \). Thirdly, the horizontal line indicated as “DE” is drawn such that the vertical distance shown as “BD” is equal to the distance “x”. The ordinate of “BE” then gives the value of “\( d_o \)”. The ordinate of point ‘F’ on the consolidation curve represents the deformation at 50% primary consolidation and the abscissa represents the corresponding time. Finally, for 50% average degree of consolidation, the time factor equals 0.197 using the equations presented above. The coefficient of consolidation is then calculated by

\[
\frac{0.197 H_{dr}^2}{t_{50}}
\]

where \( H_{dr} \) is the average longest drainage path during consolidation. An example of the construction of the Logarithm of Time Method curve is shown in Figure 3.
In the Taylor’s Square Root of Time Fitting Method, a graph of the deformation against the square root of time is made for the incremental loading. As with Casagrande’s fitting method, the procedure is based on the similarity between the shapes of the theoretical and experimental curves when plotted versus time. The graph can be interpreted to find the coefficient of consolidation in a few steps. First, the line designated as “AB” is drawn through the early portion of the curve. Next, the line “AC” is drawn such that the segment “OC” equals 1.15 times the segment shown as “OB”. The abscissa of point “D” gives the square root of time for 90% of the consolidation. Finally, for 90% consolidation, the time factor equals 0.848. The coefficient of consolidation is then calculated by

\[
    c_v = \frac{0.848 H_{dr}^2}{t_{90}}
\]

(2-8)

where \( H_{dr} \) is the average longest drainage path during consolidation. An example of the construction of the Square Root of Time curve is shown in Figure 4.
2.4 SECONDARY SETTLEMENT

Secondary consolidation occurs after complete dissipation of excess pore water pressure; therefore at the end of the primary consolidation. When the deformation is plotted with the logarithm of time, the secondary consolidation portion of the graph is practically linear. This indicates the compression is occurring at a slower rate. Secondary consolidation differs from primary consolidation in that it takes place at a constant effective stress. It is often very difficult to separate secondary settlement from the primary settlement in the field. Both primary and secondary consolidation contributes to the total surface settlement and are often not separated for professional practice.

2.5 STRESS DISTRIBUTION

In 1885, J. Boussinesq developed equations for the state of stress within a homogeneous, isotropic, linearly elastic half-space for a point load acting perpendicular
to the surface. Naturally occurring soil deposits do not conform to these ideal material conditions. H.M. Westergaard in 1938 developed a solution for stresses at a point with varied horizontal soil layers. With Westergaard’s theory, an elastic soil is interspersed with infinitely thin but rigid layers that allow only vertical movement. Westergaard’s solution for vertical stress for a point load with a Poisson’s ratio of zero is

$$\sigma_z = \frac{Q}{z^2 \pi} \left[ \frac{1}{1 + 2 \left( \frac{r}{z} \right)^2} \right]^{3/2} \quad (2-9)$$

where

- $z =$ depth from the ground surface to the place of the stress
- $r =$ the horizontal distance from the point load to the place where the stress is desired.

This equation may be written as

$$\sigma_z = \frac{Q}{z^2} N_W \quad (2-10)$$

where $N_W$ is an influence factor that is a function of $r/z$.

The Boussineq and Westergaard theories are compared in Figure 5.

Figure 5: Relationship Between Boussinesq and Westergaard Theories (Holtz, 1981)
2.6 WICK DRAINS

Prefabricated vertical wick drains can be a cost-effective solution for accelerating the consolidation of fine grained soils and limiting long-term settlement. For structures constructed on soft soil, wick drains are used to remove the excess pore water, consolidate the compressible soil layers, and induce the consolidation settlement. Prefabricated vertical wick drains can be installed vertically to depths exceeding 200 feet. The vertical wick drains are usually placed in a triangular configuration of 3 to 12 feet depending on the desired consolidation time. As a result of this method of accelerating the consolidation process, uneven post-construction settlements can be virtually eliminated.

The wick drains function by forcing water to flow through the filter fabric of the wick drain and into the channels of the wick drain core where it can flow vertically out of the soil. This flow may be either up or down to intersecting natural drainage layers consisting of sand or to the surface where a sand drainage blanket or prefabricated horizontal strip drains are available. The water in the soil layers only needs to travel to the nearest prefabricated vertical wick drain to reach a free drainage path. The prefabricated vertical wick drain core is made of high quality flexible polypropylene which exhibits a large water flow capacity in the longitudinal direction of the core. Each vertical wick drain can provide a greater vertical discharge capacity than a 6 inch diameter sand column. The prefabricated vertical wick drain core is tightly wrapped in a geotextile filter made of spun-bonded polypropylene. This geotextile jacket has a very high water permeability while retaining the finest of soil particles. Both the core and geotextile jacket have high mechanical strength, a high degree of durability in most environments, and high resistance to chemicals, microorganisms, and bacteria. A wick drain is shown in Figure 6.
2.7 SHEAR STRENGTH TESTING

Several laboratory methods are available to determine the shear strength of soil samples. These laboratory tests include the direct shear test, triaxial test, direct simple shear test, plane strain triaxial test, and the torsional ring shear test. The direct shear test is the oldest and simplest form of shear testing. Although the direct shear test is simple to perform, the reliability of the test results are questionable because the soil is not allowed to fail along the weakest plane. The triaxial shear test is the most reliable form of shear testing. The three standard types of triaxial tests generally conducted are consolidated-drained test (CD), consolidated-undrained test (CU), and the unconsolidated-undrained test (UU and OB).

2.8 CONSOLIDATION TESTING

When a large area is loaded vertically, the compression that will occur can be assumed to be one dimensional. A consolidometer (also referred to as an oedometer) is used to simulate one-dimensional compression in a laboratory. An undisturbed soil sample representing the compressible soil layer is placed into a rigid confining ring to prevent lateral deformation. Porous stones are then placed at the top and bottom of the soil sample. The porous stones allow drainage during the consolidation process.
The soil specimen is usually 2.5 inches in diameter and 1 inch thick. Photographs of a prepared sample for testing are shown in Figure 7 (a and b).

![Figure 7: (a) Trimmed sample without porous stones. (b) Sample with porous stones](image)

The load on the specimen is applied through a lever arm and the compression is measured by a micrometer dial gauge. Each load is usually held for 24 hours after which the load is doubled. At the end of the consolidation test, the dry unit weight of the test specimen is determined. Figure 8 is a photograph a consolidometer while a test is in progress and Figure 9 depicts the general shape of the plot of deformation of the specimen against time for a given load increment.

![Figure 8: Consolidometer with a consolidation test in progress](image)
2.9 ATTERBERG LIMITS

Depending on the moisture content, the behavior of soil can be divided into four basic states: soil, semisolid, plastic, and liquid. The moisture content at the transition from soil to semisolid is the shrinkage limit, the moisture content from semisolid to plastic state is the plastic limit, and the moisture content from plastic to liquid state is the liquid limit. Collectively, these parameters are known as the Atterberg limits.

The plastic limit is the moisture content at which the soil crumbles when rolled by hand into threads approximately 1/8 inch in diameter. The plastic limit is the lower limit of the plastic stage of soil. The plastic limit test is performed by repetitively rolling an ellipsoidal sized soil sample by hand on a ground glass plate. The plastic limit test procedure is given in ASTM, Test Designation D-4318.

The liquid limit is determined by using a liquid limit device consisting of a brass cup and hard rubber base. The brass cup is dropped onto the base by a cam operated by a crank. The soil sample is formed into a paste and placed in the brass cup. A
groove is then cut at the center of the soil paste with a specific grooving tool. By use of the crank, the brass cup is lifted and dropped from a height of 0.394 inches. The moisture content required to close a distance of 0.5 inches along the bottom of the groove after 25 blows or drops is defined as the liquid limit.

Casagrande in 1932 studied the relationship of the plasticity index and the liquid limit of a variety of soils. The plasticity index is the difference between the liquid limit and the plastic limit of a soil. Based on his test results, Casagrande proposed the plasticity chart shown in Figure 10. The key feature of this plasticity chart is the A-line. The A-line separates the inorganic clays from the inorganic silts. The information provided in the chart is the basis for the classification of fine-grained soils in the Unified Soil Classification System. The U-line of the plasticity chart is the upper limit of the relationship of the plasticity index to the liquid limit for any soil. The equation for the A-line is given as \( \text{PI} = 0.73(\text{LL} - 20) \) and the equation for the U-line is given as \( \text{PI} = 0.90(\text{LL} - 8) \).

![Figure 10: Plasticity Chart (Das, 2006)](image)
3.1 INTRODUCTION

Three project sites were selected for this study. All three project sites consisted of a subsurface investigation performed with a combination of undisturbed soil borings and cone penetrometer tests (CPT).

3.2 SOIL BORINGS

The soil borings were made using a rotary drill rig mounted on an all-terrain vehicle. Undisturbed samples of cohesive or semi-cohesive subsoils were obtained at close intervals or changes in strata using a 3-in. diameter thin wall Shelby tube sampling barrel. The undisturbed samples were immediately extruded from the sampling barrel in the field while at the project site. Pocket penetrometer tests were also performed on trimmed ends of the extruded samples to provide a general indication of the soil's shear strength or consistency. The results of these tests are typically shown on the boring logs under a column heading "PP." All samples were inspected and visually classified by a soil technician in the field. Representative portions of the samples were then placed in moisture proof containers and returned to the laboratory for additional testing.

Cohesionless soils were obtained during the performance of in situ Standard Penetration Tests. This test consists of driving a 2-in. diameter split spoon sampler 1 foot into the soil after first seating the sampler 6 inches. A 140-lb weight dropped 30 inches is used to advance the sampler. The number of blows required to drive the sampler through the final 1-ft increment is indicative of the relative density or approximate consistency of the subsoils tested. The results of the Standard Penetration Tests are typically recorded on the boring log and shown under a column heading "SPT." Representative samples were also placed in moisture proof containers for preservation of their natural moisture content.
3.3 CONE PENETROMETER TESTS

Cone penetrometer tests (CPT) were performed using a 10-cm² cross-sectional area cone with a 60° apex angled tip and 150-cm² sleeve area. The penetrometer is hydraulically advanced into the ground at the rate of 2-cm/sec from a track-mounted unit. CPT parameters (tip resistance, friction resistance, and pore pressure) are recorded at 5-cm depth intervals. The results of the CPTs were then plotted graphically with depth. The plots provide corrected cone tip resistance \( (q_t) \), sleeve friction resistance \( (f_s) \), and pore pressure behind cone tip \( (u_2) \). Testing is performed in general accordance with methods and procedures outlined in ASTM D 5778-07. Upon completion of the CPTs, the holes were then backfilled in general accordance with current regulatory requirements.

3.4 LABORATORY TESTS

Soil mechanics laboratory tests were performed on the undisturbed samples obtained from the soil borings at the three project sites. Included in these laboratory tests were natural water content, unit weight, and unconfined compression shear (UC), and unconsolidated undrained triaxial compression shear (OB). In addition, Atterberg liquid and plastic limits were performed on selected representative samples. These tests are necessary to confirm the classification of the subsoils and provide the relative strength and compressibility of the subsoils. Consolidation tests were also performed on select samples obtained from the soil borings. The results of these laboratory tests are tabulated on the boring logs for each project site and are included in Appendices A, B, and C. The results of the consolidation tests are shown graphically on separate sheets following the boring logs.

3.5 PROJECT SITE 1

Five undisturbed sample type soil test borings (designated as B-1 through B-5) were drilled between 22 January and 2 February 2010. The borings were made using a
rotary drill rig mounted on an all-terrain vehicle. Borings 1,2, and 3 were each drilled to a depth of 100 feet below the existing ground surface, and Borings 4 and 5 were each drilled to a depth of 150 feet below the existing ground surface. Twenty CPTs (designated as CPT-1 through CPT-20) were performed between 27 January and 3 February 2010. The CPTs were made from a track mounted unit. The boring locations in relation to the proposed tanks and the logs of the borings and CPTs are included in Appendix A.

3.6 PROJECT SITE 2

Seven undisturbed sample type soil borings (designated as B-1 through B-7) were drilled between 24 February and 3 March 2010. The borings were made using a rotary drill rig mounted on an all-terrain vehicle. Borings 1,2,3, and 4 were each drilled to a depth of 100 feet below the existing ground surface, and Borings 5,6, and 7 were each drilled to a depth of 150 feet below the existing ground surface. Twenty-five CPTs were performed between 5 and 15 March 2010. The CPTs were made from a track mounted unit. The boring locations in relation to the proposed tanks and the logs of the borings and CPTs are included in Appendix B.

3.7 PROJECT SITE 3

Seven undisturbed sample type soil test borings (designated as T-19, T-20, T-21, T-23, T-24, T-31, and NT-31) were drilled between 6 April and 10 May 2010. The borings were made using a rotary drill rig mounted on an all-terrain vehicle. Borings T-20, T-21, T-23, and T-24 were each drilled to a depth of 100 feet below the existing ground surface, and Borings T-19, T-31, and NT-31 were each drilled to a depth of 150 feet below the existing ground surface. Thirty-five CPTs (designated as OTK19-1 through OTK19-4, TK19-1 through TK19-4, TK20-1 through TK20-5, TK21-1 through TK21-4, OTK22-1, OTK22-3, OTK22-4, TK22-1 through TK22-3, TK23-1 through TK23-4, TK24-1, TK24-2, TK24-3, and TK24-5, and TK31-1 through TK31-4) were performed
between 12 April and 21 May 2010. The CPTs were made from a track mounted unit. The boring locations in relation to the proposed tanks and the logs of the borings and CPTs are included in Appendix C.
CHAPTER 4  
EVALUATION OF DATA AND SETTLEMENT ANALYSES

4.1 INTRODUCTION

For this study, settlement analyses were performed by several methods for comparison purposes. Westergaard theory was executed by the use of spreadsheets and hand computations. By using spreadsheets and hand computations, the analyses were performed assuming a two-dimensional space. The computer software, Settle3D by RocScience, was also used. Westergaard theory was selected within the software to execute the computations. Settle3D is a three-dimensional program for the analysis of vertical consolidation and settlement beneath foundations and surface loads.

4.2 SETTLEMENT ANALYSES USING SPREADSHEET CALCULATIONS

Settlement analyses were performed using stress distribution by Westergaard theory and the rate of consolidation by Terzaghi theory. In order to simplify the calculations, a series of spreadsheets were set up to calculate time rate of settlement based on an applied load over the specified area. Wick drain calculations were also incorporated into the spreadsheets.

4.3 SETTLEMENT ANALYSES USING SETTLE3D SOFTWARE

Settlement analyses were performed utilizing Settle3D software by RocScience. The Settle3D program combines the simplicity of one-dimensional analysis with the visualization capabilities three-dimensional analyses. Modeling can be staged, and time-dependent consolidation analysis can be performed including primary and secondary consolidation (creep) at defined time intervals. A variety of linear and non-linear soil types can be modeled.
4.4 PROJECT SITE 1

Project site 1 consists of three 228-ft diameter and two 310-ft diameter crude oil storage tanks. The storage product has a specific gravity no greater than 0.92. An earth preload was placed atop the tank footprint for at least two months prior to construction of the tanks. In addition, consolidation settlement in the foundation soils was accelerated with the use of prefabricated vertical wick drains. Applicable figures for the earth preload and installation of wick drains are included in Appendix A.

The preload program has two benefits: it mitigates post construction settlement of the tanks, and it allows for storage of more product (i.e., larger bearing intensities). Preliminary analyses indicated the storage tanks at this site may be designed for a 3,100 psf bearing intensity in association with the preload. Considering a 2-ft thick tank pad and a specific gravity of 0.92, this corresponds to a maximum height of 50 feet of product. Prior to tank construction, wick drains were installed to the approximate 80-ft depth in a 5-ft triangular grid pattern, and an earth preload was placed atop the tank footprint for approximately two months. After removal of the earth preload, the tanks were constructed with an instrumentation and staged hydrotest program. The instrumentation included pore pressure transducers, slope inclinometers, and survey settlement points. Instrumentation readings were recorded and will be used to analyze the accuracy of the calculations performed for this study.

4.4.1 EARTH PRELOAD

An earth preload was used prior to construction of the tank foundations to facilitate consolidation of the subsoils and limit post construction settlement of the tanks. The preload also enabled larger bearing intensities (i.e., larger product storage heights) due to a gain in shear strength in the foundation soils. Stability analyses were performed to determine the maximum earth preload height that can be placed with the existing soil conditions. The slope stability analyses were performed using a program developed by the U.S. Army Corps of Engineers, New Orleans District, entitled "Slope
Based on the stability analyses, a maximum earth loading pressure of approximately 2,200 psf (average 20-ft fill height assuming a unit weight of 110 pcf for the fill material) can be placed using side slopes of 1 vertical to 3 horizontal while providing a factor of safety approximately equal to 1.1 against a slope failure. This factor of safety against a slope failure assumed strength gains in the subsoils would not occur as the earth preload was constructed. Based on an average fill height of 20 feet, residual settlement for tank foundations constructed after earth preloading will be reduced. In addition, due to the reduction in total tank settlement, preloading with earth reduces the potential for releveling after construction and performance of hydrotesting.

Assuming a 20-ft high earth preload is constructed and allowed to remain in place for a minimum of two months (with wick drains), settlement estimates of the foundation soils will be significantly reduced. A summary of settlement estimates for a 308 to 390-ft wide preload with an average 20-ft earth surcharge is shown in Table 1. These settlement estimates should be considered ground surface settlement realized at the top of the sand pad.

<table>
<thead>
<tr>
<th>TYPE OF ANALYSIS</th>
<th>TANK DIAMETER IN FEET</th>
<th>SETTLEMENT AT THE CENTER OF TANK</th>
<th>SETTLEMENT AT THE EDGE OF TANK</th>
</tr>
</thead>
<tbody>
<tr>
<td>WESTERGAARD THEORY BY SPREADSHEET</td>
<td>228</td>
<td>28 TO 43 INCHES</td>
<td>23 TO 35 INCHES</td>
</tr>
<tr>
<td></td>
<td>310</td>
<td>30 TO 45 INCHES</td>
<td>25 TO 38 INCHES</td>
</tr>
<tr>
<td>SETTLE3D SOFTWARE</td>
<td>228</td>
<td>30 TO 45 INCHES</td>
<td>25 TO 37 INCHES</td>
</tr>
<tr>
<td></td>
<td>310</td>
<td>32 TO 47 INCHES</td>
<td>27 TO 40 INCHES</td>
</tr>
</tbody>
</table>

4.4.2 WICK DRAINS

The use of wick drains in combination with an earth preload and a staged hydrotest loading program permits the application of bearing pressures up to 3,100 psf
(tank product heights no greater than 50 feet). It is estimated the required soil strengthening may be achieved in approximately two months. Wick drains should be installed in a triangular array with a center to center spacing of 5 feet to an approximate depth of 80 feet, or practical refusal.

4.4.3 WICK DRAIN INSTALLATION

After placement of approximately 1 foot of fill above the prepared subgrade, vertical wick drains were inserted through the fill pad. These vertical wicks were installed in an equilateral triangular pattern as shown in Appendix A. All wick drains were installed to an approximate depth of 80 feet, or practical refusal in the sand deposits using a mandrel that protects the wick drain during installation. These drains were also installed to a plumbness within 1% of vertical and within an area no more than 6 inches from the design location. In some instances vibratory assistance was used to advance the wick. Once each wick had been installed to the required depth and the mandrel has been withdrawn, the wick was cut to provide excess wick length at the ground surface. This excess length was then pinned or stapled to a horizontal wick drain in accordance with the manufacturer’s recommendations. The horizontal strip drains also meet the material requirements shown in Appendix A. The horizontal drains were arranged as shown on the figures in Appendix A, with no more than two rows of vertical wick drains connecting to each horizontal strip drain. Two 6-in. wide horizontal strip drains were also provided between the sand pad and the tank ring walls’ weep holes as shown on the figure in Appendix A.

4.4.4 ESTIMATED TANK SETTLEMENT AFTER EARTH PRELOAD

The estimated two-month duration for an earth preload (approximately 20 feet high comprising clay and a 2-ft thick sand pad) results in settlement of the sand pad and strength gain in the foundation soils. Settlement and strength gain is accelerated by the use of wick drains. These results are summarized in Table 1 for tank diameters of 228 and 310 feet. After the earth preload period elapses and it is determined that adequate consolidation has occurred, the preload material is removed, and the tanks are
constructed. Estimations of the amount of settlement that would be experienced for tank diameters of 228 and 310 feet after the earth preload period were performed by Westergaard theory using spreadsheets and the Settle3D software. These estimates are summarized in Table 2. Estimations of consolidation settlement at the center and edge of each tank are shown. Consolidation settlement will occur over a long period of time and at a diminishing rate. The maximum differential settlement is estimated as 4 to 4½ inches at the center and 1 to 1½ inches at the edge of a 228-ft diameter tank. The maximum differential settlement is estimated as 4½ to 5 inches at the center and 1 to 1½ inches at the edge of a 310-ft diameter tank. Elastic settlement and differential settlement at the center and edge of each tank were also estimated. Elastic settlement occurs instantaneously, and it is estimated no more than 1 inch of differential settlement at the center and at the edge for both tank sizes.

<table>
<thead>
<tr>
<th>TYPE OF ANALYSIS</th>
<th>TANK DIAMETER IN FEET</th>
<th>MAXIMUM SETTLEMENT IN INCHES</th>
<th>MINIMUM SETTLEMENT IN INCHES</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>CENTER OF TANK</td>
<td>EDGE OF TANK</td>
</tr>
<tr>
<td>WESTERGAARD THEORY BY SPREADSHEET</td>
<td>228</td>
<td>22.5 TO 23</td>
<td>6 TO 6.5</td>
</tr>
<tr>
<td></td>
<td>310</td>
<td>25 TO 25.5</td>
<td>6.5 TO 7</td>
</tr>
<tr>
<td>SETTLE3D SOFTWARE</td>
<td>228</td>
<td>25 TO 26</td>
<td>10 TO 11</td>
</tr>
<tr>
<td></td>
<td>310</td>
<td>30 TO 31</td>
<td>11 TO 12</td>
</tr>
</tbody>
</table>

4.5 PROJECT SITE 2

Project site 2 consists of four 135-ft diameter asphalt tanks and one 110-ft diameter MDO tanks. The asphalt product has a specific gravity no greater than 1.10 and the MDO product has a specific gravity of no greater than 0.92. An earth preload was placed atop the tank footprint for at least two months prior to construction of the tanks. In addition, consolidation settlement in the foundation soils was accelerated with the use of prefabricated vertical wick drains. Applicable figures for the earth preload and installation of wick drains are included in Appendix B.
As discussed in the previous section for project site 1, the preload program has two benefits: it mitigates post construction settlement of the tanks, and it allows for storage of more product (i.e., larger bearing intensities). Preliminary analyses indicated the storage tanks at this site may be designed for a 3,100 psf bearing intensity in association with the preload. Considering a 2-ft thick tank pad and a specific gravities of 1.10 and 0.92, this corresponds to a maximum heights of 42 and 51 feet of product, respectively. Prior to tank construction, wick drains were installed to the approximate 80-ft depth in a 5-ft triangular grid pattern, and an earth preload was placed atop the tank footprint for approximately two months. After removal of the earth preload, the tanks were constructed with an instrumentation and staged hydrotest program. The instrumentation included pore pressure transducers, slope inclinometers, and survey settlement points. Instrumentation readings were recorded and will be used to analyze the accuracy of the calculations performed for this study.

4.5.1 EARTH PRELOAD

An earth preload was used prior to construction of the tank foundations to facilitate consolidation of the subsoils and limit post construction settlement of the tanks. The preload also enabled larger bearing intensities (i.e., larger product storage heights) due to a gain in shear strength in the foundation soils. Stability analyses were performed to determine the maximum earth preload height that can be placed with the existing soil conditions. The slope stability analyses were performed using a program developed by the U.S. Army Corps of Engineers, New Orleans District, entitled "Slope Stability with Uplift Computations" (UPLIFT, Version 2.0, 4 August 1994, FS004) which utilizes the LMVD Method of Planes.

Based on the stability analyses, a maximum earth loading pressure of approximately 2,200 psf (average 20-ft fill height assuming a unit weight of 110 pcf for the fill material) can be placed using side slopes of 1 vertical to 3 horizontal while providing a factor of safety approximately equal to 1.1 against a slope failure. This factor of safety against a slope failure assumed strength gains in the subsoils would not occur as the earth preload was constructed. Based on an average fill height of 20 feet,
residual settlement for tank foundations constructed after earth preloading will be reduced. In addition, due to the reduction in total tank settlement, preloading with earth reduces the potential for releveling after construction and performance of hydrotesting.

Assuming a 20-ft high earth preload is constructed and allowed to remain in place for a minimum of two months (with wick drains), settlement estimates of the foundation soils will be significantly reduced. A summary of settlement estimates for a 190 to 215-ft wide preload with an average 20-ft earth surcharge is shown in Table 3. These settlement estimates should be considered ground surface settlement realized at the top of the sand pad.

<table>
<thead>
<tr>
<th>TYPE OF ANALYSIS</th>
<th>TANK DIAMETER IN FEET</th>
<th>SETTLEMENT AT THE CENTER OF TANK</th>
<th>SETTLEMENT AT THE EDGE OF TANK</th>
</tr>
</thead>
<tbody>
<tr>
<td>WESTERGAARD THEORY BY SPREADSHEET</td>
<td>110</td>
<td>26 TO 39 INCHES</td>
<td>15 TO 23 INCHES</td>
</tr>
<tr>
<td></td>
<td>135</td>
<td>28 TO 42 INCHES</td>
<td>22 TO 34 INCHES</td>
</tr>
<tr>
<td>SETTLE3D SOFTWARE</td>
<td>110</td>
<td>28 TO 42 INCHES</td>
<td>17 TO 25 INCHES</td>
</tr>
<tr>
<td></td>
<td>135</td>
<td>30 TO 44 INCHES</td>
<td>25 TO 38 INCHES</td>
</tr>
</tbody>
</table>

4.5.2 WICK DRAINS

The use of wick drains in combination with an earth preload and a staged hydrotest loading program permits the application of bearing pressures up to 3,100 psf (tank product heights no greater than 51 feet for the MDO product and 42 feet for the asphalt product). It is estimated the required soil strengthening may be achieved in approximately two months. Wick drains should be installed in a triangular array with a center to center spacing of 5 feet to an approximate depth of 80 feet, or practical refusal.

4.5.3 WICK DRAIN INSTALLATION

After placement of approximately 1 foot of fill above the prepared subgrade, vertical wick drains were inserted through the fill pad. These vertical wicks were
installed in an equilateral triangular pattern as shown in Appendix B. All wick drains were installed to an approximate depth of 80 feet, or practical refusal in the sand deposits using a mandrel that protects the wick drain during installation. These drains were also installed to a plumbness within 1% of vertical and within an area no more than 6 inches from the design location. In some instances vibratory assistance was used to advance the wick. Once each wick had been installed to the required depth and the mandrel has been withdrawn, the wick was cut to provide excess wick length at the ground surface. This excess length was then pinned or stapled to a horizontal wick drain in accordance with the manufacturer’s recommendations. The horizontal strip drains also meet the material requirements shown in Appendix B. The horizontal drains were arranged as shown on the figures in Appendix B, with no more than two rows of vertical wick drains connecting to each horizontal strip drain. Two 6-in. wide horizontal strip drains were also provided between the sand pad and the tank ring walls’ weep holes as shown on the figure in Appendix B.

4.5.4 ESTIMATED TANK SETTLEMENT AFTER EARTH PRELOAD

The estimated two-month duration for an earth preload (approximately 20 feet high comprising clay and a 2-ft thick sand pad) results in settlement of the sand pad and strength gain in the foundation soils. Settlement and strength gain is accelerated by the use of wick drains. These results are summarized in Table 3 for tank diameters of 110 and 135 feet. After the earth preload period elapses and it is determined that adequate consolidation has occurred, the preload material is removed, and the tanks are constructed. Estimations of the amount of settlement that would be experienced for tank diameters of 110 and 135 feet after the earth preload period were performed by Westergaard theory using spreadsheets and the Settle3D software. These estimates are summarized in Table 4. Estimations of consolidation settlement and differential settlement at the center and edge of each tank are shown. Consolidation settlement will occur over a long period of time and at a diminishing rate. The maximum differential settlement is estimated as 3½ to 4 inches at the center and 1 to 1½ inches at the edge of a 135-ft diameter tank. The maximum differential settlement is estimated as 3 to 3½
inches at the center and \( \frac{1}{2} \) to 1 inch at the edge of a 110-ft diameter tank. Elastic settlement and differential settlement at the center and edge of each tank were also estimated. Elastic settlement occurs instantaneously, and it is estimated no more than 1 inch of differential settlement at the center and at the edge for both tank sizes.

**TABLE 4: ESTIMATED CONSOLIDATION SETTLEMENT AFTER TANK CONSTRUCTION**

<table>
<thead>
<tr>
<th>TYPE OF ANALYSIS</th>
<th>TANK DIAMETER IN FEET</th>
<th>MAXIMUM SETTLEMENT IN INCHES</th>
<th>MINIMUM SETTLEMENT IN INCHES</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>CENTER OF TANK</td>
<td>EDGE OF TANK</td>
</tr>
<tr>
<td>WESTERGAARD THEORY BY SPREADSHEET</td>
<td>110</td>
<td>17 TO 17.5</td>
<td>3 TO 3.5</td>
</tr>
<tr>
<td></td>
<td>135</td>
<td>19.5 TO 20</td>
<td>5.5 TO 6</td>
</tr>
<tr>
<td>SETTLE3D SOFTWARE</td>
<td>228</td>
<td>20 TO 21</td>
<td>4 TO 5</td>
</tr>
<tr>
<td></td>
<td>310</td>
<td>22 TO 23</td>
<td>7 TO 8</td>
</tr>
</tbody>
</table>

4.6 PROJECT SITE 3

Project site 3 consists of five 228-ft diameter and two 150-ft diameter crude oil tanks. The storage product has a specific gravity no greater than 0.92. An earth preload was placed atop the tank footprint for at least two months prior to construction of the tanks. In addition, consolidation settlement in the foundation soils was accelerated with the use of prefabricated vertical wick drains. Applicable figures for the earth preload and installation of wick drains are included in Appendix C.

As discussed in the previous section for project sites 1 and 2, the preload program has two benefits: it mitigates post construction settlement of the tanks, and it allows for storage of more product (i.e., larger bearing intensities). Preliminary analyses indicated the storage tanks at this site may be designed for a 2,900 psf bearing intensity in association with the preload. Considering a 2-ft thick tank pad and a specific gravity of 0.92, this corresponds to a maximum heights of 46 feet of product. Prior to tank construction, wick drains were installed to the approximate 80-ft depth in a 5-ft triangular grid pattern, and an earth preload was placed atop the tank footprint for approximately two months. After removal of the earth preload, the tanks were constructed with an instrumentation and staged hydrotest program. The instrumentation included pore
pressure transducers, slope inclinometers, and survey settlement points. Instrumentation readings were recorded and will be used to analyze the accuracy of the calculations performed for this study.

4.6.1 EARTH PRELOAD

An earth preload was used prior to construction of the tank foundations to facilitate consolidation of the subsoils and limit post construction settlement of the tanks. The preload also enabled larger bearing intensities (i.e., larger product storage heights) due to a gain in shear strength in the foundation soils. Stability analyses were performed to determine the maximum earth preload height that can be placed with the existing soil conditions. The slope stability analyses were performed using a program developed by the U.S. Army Corps of Engineers, New Orleans District, entitled "Slope Stability with Uplift Computations" (UPLIFT, Version 2.0, 4 August 1994, FS004) which utilizes the LMVD Method of Planes.

Based on the stability analyses, a maximum earth loading pressure of approximately 2,915 psf (average 26.5-ft fill height assuming a unit weight of 110 pcf for the fill material) can be placed using side slopes of 1 vertical to 3 horizontal while providing a factor of safety approximately equal to 1.1 against a slope failure. This factor of safety against a slope failure assumed strength gains in the subsoils would not occur as the earth preload was constructed. Based on an average fill height of 26.5 feet, residual settlement for tank foundations constructed after earth preloading will be reduced. In addition, due to the reduction in total tank settlement, preloading with earth reduces the potential for releveling after construction and performance of hydrotesting.

Assuming a 26.5-ft high earth preload is constructed and allowed to remain in place for a minimum of two months (with wick drains), settlement estimates of the foundation soils will be significantly reduced. A summary of settlement estimates for a 306 to 384-ft wide preload with an average 26.5-ft earth surcharge is shown in Table 5. These settlement estimates should be considered ground surface settlement realized at the top of the sand pad.
<table>
<thead>
<tr>
<th>TYPE OF ANALYSIS</th>
<th>TANK DIAMETER IN FEET</th>
<th>SETTLEMENT AT THE CENTER OF TANK</th>
<th>SETTLEMENT AT THE EDGE OF TANK</th>
</tr>
</thead>
<tbody>
<tr>
<td>WESTERGAARD THEORY BY SPREADSHEET</td>
<td>110</td>
<td>55 TO 82 INCHES</td>
<td>46 TO 69 INCHES</td>
</tr>
<tr>
<td></td>
<td>135</td>
<td>59 TO 88 INCHES</td>
<td>50 TO 76 INCHES</td>
</tr>
<tr>
<td>SETTLE3D SOFTWARE</td>
<td>110</td>
<td>57 TO 84 INCHES</td>
<td>48 TO 71 INCHES</td>
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<tr>
<td></td>
<td>135</td>
<td>62 TO 89 INCHES</td>
<td>52 TO 78 INCHES</td>
</tr>
</tbody>
</table>

4.6.2 WICK DRAINS

The use of wick drains in combination with an earth preload and a staged hydrotest loading program permits the application of bearing pressures up to 2,900 psf (tank product heights no greater than 46 feet). It is estimated the required soil strengthening may be achieved in approximately two months. Wick drains should be installed in a triangular array with a center to center spacing of 5 feet to an approximate depth of 80 feet, or practical refusal.

4.6.3 WICK DRAIN INSTALLATION

After placement of approximately 1 foot of fill above the prepared subgrade, vertical wick drains were inserted through the fill pad. These vertical wicks were installed in an equilateral triangular pattern as shown in Appendix C. All wick drains were installed to an approximate depth of 80 feet, or practical refusal in the sand deposits using a mandrel that protects the wick drain during installation. These drains were also installed to a plumbness within 1% of vertical and within an area no more than 6 inches from the design location. In some instances vibratory assistance was used to advance the wick. Once each wick had been installed to the required depth and the mandrel has been withdrawn, the wick was cut to provide excess wick length at the ground surface. This excess length was then pinned or stapled to a horizontal wick drain in accordance with the manufacturer’s recommendations. The horizontal strip drains also meet the material requirements shown in Appendix C. The horizontal drains were arranged as shown on the figures in Appendix C, with no more than two rows of
vertical wick drains connecting to each horizontal strip drain. Two 6-in. wide horizontal strip drains were also provided between the sand pad and the tank ring walls’ weep holes as shown on the figure in Appendix C.

4.6.4 ESTIMATED TANK SETTLEMENT AFTER EARTH PRELOAD

The estimated two-month duration for an earth preload (approximately 26.5 feet high comprising clay and a 2-ft thick sand pad) results in settlement of the sand pad and strength gain in the foundation soils. Settlement and strength gain is accelerated by the use of wick drains. These results are summarized in Table 5 for tank diameters of 228 and 150 feet. After the earth preload period elapses and it is determined that adequate consolidation has occurred, the preload material is removed, and the tanks are constructed. Estimations of the amount of settlement that would be experienced for tank diameters of 228 and 150 feet after the earth preload period were performed by Westergaard theory using spreadsheets and the Settle3D software. These estimates are summarized in Table 6. Estimations of consolidation settlement and differential settlement at the center and edge of each tank are shown. Consolidation settlement will occur over a long period of time and at a diminishing rate. The maximum differential settlement is estimated as 4 to 5 inches at the center and 2 to 3 inches at the edge of a 228-ft diameter tank. The maximum differential settlement is estimated as 1½ to 2½ inches at the center and 1 to 2 inch at the edge of a 150-ft diameter tank. Elastic settlement and differential settlement at the center and edge of each tank were also estimated. Elastic settlement occurs instantaneously, and it is estimated no more than 1 inch of differential settlement at the center and at the edge for both tank sizes.
TABLE 6: ESTIMATED CONSOLIDATION SETTLEMENT AFTER TANK CONSTRUCTION

<table>
<thead>
<tr>
<th>TYPE OF ANALYSIS</th>
<th>TANK DIAMETER IN FEET</th>
<th>MAXIMUM SETTLEMENT IN INCHES</th>
<th>MINIMUM SETTLEMENT IN INCHES</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>CENTER OF TANK</td>
<td>EDGE OF TANK</td>
</tr>
<tr>
<td>WESTERGAARD THEORY BY SPREADSHEET</td>
<td>150</td>
<td>14.5 TO 15</td>
<td>3 TO 3.5</td>
</tr>
<tr>
<td></td>
<td>228</td>
<td>20.5 TO 21</td>
<td>11 TO 11.5</td>
</tr>
<tr>
<td>SETTLE3D SOFTWARE</td>
<td>150</td>
<td>18 TO 19</td>
<td>5 TO 8</td>
</tr>
<tr>
<td></td>
<td>228</td>
<td>22 TO 24</td>
<td>13 TO 14</td>
</tr>
</tbody>
</table>

4.7 INSTRUMENTATION

The use of instrumentation is recommended to monitor the effectiveness of the earth preload prior to tank construction and to monitor the tanks during hydrotesting. These instruments are monitored and data retrieved and evaluated on a periodic basis during the earth preload and hydrotest. The readings obtained during the instrumentation program are presented in Appendices A, B, and C for each project site.

Once the subgrade is prepared and the tank pad is installed, settlement plates are installed prior to the placement of the earth preload materials. Settlement plates are an instrumentation tool used to evaluate settlement of the ground surface due to placement of earth preload clay fill soils. The settlement plates are located at the center and edge of the crown of each preload configuration. The elevation of the settlement plate and riser is determined prior to any fill placement using a benchmark sufficiently removed from the surcharge area so as not to be influenced by the fill. Once this initial elevation is determined, the plate should not be disturbed during fill placement and compaction.

Vibrating wire type pore pressure transducers are also installed beneath each tank pad. These transducers are read during the earth preload and hydrotest. The leads are buried and threaded through the tank ringwall to a remote reading station. A minimum of three nests of pore pressure transducers are installed. One nest is installed at the centers of the proposed tanks, and the second is installed at a point approximately one-half to two-thirds of the tank radius from each tank’s center. A third
nest is installed outside the tank footprint. Each transducer nest should consist of three vibrating wire pore pressure transducers. The transducers at each nest are installed at depths of approximately 15, 30, and 45 feet below the existing ground surface. Transducers are monitored weekly (at a minimum) during the earth preload program, at the end of filling to a given stage loading of the hydrotest, and approximately twice weekly during all holding periods of the hydrotest.

Lateral flow may produce large deformations of the tank without strength gains of the foundation soils. Therefore, a minimum of four inclinometers are spaced at 90 degree intervals along the outside of each tank. The inclinometers are set to a depth of approximately 100 feet. All inclinometers are monitored once at the end of filling for each stage of the hydrotest and approximately twice weekly thereafter.

Level surveys of the tank ringwall are performed in conjunction with the subsurface instrumentation. A minimum of 16 survey points are equally spaced around the perimeter of each tank. These survey points will be used in conjunction with the subsurface instrumentation to evaluate the performance of the stage loading process. These settlements are then referenced to an established benchmark prior to the stage loading. The benchmark should be located a minimum of 500 feet from the nearest tank so the benchmark elevation is not affected by tank loading. The settlement points are monitored at the end of filling to a given stage loading, and approximately twice weekly during all holding periods of the hydrotest.
Tanks foundations may be constructed utilizing different computational techniques. This study focused on grade supported tanks constructed under an earthen preload and instrumentation program. Settlement estimation methods were performed to compare the results to the actual settlement that has occurred to date. Instrumentation data were available to compare the computed settlement values to the actual settlement that has occurred at each site. Settlement analyses were performed using stress distribution by Westergaard theory and the rate of consolidation by Terzaghi theory via spreadsheets. Settlement analyses were also performed using Settle3D software by RocScience. Based on the results, the two methods are within reasonable margins of each other for the tank structures. Because the tank structure construction is not complete, the settlement values reported in this document cannot be directly compared to the settlement readings available to date. The settlement values shown in Chapter 4 are ultimate settlements that will occur approximately 50 to 60 years from the date of initial construction. However, calculations were performed with the appropriate time lapses and the Settle3D model was within ten percent of the actual readings taken from the field instrumentation.

The Settle3D model tends to predict slightly larger amounts of settlement. The variation in results is due to the many assumptions that are made for both methods. The variation in assumptions can lead to great variance in settlement predictions. When predicting settlement, it is important to identify the state of soil and determine if each soil strata is experiencing virgin compression or has reached the zone of recompression. When estimating time rate of settlement, it is essential to have adequate consolidation test results for compressible soil zones. The rate of consolidation and consolidation coefficients determined from the test results will drastically help with the predictions of consolidation settlement. Another factor to consider is the amount of soil disturbance experienced during construction. For example, the installation of prefabricated wick drains can have a large impact on the rate of consolidation.
REFERENCES


DENOTES LOCATION OF CONE PENETROMETER TESTS PERFORMED:
27 JANUARY THROUGH 3 FEBRUARY 2010

DENOTES LOCATION OF UNDISTURBED SOIL BORINGS DRILLED:
22 JANUARY THROUGH 2 FEBRUARY 2010
PROPOSED 228' DIAMETER TANK

6" DISCHARGE PIPE
PRELOAD CONSTRUCTION
ACCESS PATH

TRIANGULAR PATTERN OF WICK DRAINS
THROUGHOUT TANK FOOTPRINT

COLLECTION DITCH
PRELOAD CONSTRUCTION
ACCESS PATH

PLAN VIEW

SAND DRAINAGE PAD

SECTION A-A

NOTE: DRAWING INDICATES SCHEMATIC OF
PROPOSED RECOMMENDATIONS. THE
LIMITS OF THE EARTH PRELOAD MAY BE
PROPORTIONED ACCORDING TO
ALTERNATE TANK DIAMETERS

SECTION A-A

NOT TO SCALE
NOTE: DRAWING INDICATES SCHEMATIC OF PROPOSED RECOMMENDATIONS. THE LIMITS OF THE EARTH PRELOAD MAY BE PROPORTIONED ACCORDING TO ALTERNATE TANK DIAMETERS
LEGEND AND NOTES FOR
LOG OF BORING AND TEST RESULTS

PP  Pocket penetrometer: Resistance in tons per square foot

SPT  Standard Penetration Test: Number of blows of a 140-lb hammer dropped 30 inches required to drive 2-in. O.D., 1.4-in. I.D. sampler a distance of 1 foot into the soil after first seating it 6 inches

SPLR  Type of Sampling  □ Shelby  □ SPT  □ Auger  □ No sample

SYMBOL
Clay  Silt  Sand  Peat/Humus  Shells  Stone/Gravel

Predominant type shown heavy; Modifying type shown light

USC  Unified Soil Classification

DENSITY  Unit weight in pounds per cubic foot

SHEAR TESTS

TYPE
UC  Unconfined compression shear
OB  Unconsolidated undrained triaxial compression shear on one specimen confined at the approximate overburden pressure
UU  Unconsolidated undrained triaxial compression shear
CU  Consolidated undrained triaxial compression shear
DS  Direct shear

Ø  Angle of internal friction in degrees

C  Cohesion in pounds per square foot

ATTERBERG LIMITS

LL  Liquid Limit
PL  Plastic Limit
PI  Plasticity Index

OTHER TESTS

CON  Consolidation
PD  Particle size distribution (sieve and/or hydrometer)

k  Coefficient of permeability in centimeters per second
SP  Swelling pressure in pounds per square foot

Other laboratory test results reported on separate figures
<table>
<thead>
<tr>
<th>Scale in Feet</th>
<th>PP</th>
<th>SPT</th>
<th>Symbol</th>
<th>Visual Classification</th>
<th>USC</th>
<th>Sample Number</th>
<th>Depth in Feet</th>
<th>Water-Content Percent</th>
<th>Density</th>
<th>Shear Tests</th>
<th>Atterberg Limits</th>
<th>Other Tests</th>
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<td>0</td>
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<td></td>
<td>Stiff gray &amp; tan clay w/wood, silty sand pockets, &amp; trace of roots</td>
<td>CH</td>
<td>1</td>
<td>0-2.67</td>
<td>33</td>
<td>88</td>
<td>UC</td>
<td>1158</td>
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<tr>
<td>0.50</td>
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<td></td>
<td></td>
<td>Medium stiff gray &amp; tan sandy clay w/trace of decayed wood</td>
<td>CL</td>
<td>2</td>
<td>5-6</td>
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<td>90</td>
<td>UC</td>
<td>655</td>
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</tr>
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<td>Soft gray silty clay</td>
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<td>20</td>
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**Boring:**

Refer to "Legends & Notes"

**Comments:**

A-50
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## LOG OF BORING AND TEST RESULTS

(Sheet 2 of 3)

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A-54
## LOG OF BORING AND TEST RESULTS

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**Job No.:**

**Date Drilled:** 1/25 & 27/10

**Boring:** T-15 B-4

Refer to "Legends & Notes"

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<th>Visual Classification</th>
<th>USC</th>
<th>Sample Number</th>
<th>Depth in Feet</th>
<th>Water Content Percent</th>
<th>Density</th>
<th>Shear Tests</th>
<th>Atterberg Limits</th>
<th>Other Tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>0.25</td>
<td>30</td>
<td></td>
<td>Medium stiff tan &amp; gray sandy clay</td>
<td>CL</td>
<td>23</td>
<td>103-104</td>
<td>31</td>
<td>91</td>
<td>119</td>
<td>UC -- 601</td>
<td>35</td>
</tr>
<tr>
<td></td>
<td></td>
<td>30</td>
<td></td>
<td>Stiff gray sandy clay</td>
<td>CL</td>
<td>24</td>
<td>106-107</td>
<td>26</td>
<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td></td>
<td></td>
<td>30</td>
<td></td>
<td>Medium dense gray silty sand</td>
<td>SM</td>
<td>25</td>
<td>109-110</td>
<td>28</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>35</td>
<td></td>
<td>Stiff gray sandy clay</td>
<td>CL</td>
<td>26</td>
<td>112-113</td>
<td>28</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td></td>
<td></td>
<td>34</td>
<td></td>
<td>Soft gray sandy clay</td>
<td>CL</td>
<td>27</td>
<td>115-118</td>
<td>27</td>
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</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Loose gray silty sand w/clay pockets &amp; layers</td>
<td>SM</td>
<td>28</td>
<td>118-119</td>
<td>25</td>
<td>102</td>
<td>126</td>
<td>UC -- 420</td>
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<tr>
<td>70=3'</td>
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<td></td>
<td></td>
<td>Very stiff gray sandy clay</td>
<td>CL</td>
<td>29</td>
<td>123-124</td>
<td></td>
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<td></td>
</tr>
<tr>
<td>66=4'</td>
<td></td>
<td></td>
<td></td>
<td>Very dense gray fine sand</td>
<td>SP</td>
<td>30</td>
<td>129-130</td>
<td>25</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>63=1'</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>31</td>
<td>134-135</td>
<td>25</td>
<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td>60=1'</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>32</td>
<td>139-140</td>
<td>26</td>
<td></td>
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<tr>
<td>47=5'</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>33</td>
<td>144-145</td>
<td>25</td>
<td></td>
<td></td>
<td></td>
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<td>34</td>
<td>149-150</td>
<td></td>
<td></td>
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<td></td>
</tr>
</tbody>
</table>

**Ground Elev.:**

**Gr. Water Depth:** See Text

**Job No.:**

**Date Drilled:** 1/28/10

**Boring:** T-14 B-5

Refer to "Legends & Notes"

**Comments:**

A-58
CONSOLIDATION TEST REPORT

<table>
<thead>
<tr>
<th>Natural Saturation</th>
<th>Moisture</th>
<th>Dry Dens. (pcf)</th>
<th>LL</th>
<th>PI</th>
<th>Sp. Gr.</th>
<th>P_c (tsf)</th>
<th>C_c</th>
<th>Initial Void Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>99.6 %</td>
<td>25.3 %</td>
<td>99.0</td>
<td>39</td>
<td>25</td>
<td>2.69</td>
<td>9.54</td>
<td>0.30</td>
<td>0.684</td>
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</tbody>
</table>

MATERIAL DESCRIPTION

MST GRN-G SACL

<table>
<thead>
<tr>
<th>USCS</th>
<th>AASHTO</th>
</tr>
</thead>
<tbody>
<tr>
<td>CL</td>
<td></td>
</tr>
</tbody>
</table>

Project:

Source: T14-5

Sample No.: 18

Elev./Depth: 78.0'

Remarks:
TESTED BY: RR
CHECKED BY: RNE

Figure
Dial Reading vs. Time

Source: T14-5
Sample No.: 18
Elev./Depth: 78.0'

**Load No. 1**
Load = 0.49 tsf

- $D_0 = 0.00027$
- $D_{50} = 0.00202$
- $D_{100} = 0.00376$
- $T_{50} = 0.36$ min.

**$C_v @ T_{50}$**
0.85 ft.²/day

$C_{\alpha} = 0.001$

**Load No. 2**
Load = 0.97 tsf

- $D_0 = 0.00429$
- $D_{50} = 0.00632$
- $D_{100} = 0.00835$
- $T_{50} = 0.97$ min.

**$C_v @ T_{50}$**
0.31 ft.²/day

$C_{\alpha} = 0.001$

EUSTIS ENGINEERING COMPANY, INC.
Dial Reading vs. Time

Source: T14-5
Sample No.: 18
Elev./Depth: 78.0'

Load No. = 3
Load = 1.93 tsf
\[ D_0 = 0.00884 \]
\[ D_{50} = 0.01159 \]
\[ D_{100} = 0.01435 \]
\[ T_{50} = 1.01 \text{ min.} \]

\[ C_v @ T_{50} \]
0.30 ft.²/day
\[ C_\alpha = 0.001 \]

Load No. = 4
Load = 3.85 tsf
\[ D_0 = 0.01612 \]
\[ D_{50} = 0.01976 \]
\[ D_{100} = 0.02341 \]
\[ T_{50} = 0.88 \text{ min.} \]

\[ C_v @ T_{50} \]
0.33 ft.²/day
\[ C_\alpha = 0.001 \]
Dial Reading vs. Time

Source: T14-5  
Sample No.: 18  
Elev./Depth: 78.0'

Load No. = 5
Load = 7.69 tsf
D₀ = 0.02534
D₅₀ = 0.03179
D₁₀₀ = 0.03823
T₅₀ = 1.18 min.

Cᵥ @ T₅₀
0.24 ft.²/day
Cₒ = 0.003

---

Load No. = 6
Load = 15.37 tsf
D₀ = 0.04220
D₅₀ = 0.05451
D₁₀₀ = 0.06683
T₅₀ = 3.81 min.

Cᵥ @ T₅₀
0.07 ft.²/day
Cₒ = 0.006

---

EUSTIS ENGINEERING COMPANY, INC.
Dial Reading vs. Time

Source: T14-5
Sample No.: 18
Elev./Depth: 78.0'

Load No. = 7
Load = 30.73 tsf
$D_0 = 0.07347$
$D_{50} = 0.09099$
$D_{100} = 0.10851$
$T_{50} = 8.35$ min.

$C_v @ T_{50}$
0.03 ft.²/day

$C_\alpha = 0.009$

Load No. = 8
Load = 45.21 tsf
$D_0 = 0.11202$
$D_{50} = 0.12255$
$D_{100} = 0.13309$
$T_{50} = 15.64$ min.

$C_v @ T_{50}$
0.01 ft.²/day

$C_\alpha = 0.009$

EUSTIS ENGINEERING COMPANY, INC.
Dial Reading vs. Time

Source: T14-5 Sample No.: 18 Elev./Depth: 78.0'

Load No. = 9
Load = 15.37 tsf
D₀ = 0.13607
D₅₀ = 0.13348
D₅₀₀ = 0.13089
T₅₀ = 1.13 min.

Cᵥ @ T₅₀
0.19 ft.²/day

Load No. = 10
Load = 3.85 tsf
D₀ = 0.12733
D₅₀ = 0.11932
D₅₀₀ = 0.11131
T₅₀ = 17.00 min.

Cᵥ @ T₅₀
0.01 ft.²/day
Dial Reading vs. Time

Source: T14-5  
Sample No.: 18  
Elev./Depth: 78.0'

Load No. = 11
Load = 0.97 tsf
\(D_0 = 0.11214\)
\(D_{50} = 0.10165\)
\(D_{100} = 0.09116\)
\(T_{50} = 46.48\) min.

\(C_v \text{ at } T_{50}\)
0.01 ft.\(^2\)/day
CONSOLIDATION TEST REPORT

![Graph showing void ratio versus applied pressure and CV vs applied pressure]

<table>
<thead>
<tr>
<th>Natural</th>
<th>Dry Dens. (pcf)</th>
<th>LL</th>
<th>PI</th>
<th>Sp. Gr.</th>
<th>$P_c$ (tsf)</th>
<th>$C_c$</th>
<th>Initial Void Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Saturation</td>
<td>99.8%</td>
<td>26.9%</td>
<td>97.2</td>
<td>33</td>
<td>16</td>
<td>2.70</td>
<td>4.93</td>
</tr>
</tbody>
</table>

MATERIAL DESCRIPTION

SO G SICL

USCS | CL

Proje
Projé

Source: T15-2

Sample No.: 7
Elev./Depth: 19.0'

Remarks:
TESTED BY: RR, LWR
CHECKED BY: RNE

Figure
Dial Reading vs. Time

Source: T15-2  Sample No.: 7  Elev./Depth: 19.0'

Load No. = 1
Load = 0.12 tsf

\[
\begin{align*}
D_0 &= 0.00206 \\
D_{90} &= 0.01737 \\
D_{100} &= 0.01907 \\
T_{90} &= 11.80 \text{ min.}
\end{align*}
\]

\[C_V @ T_{90} = 0.12 \text{ ft.}^2/\text{day}\]

---

Load No. = 2
Load = 0.25 tsf

\[
\begin{align*}
D_0 &= 0.01832 \\
D_{90} &= 0.02075 \\
D_{100} &= 0.02102 \\
T_{90} &= 4.58 \text{ min.}
\end{align*}
\]

\[C_V @ T_{90} = 0.29 \text{ ft.}^2/\text{day}\]
Dial Reading vs. Time

Source: T15-2  Sample No.: 7  Elev./Depth: 19.0'

Load No. = 3  
Load = 0.49 tsf  
D₀ = 0.02257  
D₉₀ = 0.02599  
D₁₀₀ = 0.02637  
T₉₀ = 2.89 min.

CV @ T₉₀  
0.46 ft²/day

Load No. = 4  
Load = 0.97 tsf  
D₀ = 0.02788  
D₉₀ = 0.03229  
D₁₀₀ = 0.03278  
T₉₀ = 1.44 min.

CV @ T₉₀  
0.91 ft²/day

EUSTIS ENGINEERING COMPANY, INC.
Dial Reading vs. Time

Source: T15-2  Sample No.: 7  Elev./Depth: 19.0'

<table>
<thead>
<tr>
<th>Load No.</th>
<th>5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load</td>
<td>1.93 tsf</td>
</tr>
<tr>
<td>$D_0$</td>
<td>0.03585</td>
</tr>
<tr>
<td>$D_{90}$</td>
<td>0.04150</td>
</tr>
<tr>
<td>$D_{100}$</td>
<td>0.04213</td>
</tr>
<tr>
<td>$T_{90}$</td>
<td>1.25 min.</td>
</tr>
<tr>
<td>$C_v @ T_{90}$</td>
<td>1.02 ft.$^2$/day</td>
</tr>
</tbody>
</table>

Load No. = 6
Load= 3.85 tsf
$D_0$ = 0.04549
$D_{90}$ = 0.05099
$D_{100}$ = 0.05160
$T_{90}$ = 0.68 min.

$C_v @ T_{90}$
1.85 ft.$^2$/day
Dial Reading vs. Time

Source: T15-2
Sample No.: 7
Elev./Depth: 19.0'

Load No. = 7
Load = 7.69 tsf
D0 = 0.05755
D50 = 0.06393
D100 = 0.06464
T90 = 0.61 min.

CV @ T90
1.98 ft.²/day

Load No. = 8
Load = 15.37 tsf
D0 = 0.07153
D90 = 0.07872
D100 = 0.07952
T90 = 0.57 min.

CV @ T90
2.03 ft.²/day
Dial Reading vs. Time

Source: T15-2
Sample No.: 7
Elev./Depth: 19.0'

Load No. = 9
Load = 30.73 tsf
\(D_0 = 0.08393\)
\(D_{90} = 0.09476\)
\(D_{100} = 0.09597\)
\(T_{90} = 0.30\) min.

\(C_v @ T_{90}\)
3.75 ft.2/day

Load No. = 10
Load = 45.21 tsf
\(D_0 = 0.10280\)
\(D_{90} = 0.10639\)
\(D_{100} = 0.10679\)
\(T_{90} = 0.32\) min.

\(C_v @ T_{90}\)
3.33 ft.2/day

EUSTIS ENGINEERING COMPANY, INC.
Dial Reading vs. Time

Source: T15-2
Sample No.: 7
Elev./Depth: 19.0'

Load No. = 11
Load = 15.37 tsf

\[ D_0 = 0.11374 \]
\[ D_{90} = 0.10917 \]
\[ D_{100} = 0.10866 \]
\[ T_{90} = 0.13 \text{ min.} \]

\[ C_v @ T_{90} = 8.43 \text{ ft}^2/\text{day} \]

Load No. = 12
Load = 3.85 tsf

\[ D_0 = 0.10657 \]
\[ D_{90} = 0.10529 \]
\[ D_{100} = 0.10514 \]
\[ T_{90} = 0.56 \text{ min.} \]

\[ C_v @ T_{90} = 1.91 \text{ ft}^2/\text{day} \]

EUSTIS ENGINEERING COMPANY, INC.
Dial Reading vs. Time

Source: T15-2  
Sample No.: 7  
Elev./Depth: 19.0'

Load No. = 13  
Load = 0.97 tsf
  \( D_0 = 0.10456 \)
  \( D_{90} = 0.10199 \)
  \( D_{100} = 0.10170 \)
  \( T_{90} = 0.75 \text{ min.} \)

\[ C_V @ T_{90} \]
1.45 ft.\(^2\)/day

Load No. = 14
Load = 0.25 tsf
  \( D_0 = 0.10165 \)
  \( D_{90} = 0.09901 \)
  \( D_{100} = 0.09872 \)
  \( T_{90} = 2.51 \text{ min.} \)

\[ C_V @ T_{90} \]
0.44 ft.\(^2\)/day

EUSTIS ENGINEERING COMPANY, INC.
# CONSOLIDATION TEST REPORT

## MATERIAL DESCRIPTION

<table>
<thead>
<tr>
<th>USCS</th>
<th>AASHTO</th>
</tr>
</thead>
<tbody>
<tr>
<td>CL</td>
<td></td>
</tr>
</tbody>
</table>

### Source: T15-2

<table>
<thead>
<tr>
<th>Sample No.: 12</th>
<th>Elev./Depth: 44.0'</th>
</tr>
</thead>
</table>

### Remarks:

TESTED BY: RR, LWR  
CHECKED BY: RNE

<table>
<thead>
<tr>
<th>Natural</th>
<th>Dry Dens. (pcf)</th>
<th>LL</th>
<th>PI</th>
<th>Sp. Gr.</th>
<th>P_e (tsf)</th>
<th>C_c</th>
<th>Initial Void Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Saturation</td>
<td>Moisture</td>
<td>99.9%</td>
<td>38.0%</td>
<td>82.7</td>
<td>39</td>
<td>26</td>
<td>2.70</td>
</tr>
</tbody>
</table>

### Graphs:

- **Top Graph:** Void Ratio vs. Applied Pressure (tsf)
- **Bottom Graph:** C_V (ft²/day) vs. Applied Pressure (tsf)
Dial Reading vs. Time

Source: T15-2  Sample No.: 12  Elev./Depth: 44.0'

Load No. = 1
Load = 0.12 tsf
\( D_0 = -0.00024 \)
\( D_{50} = 0.00244 \)
\( D_{100} = 0.00513 \)
\( T_{50} = 2.49 \text{ min.} \)

\[ C_v @ T_{50} \]
0.12 ft.²/day

\[ C_\alpha = 0.001 \]

Load No. = 2
Load = 0.25 tsf
\( D_0 = 0.00440 \)
\( D_{50} = 0.00682 \)
\( D_{100} = 0.00924 \)
\( T_{50} = 1.55 \text{ min.} \)

\[ C_v @ T_{50} \]
0.20 ft.²/day

\[ C_\alpha = 0.002 \]

Figure
Dial Reading vs. Time

Source: T15-2  
Sample No.: 12  
Elev./Depth: 44.0'

Load No. = 3  
Load = 0.49 tsf  
$D_0 = 0.01069$  
$D_{50} = 0.01459$  
$D_{100} = 0.01849$  
$T_{50} = 2.45$ min.

$C_v @ T_{50}$  
0.12 ft./day

$C_\alpha = 0.002$

Load No. = 4  
Load = 0.97 tsf  
$D_0 = 0.01935$  
$D_{50} = 0.02745$  
$D_{100} = 0.03554$  
$T_{50} = 3.21$ min.

$C_v @ T_{50}$  
0.09 ft./day

$C_\alpha = 0.001$
Dial Reading vs. Time

Source: T15-2  Sample No.: 12  Elev./Depth: 44.0′

Load No. = 5
Load = 1.93 tsf
\[ D_0 = 0.03346 \]
\[ D_{50} = 0.04701 \]
\[ D_{100} = 0.06055 \]
\[ T_{50} = 2.75 \text{ min.} \]

\[ C_v @ T_{50} \]
0.10 ft.²/day

\[ C_\alpha = 0.005 \]

Load No. = 6
Load = 3.85 tsf
\[ D_0 = 0.06372 \]
\[ D_{50} = 0.07726 \]
\[ D_{100} = 0.09081 \]
\[ T_{50} = 2.68 \text{ min.} \]

\[ C_v @ T_{50} \]
0.09 ft.²/day

\[ C_\alpha = 0.006 \]
Dial Reading vs. Time

Source: T15-2
Sample No.: 12
Elev./Depth: 44.0'

Load No. = 7
Load = 7.69 tsf
D₀ = 0.09355
D₅₀ = 0.10784
D₁₀₀ = 0.12213
T₅₀ = 1.48 min.

CV @ T₅₀
0.16 ft.²/day
C₀ = 0.005

Load No. = 8
Load = 15.37 tsf
D₀ = 0.12535
D₅₀ = 0.13824
D₁₀₀ = 0.15113
T₅₀ = 0.98 min.

CV @ T₅₀
0.22 ft.²/day
C₀ = 0.005
Dial Reading vs. Time

Source: T15-2  
Sample No.: 12  
Elev./Depth: 44.0'

Load No. = 9  
Load = 3.85 tsf  
\[ D_0 = 0.16124 \]  
\[ D_{50} = 0.15589 \]  
\[ D_{100} = 0.15054 \]  
\[ T_{50} = 1.05 \text{ min.} \]  
\[ C_v @ T_{50} \]  
0.19 ft.²/day

Load No. = 10  
Load = 0.97 tsf  
\[ D_0 = 0.14988 \]  
\[ D_{50} = 0.14581 \]  
\[ D_{100} = 0.14174 \]  
\[ T_{50} = 2.55 \text{ min.} \]  
\[ C_v @ T_{50} \]  
0.08 ft.²/day
Dial Reading vs. Time

Source: T15-2  Sample No.: 12  Elev./Depth: 44.0'

Load No. = 11
Load = 0.25 tsf
D_0 = 0.14405
D_{50} = 0.13632
D_{100} = 0.12858
T_{50} = 13.17 min.

C_V @ T_{50} = 0.02 ft.²/day
CONSOLIDATION TEST REPORT

<table>
<thead>
<tr>
<th>Natural Saturation</th>
<th>Moisture (pcf)</th>
<th>LL</th>
<th>PI</th>
<th>Sp. Gr.</th>
<th>$P_c$ (tsf)</th>
<th>$C_c$</th>
<th>Initial Void Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>95.6 %</td>
<td>23.6 %</td>
<td>101.1</td>
<td>47</td>
<td>29</td>
<td>2.70</td>
<td>5.32</td>
<td>0.19</td>
</tr>
</tbody>
</table>

MATERIAL DESCRIPTION

MST BR SICL W/ CONC, RTS

USCS: CL

AASHTO:

Remarks:
TESTED BY: RR
CHECKED BY: RNE

Source: T15-4
Sample No.: 1
Elev./Depth: 2.0'
Dial Reading vs. Time

Source: T15-4  Sample No.: 1  Elev./Depth: 2.0'  

Load No. = 1
Load = 0.49 tsf
D₀ = 0.00329
D₅₀ = 0.00617
D₁₀₀ = 0.00904
T₅₀ = 0.63 min.

Cᵥ @ T₅₀
0.49 ft.²/day
Cα = 0.001

Load No. = 2
Load = 0.97 tsf
D₀ = 0.00795
D₅₀ = 0.01078
D₁₀₀ = 0.01361
T₅₀ = 4.49 min.

Cᵥ @ T₅₀
0.07 ft.²/day
Cα = 0.001

EUSTIS ENGINEERING COMPANY, INC.
Dial Reading vs. Time

Source: T15-4  Sample No.: 1  Elev./Depth: 2.0'

Load No. = 3
Load = 1.93 tsf

- $D_0 = 0.01351$
- $D_{50} = 0.01732$
- $D_{100} = 0.02113$
- $T_{50} = 4.18$ min.

$C_v@T_{50} = 0.07$ ft.²/day

$C_\alpha = 0.002$

Load No. = 4
Load = 3.85 tsf

- $D_0 = 0.02171$
- $D_{50} = 0.02749$
- $D_{100} = 0.03328$
- $T_{50} = 3.50$ min.

$C_v@T_{50} = 0.08$ ft.²/day

$C_\alpha = 0.002$

EUSTIS ENGINEERING COMPANY, INC.
Dial Reading vs. Time

Source: T15-4  Sample No.: 1  Elev./Depth: 2.0'

Load No. = 5
Load = 7.69 tsf

- $D_0 = 0.03461$
- $D_{50} = 0.04327$
- $D_{100} = 0.05194$
- $T_{50} = 5.36$ min.

$C_v @ T_{50} = 0.05$ ft.$^2$/day

$C_\alpha = 0.002$

Load No. = 6
Load = 15.37 tsf

- $D_0 = 0.05291$
- $D_{50} = 0.06354$
- $D_{100} = 0.07416$
- $T_{50} = 4.63$ min.

$C_v @ T_{50} = 0.06$ ft.$^2$/day

$C_\alpha = 0.002$

Figure
Dial Reading vs. Time

Source: T15-4
Sample No.: 1
Elev./Depth: 2.0'

**Load No. = 7**
Load = 30.73 tsf

- \(D_0 = 0.07720\)
- \(D_{50} = 0.08901\)
- \(D_{100} = 0.10082\)
- \(T_{50} = 4.72 \text{ min.}\)

- \(C_v \ @ \ T_{50} = 0.05 \text{ ft.}^2/\text{day}\)
- \(C_{\alpha} = 0.003\)

**Load No. = 8**
Load = 45.21 tsf

- \(D_0 = 0.10174\)
- \(D_{50} = 0.10877\)
- \(D_{100} = 0.11580\)
- \(T_{50} = 7.48 \text{ min.}\)

- \(C_v \ @ \ T_{50} = 0.03 \text{ ft.}^2/\text{day}\)
- \(C_{\alpha} = 0.003\)
Dial Reading vs. Time

Source: T15-4  Sample No.: 1  Elev./Depth: 2.0'

Load No. = 9
Load = 15.37 tsf

\( D_0 = 0.11723 \)
\( D_{50} = 0.11607 \)
\( D_{100} = 0.11491 \)
\( T_{50} = 0.38 \text{ min.} \)

\( C_v @ T_{50} \)
0.59 ft.2/day

Load No. = 10
Load = 3.85 tsf

\( D_0 = 0.11285 \)
\( D_{50} = 0.10949 \)
\( D_{100} = 0.10613 \)
\( T_{50} = 4.49 \text{ min.} \)

\( C_v @ T_{50} \)
0.05 ft.2/day
Dial Reading vs. Time

Source: T15-4
Sample No.: 1
Elev./Depth: 2.0'

Load No. = 11
Load = 0.97 tsf
D_0 = 0.10684
D_{50} = 0.10125
D_{100} = 0.09565
T_{50} = 24.01 min.

C_v @ T_{50}
0.01 ft.²/day
CONSOLIDATION TEST REPORT

<table>
<thead>
<tr>
<th>Applied Pressure - tsf</th>
<th>Void Ratio</th>
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<tbody>
<tr>
<td></td>
<td>.73</td>
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<tr>
<td></td>
<td>.68</td>
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<td>.62</td>
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<td>.23</td>
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<td></td>
<td>.20</td>
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<table>
<thead>
<tr>
<th>CV (ft²/day)</th>
<th>.00</th>
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<tr>
<td>0.1</td>
<td>.08</td>
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<tr>
<td>0.2</td>
<td>.16</td>
</tr>
<tr>
<td>0.3</td>
<td>.24</td>
</tr>
<tr>
<td>0.4</td>
<td>.32</td>
</tr>
<tr>
<td>0.5</td>
<td>.40</td>
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<tr>
<th>Natural</th>
<th>Dry Dens. (pcf)</th>
<th>LL</th>
<th>PI</th>
<th>Sp. Gr.</th>
<th>Pₚ (tsf)</th>
<th>Cc</th>
<th>Initial Void Ratio</th>
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</thead>
<tbody>
<tr>
<td>100.0 %</td>
<td>26.9 %</td>
<td>96.9</td>
<td>38</td>
<td>24</td>
<td>2.69</td>
<td>5.81</td>
<td>0.26</td>
</tr>
</tbody>
</table>

MATERIAL DESCRIPTION

ST LT-G SAACL

USCS

AASHTO

CL

Source: T15-4

Sample No.: 18

Elev./Depth: 78.0'

Remarks:
TESTED BY: RR
CHECKED BY: RNE

Figure
Dial Reading vs. Time

Source: T15-4
Sample No.: 18
Elev./Depth: 78.0'

Load No. = 1
Load = 0.49 tsf

\[ D_0 = -0.00156 \]
\[ D_{50} = 0.00228 \]
\[ D_{100} = 0.00612 \]
\[ T_{50} = 2.12 \text{ min.} \]

\[ C_v @ T_{50} \]
0.15 ft.2/day

\[ C_\alpha = 0.001 \]

---

Load No. = 2
Load = 0.97 tsf

\[ D_0 = 0.00523 \]
\[ D_{50} = 0.00985 \]
\[ D_{100} = 0.01448 \]
\[ T_{50} = 1.81 \text{ min.} \]

\[ C_v @ T_{50} \]
0.17 ft.2/day

\[ C_\alpha = 0.001 \]
Dial Reading vs. Time

Source: T15-4
Sample No.: 18
Elev./Depth: 78.0'

Load No. = 3
Load = 1.93 tsf

- $D_0 = 0.01473$
- $D_{50} = 0.02019$
- $D_{100} = 0.02565$
- $T_{50} = 1.69$ min.

$C_v @ T_{50}$
0.17 ft.$^2$/day

$C_\alpha = 0.002$

---

Load No. = 4
Load = 3.85 tsf

- $D_0 = 0.02713$
- $D_{50} = 0.03473$
- $D_{100} = 0.04232$
- $T_{50} = 1.32$ min.

$C_v @ T_{50}$
0.22 ft.$^2$/day

$C_\alpha = 0.002$
Dial Reading vs. Time

Source: T15-4
Sample No.: 18
Elev./Depth: 78.0'

Load No. = 5
Load = 7.69 tsf
D_0 = 0.04439
D_50 = 0.05508
D_100 = 0.06578
T_50 = 1.87 min.

C_v @ T_50
0.14 ft.^2/day

C_0 = 0.004

Load No. = 6
Load = 15.37 tsf
D_0 = 0.06933
D_50 = 0.08366
D_100 = 0.09799
T_50 = 3.23 min.

C_v @ T_50
0.08 ft.^2/day

C_0 = 0.005
Dial Reading vs. Time

Source: T15-4
Sample No.: 18
Elev./Depth: 78.0'

Load No. = 7
Load = 30.73 tsf
D_0 = 0.10320
D_{50} = 0.11861
D_{100} = 0.13401
T_{50} = 5.60 min.

C_v @ T_{50}
0.04 ft.²/day

C_α = 0.007

Load No. = 8
Load = 45.21 tsf
D_0 = 0.13828
D_{50} = 0.14737
D_{100} = 0.15645
T_{50} = 11.69 min.

C_v @ T_{50}
0.02 ft.²/day

C_α = 0.006
Dial Reading vs. Time

Source: T15-4  Sample No.: 18  Elev./Depth: 78.0’

---

**Load No.: 9**

Load = 15.37 tsf

\[ D_0 = 0.15708 \]

\[ D_{50} = 0.15469 \]

\[ D_{100} = 0.15229 \]

\[ T_{50} = 1.68 \text{ min.} \]

**C_v @ T_{50}**

0.12 ft.²/day

---

**Load No.: 10**

Load = 3.85 tsf

\[ D_0 = 0.14940 \]

\[ D_{50} = 0.14234 \]

\[ D_{100} = 0.13528 \]

\[ T_{50} = 14.26 \text{ min.} \]

**C_v @ T_{50}**

0.01 ft.²/day
Dial Reading vs. Time

Source: T15-4  Sample No.: 18  Elev./Depth: 78.0'

Load No. = 11
Load = 0.97 tsf
D₀ = 0.13651
D₅₀ = 0.12681
D₁₀₀ = 0.11711
T₅₀ = 29.76 min.

Cᵥ @ T₅₀
0.01 ft.²/day
## CONSOLIDATION TEST REPORT

<table>
<thead>
<tr>
<th>Natural</th>
<th>Dry Dens. (pcf)</th>
<th>LL</th>
<th>PI</th>
<th>Sp. Gr.</th>
<th>$P_c$ (tsf)</th>
<th>$C_c$</th>
<th>Initial Void Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Saturation</td>
<td>Moisture</td>
<td></td>
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<td></td>
<td></td>
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<td></td>
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<tr>
<td>99.6 %</td>
<td>34.3 %</td>
<td>87.3</td>
<td>45</td>
<td>27</td>
<td>2.70</td>
<td>1.02</td>
<td>0.25</td>
</tr>
</tbody>
</table>

### MATERIAL DESCRIPTION

**USCS**

SO G SACL W/ SH FRAG

**AASHTO**

CL

### Project

- LOCATION: UNIDENTIFIED, PERMITTED TO BE IDENTIFIED LATER

### Remarks:

- TESTED BY: RR
- CHECKED BY: RNE

### Source:

- T16-1
- Sample No.: 8
- Elev./Depth: 28.0'

---

**Figure**
Dial Reading vs. Time

Source: T16-1
Sample No.: 8
Elev./Depth: 28.0'

Load No. = 1
Load = 0.13 tsf

\[ D_0 = -0.00030 \]
\[ D_{50} = 0.00123 \]
\[ D_{100} = 0.00277 \]
\[ T_{50} = 3.20 \text{ min.} \]

\[ C_V @ T_{50} = 0.15 \text{ ft.}^2/\text{day} \]
\[ C_\alpha = 0.000 \]

Load No. = 2
Load = 0.25 tsf

\[ D_0 = 0.00253 \]
\[ D_{50} = 0.00509 \]
\[ D_{100} = 0.00764 \]
\[ T_{50} = 5.74 \text{ min.} \]

\[ C_V @ T_{50} = 0.08 \text{ ft.}^2/\text{day} \]
\[ C_\alpha = 0.002 \]
Dial Reading vs. Time

Source: T16-1  Sample No.: 8  Elev./Depth: 28.0'

Load No. = 3
Load = 0.50 tsf
D₀ = 0.00874
D₅₀ = 0.01599
D₁₀₀ = 0.02325
T₅₀ = 7.15 min.

Cᵥ @ T₅₀
0.07 ft.²/day
Cₓ = 0.002

Load No. = 4
Load = 1.00 tsf
D₀ = 0.02514
D₅₀ = 0.03470
D₁₀₀ = 0.04425
T₅₀ = 5.46 min.

Cᵥ @ T₅₀
0.08 ft.²/day
Cₓ = 0.004
**Dial Reading vs. Time**

**Source:** T16-1

**Sample No.:** 8

**Elev./Depth:** 28.0'

**Load No. =** 5

**Load =** 2.00 tsf

\[ D_0 = 0.04545 \]

\[ D_{50} = 0.05959 \]

\[ D_{100} = 0.07373 \]

\[ T_{50} = 4.59 \text{ min.} \]

\[ C_v @ T_{50} = 0.09 \text{ ft}^2/\text{day} \]

\[ C_\alpha = 0.004 \]

---

**Load No. =** 6

**Load =** 4.00 tsf

\[ D_0 = 0.07704 \]

\[ D_{50} = 0.09078 \]

\[ D_{100} = 0.10451 \]

\[ T_{50} = 4.22 \text{ min.} \]

\[ C_v @ T_{50} = 0.10 \text{ ft}^2/\text{day} \]

\[ C_\alpha = 0.005 \]
Dial Reading vs. Time

Source: T16-1
Sample No.: 8
Elev./Depth: 28.0'

Load No. = 7
Load = 8.00 tsf
D₀ = 0.10600
D₅₀ = 0.12427
D₁₀₀ = 0.14255
T₅₀ = 2.79 min.

Cᵥ @ T₅₀
0.13 ft.²/day

Cα = 0.004

Load No. = 8
Load = 16.00 tsf
D₀ = 0.14546
D₅₀ = 0.16122
D₁₀₀ = 0.17698
T₅₀ = 2.33 min.

Cᵥ @ T₅₀
0.15 ft.²/day

Cα = 0.005

EUSTIS ENGINEERING COMPANY, INC.
Dial Reading vs. Time

Source: T16-1  
Sample No.: 8  
Elev./Depth: 28.0'

Load No. = 9
Load = 4.00 tsf
\[ D_0 = 0.18298 \]
\[ D_{50} = 0.18111 \]
\[ D_{100} = 0.17924 \]
\[ T_{50} = 0.36 \text{ min.} \]

\[ C_V @ T_{50} \]
0.92 ft.\(^2\)/day

Load No. = 10
Load = 1.00 tsf
\[ D_0 = 0.17998 \]
\[ D_{50} = 0.17630 \]
\[ D_{100} = 0.17263 \]
\[ T_{50} = 4.54 \text{ min.} \]

\[ C_V @ T_{50} \]
0.07 ft.\(^2\)/day

EUSTIS ENGINEERING COMPANY, INC.
Dial Reading vs. Time

Source: T16-1  
Sample No.: 8  
Elev./Depth: 28.0'

Load No. = 11
Load = 0.25 tsf

\[
D_0 = 0.17375 \\
D_{50} = 0.16578 \\
D_{100} = 0.15781 \\
T_{50} = 23.65 \text{ min.}
\]

\[
C_v @ T_{50} = 0.01 \text{ ft.}^2/\text{day}
\]
CONSOLIDATION TEST REPORT

MATERIAL DESCRIPTION

USCS | AASHTO
---|---
MST G CL W/ SISA POC, WD | CH

<table>
<thead>
<tr>
<th>Natural</th>
<th>Dry Dens. (pcf)</th>
<th>LL</th>
<th>PI</th>
<th>Sp. Gr.</th>
<th>( P_c ) (tsf)</th>
<th>( C_c )</th>
<th>Initial Void Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Saturation</td>
<td>Moisture</td>
<td>76.2</td>
<td>64</td>
<td>46</td>
<td>2.70</td>
<td>1.60</td>
<td>0.49</td>
</tr>
</tbody>
</table>

Remarks:
TESTED BY: RR
CHECKED BY: RNE

Source: T16-1
Sample No.: 14
Elev./Depth: 58.0'

EUSTIS
Metairie, Louisiana
Lafayette, Louisiana
Gulfport, Mississippi
Dial Reading vs. Time

Source: T16-1
Sample No.: 14
Elev./Depth: 58.0'

Load No. = 2
Load = 0.50 tsf
D₀ = 0.00121
D₅₀ = 0.00416
D₁₀₀ = 0.00711
T₅₀ = 2.96 min.

CV @ T₅₀
0.16 ft²/day

Cₐ = 0.001

Load No. = 3
Load = 1.00 tsf
D₀ = 0.00697
D₅₀ = 0.01565
D₁₀₀ = 0.02433
T₅₀ = 6.51 min.

CV @ T₅₀
0.07 ft²/day

Cₐ = 0.002

Figure
Dial Reading vs. Time

Source: T16-1  Sample No.: 14  Elev./Depth: 58.0'

Load No. = 4
Load = 2.00 tsf

\[ D_0 = 0.02351 \]
\[ D_{50} = 0.03898 \]
\[ D_{100} = 0.05445 \]
\[ T_{50} = 7.45 \text{ min.} \]

\[ C_v @ T_{50} \]

0.06 ft.²/day

\[ C_\alpha = 0.007 \]

Load No. = 5
Load = 4.00 tsf

\[ D_0 = 0.05678 \]
\[ D_{50} = 0.08740 \]
\[ D_{100} = 0.11803 \]
\[ T_{50} = 14.96 \text{ min.} \]

\[ C_v @ T_{50} \]

0.03 ft.²/day

\[ C_\alpha = 0.008 \]
Dial Reading vs. Time

Source: T16-1  
Sample No.: 14  
Elev./Depth: 58.0'

Load No. = 6
Load = 8.00 tsf
D₀ = 0.12022
D₅₀ = 0.14863
D₁₀₀ = 0.17703
T₅₀ = 16.69 min.

Cᵥ @ T₅₀
0.02 ft²/day
Cᵥ = 0.014

Load No. = 7
Load = 2.00 tsf
D₀ = 0.18376
D₅₀ = 0.17431
D₁₀₀ = 0.16486
T₅₀ = 9.96 min.

Cᵥ @ T₅₀
0.03 ft²/day
Dial Reading vs. Time

Source: T16-1
Sample No.: 14
Elev./Depth: 58.0'

Load No. = 8
Load = 0.50 tsf

\[
D_0 = 0.16826 \\
D_{50} = 0.15191 \\
D_{100} = 0.13556 \\
T_{50} = 33.72 \text{ min.}
\]

\[C_v @ T_{50}\]

0.01 ft.²/day
Sandy silt to clayey silt (6)
Clayey silt to silty clay (5)
Clayey silt to silty clay (5)
Clayey silt to silty clay (5)
Silty clay to clay (4)
Clayey silt to silty clay (5)
Silty sand to sandy silt (7)
Sand to silty sand (8)
Silty sand to sandy silt (7)
Clayey silt to silty clay (5)
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A-110
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Silty clay to clay (4)
Cone No: DDG1068
Tip area [cm²]: 10
Sleeve area [cm²]: 150

Test no:
CPT-22

Client:

Project:

Depth [ft]

qc [Tsf]
fs [psf]
u2 [psi]
Su [psf]
N60 []
Test no: CPT-21
Client: NUSTAR ENERGY L.P.
Project: PROPOSED EXPANSION

Cone No: DDG1068
Tip area [cm²]: 10
Sleeve area [cm²]: 150

Position: S. JAMES PARISH, LOUISIANA

Ground level: D

Date: 2/5/2010

Scale: 1 : 120

Page: 2/2

Fig: c21.cpd
Test no: CPT-19
Client: NUSTAR ENERGY L.P.
Project: PROPOSED EXPANSION

U2

Cone No: DDG1068
Tip area [cm²]: 10
Sleeve area [cm²]: 150
Cone No: DDG1068
Tip area [cm²]: 10
Sleeve area [cm²]: 150

Test no: CPT-18
Client: NUSTAR ENERGY L.P.
Project: PROPOSED EXPANSION
Position: N30° 01.574', W90° 51.269'

Scale: 1 : 120
Date: 2/3/2010
Page: 1/2
Fig: File: c18.cpd
Sandy silt to clayey silt (6)
Clayey silt to silty clay (5)
Clayey silt to silty clay (5)
Clay (3)
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Test no: CPT-12
Client: NUSTAR ENERGY L.P.
Project: PROPOSED EXPANSION
Position: N30° 01.546', W90° 51.411'
Ground level:
Date: 2/1/2010
Scale: 1 : 120
Page: 2/2
Fig: c12.cpd

Sleeve area [cm^2]: 150
Tip area [cm^2]: 10
Cone No: DDG1068

Clayey silt to silty clay (5)
Clayey silt to silty clay (5)
Clayey silt to silty clay (5)
Clayey silt to silty clay (5)
Clayey silt to silty clay (5)
Clay (3)
Sandy silt to clayey silt (6)
Silty clay to clay (4)
Clay (3)
Silty clay to clay (4)
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Clay (3)
Silty clay to clay (4)
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Silty clay to clay (4)
Clay (3)
Silty clay to clay (4)
Silty clay to clay (4)
Silty clay to clay (4)
Test no: CPT-10
Client: NUSTAR ENERGY L.P.
Project: PROPOSED EXPANSION

Position: N30° 01.580', W90° 51.431'
Ground level: D
Date: 2/1/2010
Scale: 1 : 120
Page: 2/2
Fig: File: c10.cpd

Clayey silt to silty clay (5)
Silty clay to clay (4)
Silty clay to clay (4)
Clayey silt to silty clay (5)
Clayey silt to silty clay (5)
Clayey silt to silty clay (5)

Cone No: DDG1068
Tip area [cm²]: 10
Sleeve area [cm²]: 150
Cone No: DDG1068
Tip area (cm²): 10
Sleeve area (cm²): 150

Test no: CPT-9
Client: NUSTAR ENERGY L.P.
Project: PROPOSED EXPANSION
Position: N30° 01.570', W90° 51.403'
Ground level:
Date: 1/29/2010
Scale: 1 : 120
Page: 2/2
Fig: F
File: c9.cpd

Clayey silt to silty clay (5)
Clayey silt to silty clay (5)
Silty clay to clay (4)
Sandy silt to clayey silt (6)
Clayey silt to silty clay (5)
Clayey silt to silty clay (5)
Clayey silt to silty clay (5)
Clayey silt to silty clay (5)
Clayey silt to silty clay (5)
Clayey silt to silty clay (5)

Clayey silt to silty clay (5)
Cone No: DDG1068
Tip area [cm²]: 10
Sleeve area [cm²]: 150

Test no: CPT-8
Client: NUSTAR ENERGY L.P.
Project: PROPOSED EXPANSION

Ground level: S
Date: 1/28/2010
Scale: 1 : 120
Page: 2/2

Fig: F
File: c8.cpd

Depth [ft]

Clayey silt to silty clay (5)
Clayey silt to silty clay (5)
Clayey silt to silty clay (5)
Silty clay to clay (4)
Clayey silt to silty clay (5)
Sand to silty sand (8)
Sandy silt to clayey silt (6)
Clayey silt to silty clay (5)
Sandy silt to clayey silt (6)
Sand to silty sand (8)
Clayey silt to silty clay (5)
Sand to silty sand (8)
Sand to silty sand (8)
Clayey silt to silty clay (5)
Sandy silt to clayey silt (6)

Test no: CPT-3
Client: NUSTAR ENERGY L.P.
Project: PROPOSED EXPANSION
Position: N30° 01.615', W90° 51.366'
Ground level: D
Date: 1/27/2010
Scale: 1 : 120

Cone No: DDG1068
Tip area [cm²]: 10
Sleeve area [cm²]: 150

A-152
Silty sand to sandy silt (7)
Clayey silt to silty clay (5)
Sand to silty sand (8)
Silty sand to sandy silt (7)
Sand to silty sand (8)
Clayey silt to silty clay (5)
Clayey silt to silty clay (5)
Sandy silt to clayey silt (6)
Clayey silt to silty clay (5)
Sandy silt to clayey silt (6)
Clayey silt to silty clay (5)
Clayey silt to silty clay (5)
Sandy silt to clayey silt (6)
Clayey silt to silty clay (5)
Clayey silt to silty clay (5)
Sandy silt to clayey silt (6)
Clayey silt to silty clay (5)
▲ DENOTES LOCATION OF CONE PENETROMETER TESTS PERFORMED: 5 TO 15 MARCH 2010
● DENOTES LOCATION OF UNDISTURBED SOIL BORINGS DRILLED: 24 FEBRUARY TO 3 MARCH 2010
○ DENOTES LOCATION OF ORIGINAL PROPOSED TANK; LOCATION NO LONGER PLANNED
○ DENOTES REVISED LOCATION OF PROPOSED TANK

B-158
**CPT Soil Borings & Test Results**

- **CPT Soil Borings**
  - **Soil Type**: Medium Stiff to Stiff Gray & Tan Clay
  - **Soil Color**: Soft Gray & Tan Silty Clay, Medium Stiff Clay, Soft Clay
  - **Soil Moisture**: Medium Stiff Gray Silty Clay, Medium Stiff Gray Clay, Medium Stiff Clay
  - **Soil Type**: Medium Stiff Gray Sandy Clay, Medium Stiff Gray Clay, Medium Stiff Clay

- **Test Results**
  - **Depth (ft)**: 0 - 150
  - **Soil Pressure (psf)**: 0 - 150

**NOTE:**

The number to the left of the boring log represents standard penetration test (SPT) results.

B-164
NOTE: DRAWING INDICATES SCHEMATIC OF PROPOSED RECOMMENDATIONS. THE LIMITS OF THE EARTH PRELOAD MAY PROPORTIONED ACCORDING TO ALTERNATE TANK DIAMETERS.
NOTE: DRAWING INDICATES SCHEMATIC OF PROPOSED RECOMMENDATIONS. THE LIMITS OF THE EARTH PRELOAD MAY PROPORTIONED ACCORDING TO ALTERNATE TANK DIAMETERS.
(Estimated minimum 12' crown, additional sand fill may be added to create crown in tank floor after tank hydrotest)

Concrete ringwall

Clay fill

Sand fill

Claymax liner

Existing ground surface

(Ringwall to extend minimum 1' below sand fill)

Horizontal wick drains

Approx. edge of tank

MINIMUM WICK PENETRATION BELOW EXISTING GROUND SURFACE

Approx. center of tank

Vertical wick drains

Continuous horizontal strip drain

Clay max liner

Sand fill

Weep hole

Horizontal wick drains

Not to scale

Schematic only
1 5/16" OUTSIDE DIAMETER FLUSH COUPLED ROD INSERTED INTO 1 1/2" PIPE AND WELDED

PAPER AND GREASE WRAPPING AROUND ROD

4" LENGTH OF 1 1/2" PIPE WELDED TO BASE

2' X 2' 3/8" STEEL PLATE

WELD

EXISTING GROUND SURFACE

4" SAND BLANKET

TYPICAL SETTLEMENT PLATE
## LOG OF BORING AND TEST RESULTS

### (Sheet 1 of 2)

<table>
<thead>
<tr>
<th>Scale In Feet</th>
<th>PP</th>
<th>SPT</th>
<th>Symbol</th>
<th>Visual Classification</th>
<th>USC</th>
<th>Sample Number</th>
<th>Depth In Feet</th>
<th>Water Content Percent</th>
<th>Density</th>
<th>Shear Tests</th>
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**Comments:**

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**EUSTIS ENGINEERING**

**LOG OF BORING AND TEST RESULTS**

(Sheet 2 of 2)

**Ground Elev.: Datum:**

**Gr. Water Depth:** See Text

**: 2/24/10**

**Boring:** 1

Refer to "Legends & Notes"

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**B-172**
# LOG OF BORING AND TEST RESULTS

**Boring:** 2  
**Date:** 2/24-25/10  
**Refer to "Legends & Notes"**

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**Comments:**

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Comments:

B-174
## LOG OF BORING AND TEST RESULTS

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**Datum:**
- See Text

**Gr. Water Depth:**
- See Text

**Bored: 2/26 & 3/01/10**

**Boring:** 3

**Refer to "Legends & Notes"**

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**Comments:**

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Comments:
## LOG OF BORING AND TEST RESULTS

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**Comments:**

**B-177**
# LOG OF BORING AND TEST RESULTS

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Comments:
### Log of Boring and Test Results

**EUSTIS ENGINEERING**

**LOG OF BORING AND TEST RESULTS**

(Sheet 2 of 3)

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### Log of Boring and Test Results

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<th>Water Content Percent</th>
<th>Density (Type)</th>
<th>Shear Tests</th>
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<th>Other Tests</th>
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Visual Classification:
- Medium stiff gray clay w/trace of silt pockets
- w/silty sand pockets
- Soft light gray silty clay
- Very stiff light gray clay w/silty sand pockets
- Stiff tan silty clay

Sample Number:
- 14
- 15
- 16
- 17
- 18
- 19
- 20
- 21
- 22
- 23

Sample Depth (Feet): 52.5-55, 57.5-60, 62.5-65, 67.5-70, 72.5-75, 77.5-80, 82.5-85, 87.5-90, 92.5-95, 97.5-100

Water Content Percent:
- 38
- 22
- 19
- 19
- 28
- 28

Density (Type):
- UC
- UC
- UC
- UC

Shear Tests:
- \( o \)
- \( C \)
- \( LL \)
- \( PL \)
- \( PI \)

Atterberg Limits:
- 816
- 348
- 3392
- 1439

Comments:

(B-183)
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Comments:
# LOG OF BORING AND TEST RESULTS

**Ground Elev.:**

**Datum:**

**Gr. Water Depth:** See Table

**d:** 3/02-03/10

**Boring:** 7

Refer to "Legends & Notes"

## Scale in Feet

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**Comments:**
## Log of Boring and Test Results

**Datum:**

**Gr. Water Depth:** See Table

**Sample Number** | **Depth in Feet** | **Water Content Percent** | **Density** | **Shear Tests** | **Atterberg Limits** |
--- | --- | --- | --- | --- | --- |
CH | 14 | 52.5-55 | 34 | 88 | 118 | UC | 325 |
CL | 15 | 57.5-60 | | | | |
CH | 16 | 62.5-65 | 69 | 58 | 98 | UC | 354 | 107 | 24 | 83 |
| 17 | 67.5-70 | | | | |
CH | 18 | 72.5-75 | 20 | 109 | 132 | UC | 1981 |
SM | 19 | 77.5-80 | | | | |
CH | 20 | 82.5-85 | 23 | | | |
SM | 21 | 87.5-90 | | | | |
CH | 22 | 92.5-95 | 22 | 106 | 129 | UC | 1155 |
| 23 | 97.5-100 | | | | |

**Comments:**

**Boring:** 7

**Eustis Engineering**

(Sheet 2 of 3)
# LOG OF BORING AND TEST RESULTS

**Ground Elev.:**

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**Comments:**

- Refer to "Legends & Notes"
Sleeve area [cm²]: 150
Tip area [cm²]: 10

Cone No: DTG0704
Test no: 10007-1
Client: NUSTAR ENERGY L.P.
Project: PROPOSED EXPANSION
Position: N30° 01.303', W90° 51.348'
Ground level: D
Date: 3/11/2010
Scale: 1 : 120
Page: 1/2
Fig: F
File: 10007-1.cpd

Clay (3)
Organic material (2)
Clay (3)
Clay (3)
Silty clay to clay (4)
Clay (3)
Clay (3)
Clay (3)
Clay (3)
Clayey silt to silty clay (5)
Sandy silt to clayey silt (6)
Sand to silty sand (8)
Sand to silty sand (8)
Clayey silt to silty clay (5)
Sandy silt to clayey silt (6)
Clay (3)
Clay (3)
Silty clay to clay (4)
Clay (3)
Silty clay to clay (4)
Silty clay to clay (4)
Clay (3)
Clay (3)
Silty clay to clay (4)
Clay (3)
Silty clay to clay (4)
Silty clay to clay (4)
Sandy silt to clayey silt (6)
Clayey silt to silty clay (5)
Silty clay to clay (4)
Clayey silt to silty clay (5)
Clayey silt to silty clay (5)
Clay (3)
Silty clay to clay (4)
Silty clay to clay (4)

B-188
Cone No: DTG6704
Tip area [cm²]: 10
Sleeve area [cm²]: 150
Test no: 10007-2
Client: NUSTAR ENERGY L.P.
Project: PROPOSED EXPANSION
Position: N30° 01.323', W90° 51.348'
Ground level: D
Date: 3/15/2010
Scale: 1 : 120
Page: 1/2
Fig: F
File: 10007-2.cpd
Sleeve area [cm²]: 150
Tip area [cm²]: 10
Cone No: DTG0704
Test no: 10007-2
Client: NUSTAR ENERGY L.P.
Project: PROPOSED EXPANSION
Position: N30° 01.323', W90° 51.348'
Ground level:
Scale: 1 : 120
Date: 3/15/2010
Page: 2/2
Fig: F
File: 10007-2.cpd
Sleeve area [cm²]: 150
Tip area [cm²]: 10

Cone No: DTG0704
Test no: 10007-3
Client: NUSTAR ENERGY L.P.
Project: PROPOSED EXPANSION
Position: N30° 01.311', W90° 51.330'
Ground level: D
Date: 3/11/2010
Scale: 1 : 120
Page: 1/2
Fig: F
File: 10007-3.cpd
Silty clay to clay (4)
Clayey silt to silty clay (5)
Silty clay to clay (4)
Clayey silt to silty clay (5)
Sandy silt to clayey silt (6)
Clayey silt to silty clay (5)
Sandy silt to clayey silt (6)
Clayey silt to silty clay (5)
Sandy silt to clayey silt (6)
Clayey silt to silty clay (5)
Sandy silt to clayey silt (6)

Test no: 10009-2
Client: NUSTAR ENERGY L.P.
Project: PROPOSED EXPANSION
Position: N30° 01.305', W90° 51.385'
Ground level: D
Date: 3/9/2010
Scale: 1 : 120
Page: 2/2
Fig: 10009-2.cpd

Cone No: DTG6704
Tip area [cm2]: 10
Sleeve area [cm2]: 150

B-197
Test no: 10011-1
Client: NUSTAR ENERGY L.P.
Project: PROPOSED EXPANSION
Position: N30° 01.260', W90° 51.426'
Ground level: D
Date: 3/11/2010
Scale: 1 : 120
Page: 1/2
Fig: F
File: 10011-1.cpd

Cone No: DTG0704
Tip area [cm²]: 10
Sleeve area [cm²]: 150
Test no: 10811-2
Client:
Project:

Cone No: DTG6704
Tip area [cm²]: 10
Sleeve area [cm²]: 150

Ground level: D
Date: 3/9/2010
Scale: 1 : 120
Page: 2/2

File: 10811-2.cpd

Sand to silty sand (8)
Silty sand to sandy silt (7)
Silty sand to sandy silt (7)
Clay (3)
Sand to silty sand (8)
Sandy silt to sandy silt (7)
Silty sand to sandy silt (7)

B-203
Clay (3)
Clayey silt to silty clay (5)
Sandy silt to clayey silt (6)
Clay (3)
Silty sand to sandy silt (7)
Clayey silt to silty clay (5)
Clay (3)
Clay (3)
Clay (3)
Clay (3)
Clayey silt to silty clay (5)
Sandy silt to clayey silt (6)
Sand to silty sand (8)
Clayey silt to silty clay (5)
Sand to silty sand (8)
Silty clay to clay (4)
Sand to silty sand (8)
Silty clay to clay (4)
Sand to silty sand (8)
Silty clay to clay (4)
Clay (3)
Silty clay to clay (4)
Clayey silt to silty clay (5)
Sensitive fine grained (1)
Silty clay to clay (4)
Clayey silt to silty clay (5)
Silty clay to clay (4)
Clayey silt to silty clay (5)
Silty sand to sandy silt (7)

Test no: 10011-3
Client: NUSTAR ENERGY L.P.
Project: PROPOSED EXPANSION
Position: N30° 01.277', W90° 51.407'
Date: 3/10/2010
Scale: 1 : 120
Test no: 10011-3
Client:
Project:

Sandy silt to clayey silt (6)
Sand to silty sand (8)
Silty sand to sandy silt (7)
Clayey silt to silty clay (5)
Clay (3)
Sandy silt to clayey silt (6)
Clayey silt to silty clay (5)
Sand to silty sand (8)
Sand to silty sand (8)
Sand to silty sand (8)
Sand to silty sand (8)

Cone No: DTG0704
Tip area [cm²]: 10
Sleeve area [cm²]: 150

Date: 3/10/2010
Scale: 1:120
Depth [ft]
Cone No: DTG0704
Tip area [cm²]: 10
Sleeve area [cm²]: 150
Cone No: DTG6704
Tip area [cm²]: 10
Sleeve area [cm²]: 150

Client: NUSTAR ENERGY L.P.
Project: PROPOSED EXPANSION

Position: N30° 01.321', W90° 51.298'
Date: 3/12/2010
Scale: 1 : 120
Page: 1/2

Test no: 15002-4
File: 15002-4.cpd

Clay (3)
Clay (3)
Clay (3)
Organic material (2)
Clay (3)
Sandy silt to clayey silt (6)
Clayey silt to silty clay (5)
Clayey silt to silty clay (5)
Clay (3)
Clay (3)
Clay (3)
Sandy silt to clayey silt (6)
Clayey silt to silty clay (5)
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Clayey silt to silty clay (5)
Silty clay to clay (4)
Clay (3)
Clay (3)
Silty clay to clay (4)
Silty clay to clay (4)
Clayey silt to silty clay (5)
Clay (3)
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Clayey silt to silty clay (5)
Clayey silt to silty clay (5)
Sandy silt to clayey silt (6)
Sandy silt to clayey silt (6)
Clayey silt to silty clay (5)
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Sandi...
Sleeve area [cm²]: 150
Tip area [cm²]: 10
Cone No: DTG0704
Test no: 15003-1
Client: NUSTAR ENERGY L.P.
Project: PROPOSED EXPANSION S T. JAMES PARISH, LOUISIANA
Position: N30° 01.343', W90° 51.280'
Ground level: D
Date: 3/12/2010
Scale: 1 : 120
Page: 2/2
Fig: File: 15003-1.cpd

Clay (3)
Clay (3)
Silty clay to clay (4)
Sandy silt to clayey silt (6)
Clayey silt to silty clay (5)
Clayey silt to silty clay (5)
Sand (9)

Clay (3)
Clay (3)
Silty clay to clay (4)
Sandy silt to clayey silt (6)
Clayey silt to silty clay (5)
Clayey silt to silty clay (5)
Sand (9)
Sleeve area [cm²]: 150
Tip area [cm²]: 10

Cone No: DTG6704
Client: NUSTAR ENERGY L.P.
Project: PROPOSED EXPANSION
Position: N30° 01.346', W90° 51.268'
Date: 3/15/2010
Scale: 1 : 120
Page: 1/2
Test no: 15002-2
File: 15003-2.cpd

Organic material (2)
Clay (3)
Silty clay to clay (4)
Clay (3)
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B-216
Clay (3)

Clayey silt to silty clay (5)

Clayey silt to silty clay (5)

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Sandy silt to clayey silt (6)

Silty sand to sandy silt (7)

Silty clay to clay (4)

Silty sand to sandy silt (7)

Sand to silty sand (8)

Silty clay to clay (4)

Sandy silt to clayey silt (6)

Clayey silt to silty clay (5)

Sandy silt to clayey silt (6)

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Clayey silt to silty clay (5)

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Sleeve area [cm²]: 150
Tip area [cm²]: 10

Cone No: DTG0704
Test no: 15003-4
Client: NUSTAR ENERGY L.P.
Project: PROPOSED EXPANSION

Position: N30° 01.321', 90° 51.298'
Ground level: D
Date: 3/11/2010
Scale: 1 : 120
Page: 2/2
Fig: F
File: 15003-4.cpd

Clayey silt to silty clay (5)
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Sandy silt to clayey silt (6)
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Sandy silt to clayey silt (6)
Silty sand to sandy silt (7)
Clayey silt to silty clay (5)
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Test no: 37008-4

Client:

Project: PROPOSED PLANT EXPANSION, ASPHALT PROCESS UNIT

Position:

Date: 4/3/2010

Scale: 1:120

Page: 2/2

Fig:

File: 37008-4.cpd

Sleeve area [cm²]: 150

Tip area [cm²]: 10

Cone No: DTG0704

Clayey silt to silty clay (5)
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Sandy silt to clayey silt (6)
Silty clay to clay (4)
Silty sand to sandy silt (7)
Sandy silt to clayey silt (6)
Clayey silt to silty clay (5)
Clayey silt to silty clay (5)
Sandy silt to clayey silt (6)
Sand to silty sand (8)

Test no: 37009-2
Client: NUSTAR ENERGY L.P.
Project: PROPOSED PLANT EXPANSION, ASPHALT PROCESS UNIT
Position: 
Ground level: D
Date: 23/8/2010
Scale: 1 : 120
Page: 2/2
Fig: 
File: 37009-2.cpd

Cone No: DTG0704
Tip area [cm2]: 10
Sleeve area [cm2]: 150

Sandy silt to clayey silt (6)
Silty clay to clay (4)
Silty sand to sandy silt (7)
Sandy silt to clayey silt (6)
Clayey silt to silty clay (5)
Clayey silt to silty clay (5)
Sandy silt to clayey silt (6)
Sand to silty sand (8)
Sleeve area [cm²]: 150
Tip area [cm²]: 10
Cone No: DTG0704
Test no: 37009-3
Client: NUSTAR ENERGY L.P.
Project: PROPOSED PLANT EXPANSION, ASPHALT PROCESS UNIT

Test no: 37009-3
Client: NUSTAR ENERGY L.P.
Project: PROPOSED PLANT EXPANSION, ASPHALT PROCESS UNIT
Cone No: DTG0704
Tip area [cm²]: 10
Sleeve area [cm²]: 150
Cone No: DTG6704

Tip area [cm²]: 10
Sleeve area [cm²]: 150

Test no: 37009-4
Client: NUSTAR ENERGY L.P.
Project: PROPOSED PLANT EXPANSION, ASPHALT PROCESS UNIT
Position: T. J. PARISH, LOUISIANA
Date: 4/8/2010
Scale: 1:120
Page: 1/2

Fig: F
File: 37009-4.cpd

Depth [ft]

QC [Tsf] ——
FS [psf] ——
U2 [psi] ——
Sleeve area [cm²]: 150

Tip area [cm²]: 10

Position: 

Date: 4/8/2010

Scale: 1 : 120

Page: 2/2

Test no:

Client:

Project:

Cone No: DTG6704

Tip area [cm²]: 10

Sleeve area [cm²]: 150
DENOTES LOCATION OF CONE PENETROMETER TESTS PERFORMED: 12 APRIL THROUGH 21 MAY 2010

DENOTES LOCATION OF UNDISTURBED SOIL BORINGS DRILLED: 6 APRIL THROUGH 10 MAY 2010

DENOTES LOCATION OF ORIGINALLY PROPOSED TANK; LOCATION NO LONGER PLANNED

DENOTES REVISED LOCATION OF PROPOSED TANK

C-239
NOTE: NUMBERS TO THE LEFT OF THE LOG ARE STANDARD PENETRATION TEST RESULTS.
SECTION A-A

NOTE: DRAWING INDICATES SCHEMATIC OF PROPOSED RECOMMENDATIONS. THE LIMITS OF THE EARTH PRELOAD MAY PROPORTIONED ACCORDING TO ALTERNATE TANK DIAMETERS.
(Estimated minimum 12' crown; additional sand fill may be added to create crown in tank floor after tank hydrotest)

Existing ground surface

Clay fill

Sand fill

Claymax liner

Concrete ringwall

Clay fill

Degraded clay fill from preload

Approx. edge of tank

Approx. center of tank

Vertical wick drains

Minimum wick penetration below existing ground surface

Continuous horizontal strip drain

Ringwall

Clay max liner

SAND FILL

Horizontal wick drains

Weep hole

NOT TO SCALE
SCHEMATIC ONLY
1 5/8" OUTSIDE DIAMETER FLUSH COUPLED ROD
INSERTED INTO 1 1/2" PIPE AND WELDED

PAPER AND GREASE WRAPPING AROUND ROD

4" LENGTH OF 1 1/2" PIPE WELDED TO BASE

2' X 2' 3/8" STEEL PLATE

WELD

EXISTING GROUND SURFACE

4" SAND BLANKET

TYPICAL SETTLEMENT PLATE
<table>
<thead>
<tr>
<th>Ground Elev.:</th>
<th>Datum:</th>
<th>Gr. Water Depth:</th>
<th>Sample Number</th>
<th>Depth In Feet</th>
<th>Water Content Percent</th>
<th>Density</th>
<th>Shear Tests</th>
<th>Atterberg Limits</th>
<th>Other Tests</th>
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C-255
**LOG OF BORING AND TEST RESULTS**

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**Comments:**

C-256
# Log of Boring and Test Results

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**Water Content Percent:**

**Density:**

**Shear Tests:**

**Atterberg Limits:**

**Other Tests:**

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Comments:

C-259
## Eustis Engineering Services, L.L.C.  Log of Boring and Test Results

### (Sheet 2 of 2)

**Ground Elev.:**

**Datum:**

**Gr. Water Depth:** Se

**Date:** 5/07 & 10/10

**Boring:** T-21

**Refer to "Legends & Notes"**

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C-260
### Log of Boring and Test Results

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**Visual Classification**
- Stiff gray clay
- Medium stiff gray clay w/silt pockets & concretions
- Medium stiff gray clay
- Soft gray sandy clay
- Medium stiff gray sandy clay w/silt pockets & lenses
- w/decayed wood, roots, & sh frag

**Sample Number**
- 13
- 14
- 15
- 16
- 17
- 18
- 19
- 20
- 21
- 22

**Depth In Feet**
- 54-55
- 59-60
- 64-65
- 69-70
- 74-75
- 79-80
- 84-85
- 89-90
- 94-95
- 99-100

**Water Content Percent**
- 43
- 77
- 110
- 51
- 71
- 107
- 43
- 77
- 110
- 43

**Density**
- Dry
- Wet

**Shear Tests**
- Type
- \( \sigma \)
- C

**Atterberg Limits**
- LL
- PL
- PI

**Comments:**

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Comments:

C-263
## LOG OF BORING AND TEST RESULTS

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Comments:

C-265
### EUSTIS ENGINEERING SERVICES, L.L.C.  LOG OF BORING AND TEST RESULTS

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**Comments:**

- Soft gray clay w/silt pockets
- Medium stiff gray clay
- Medium stiff gray clay
- Medium stiff gray clay
- Loose gray silty sand w/clay lenses & trace of organic matter

**Boring:** T-31

- 4/19-20/10

**Refer to "Legends & Notes"**

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- UC
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**Visual Classification:**
- ML: Medium loose
- CL: Clayey Loam
- SC: Silt Clay
- SP: Silt Poor
- CH: Clayey Humus

**Water Content Percent:**
- Dry
- Wet

**Density:**
- UC: Undrained

**Shear Tests:**
- Type
- $\phi$
- C

**Atterberg Limits:**
- LL
- PL
- PI

**Comments:**
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C-269
# Log of Boring and Test Results

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**Comments:**

C-270
Clayey silt to silty clay (5)
Clay (3)
Clayey silt to silty clay (5)
Clay (3)
Silty clay to clay (4)
Clay (3)
Organic material (2)
Clay (3)
Clay (3)
Clay (3)
Clay (3)
Clayey silt to silty clay (5)
Clay (3)
Clay (3)
Clayey silt to silty clay (5)
Clay (3)
Silty clay to clay (4)
Clay (3)
Silty clay to clay (4)
Clay (3)
Silty clay to clay (4)
Clay (3)
Silty clay to clay (4)
Clay (3)
Silty clay to clay (4)
Clayey silt to silty clay (5)
Clayey silt to silty clay (5)
Silty clay to clay (4)
Clayey silt to silty clay (5)
Silty clay to clay (4)
Sandy silt to clayey silt (6)
Clayey silt to silty clay (5)
Silty clay to clay (4)
Clayey silt to silty clay (5)
Silty clay to clay (4)
Clayey silt to silty clay (5)
Silty clay to clay (4)
Clay (3)
Silty clay to clay (4)
Sandy silt to clayey silt (6)
Clayey silt to silty clay (5)
Clayey silt to silty clay (5)
Cone No: DTA1082
Tip area (cm²): 10
Sleeve area (cm²): 150

Test no: TK22-3
Client: NUSTAR ENERGY L.P.
Project: PROPOSED TANKAGE LAYOUT

Position: N30° 01.513', W90° 51.506'

Ground level:
Date: 4/13/2010
Scale: 1 : 120
Page: 2/2

Depth [ft]


Clayey silt to silty clay (5)
Clay (3)
Clay (3)
Silty clay to clay (4)
Clayey silt to silty clay (5)
Silty clay to clay (4)
Clayey silt to silty clay (5)
Sand to silty sand (8)
Silty sand to sandy silt (?)
Cone No: DTA1082
Tip area [cm²]: 10
Sleeve area [cm²]: 150

Test no:
TK22-4
Client:

Project:
Clayey silt to silty clay (5)
Clay (3)
Silty clay to clay (4)
Clayey silt to silty clay (5)
Sand to silty sand (8)
Settle3D Analysis Information

Project Settings

Document Name: 20878-228ft-dia.s3z
Date Created: 3/29/2011, 6:26:57 AM
Stress Computation Method: Westergaard
Time-dependent Consolidation Analysis
Time Units: months
Permeability Units: feet/year
Use average properties to calculate layered stresses
Groundwater method: Water Table
Water Unit Weight: 0.0624 kips/ft³
Depth to water table: 0 [ft]

Stage Settings

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Results

Time taken to compute: 0 seconds

Stage: Stage 1 = 0 mon

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Stage: preload 2mo = 2 mon

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D-278
Permeability [ft/y] 0.0385413 45.1198
Coefficient of Consolidation [ft^2/y] 36.5 730
Hydroconsolidation Settlement [in] 0 0
Average Degree of Consolidation [%] 0 0

Stage: preload removal = 2.01 mon

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Stage: tank installation = 3 mon

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Stage: 50 yrs = 603 mon

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**Loads**

1. **Circular Load**
   - Radius: 114 ft
   - Center: (164, -4.56749e-015)
   - Load Type: Flexible
   - Area of Load: 40621.2 ft²
   - Load: 3.1 ksf
   - Depth: 0 ft
   - Installation Stage: tank installation = 3 mon

**Embankments**

1. **Embankment**
   - Center Line: (0, 0) to (328, 0)
   - Number of Layers: 2
   - Near End Angle: 18.43 degrees
   - Far End Angle: 18.43 degrees
   - Base Width: 323

<table>
<thead>
<tr>
<th>Layer</th>
<th>Stage</th>
<th>Left Bench Width (ft)</th>
<th>Left Angle (deg)</th>
<th>Height (ft)</th>
<th>Unit Weight (kips/ft³)</th>
<th>Right Angle (deg)</th>
<th>Right Bench Width (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Stage 1 = 0 mon</td>
<td>0</td>
<td>18.43</td>
<td>2</td>
<td>0.12</td>
<td>18.43</td>
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<tr>
<td>2</td>
<td>Stage 1 = 0 mon</td>
<td>0</td>
<td>18.43</td>
<td>20</td>
<td>0.12</td>
<td>18.43</td>
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**Soil Layers**

<table>
<thead>
<tr>
<th>Layer #</th>
<th>Type</th>
<th>Thickness [ft]</th>
<th>Depth [ft]</th>
<th>Drained at Bottom</th>
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<tbody>
<tr>
<td>1</td>
<td>GS to 3</td>
<td>3</td>
<td>0</td>
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</tr>
<tr>
<td>2</td>
<td>3 to 7</td>
<td>4</td>
<td>3</td>
<td>No</td>
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<tr>
<td>3</td>
<td>7 to 14</td>
<td>7</td>
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<tr>
<td>4</td>
<td>14 to 20</td>
<td>6</td>
<td>14</td>
<td>No</td>
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<tr>
<td>5</td>
<td>20 to 30</td>
<td>10</td>
<td>20</td>
<td>No</td>
</tr>
<tr>
<td>6</td>
<td>30 to 40</td>
<td>10</td>
<td>30</td>
<td>No</td>
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<tr>
<td>7</td>
<td>40 to 50</td>
<td>10</td>
<td>40</td>
<td>No</td>
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<tr>
<td>8</td>
<td>50 to 60</td>
<td>10</td>
<td>50</td>
<td>No</td>
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<tr>
<td>9</td>
<td>60 to 70</td>
<td>10</td>
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<tr>
<td>10</td>
<td>70 to 80</td>
<td>10</td>
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<td>No</td>
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<td>11</td>
<td>80 to 90</td>
<td>10</td>
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<td>No</td>
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<td>12</td>
<td>90 to 100</td>
<td>10</td>
<td>90</td>
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Ground Surface Drained: Yes

**Soil Properties**

<table>
<thead>
<tr>
<th>Property</th>
<th>GS to 3</th>
<th>3 to 7</th>
<th>7 to 14</th>
<th>14 to 20</th>
<th>20 to 30</th>
<th>30 to 40</th>
<th>40 to 50</th>
<th>50 to 60</th>
<th>60 to 70</th>
<th>70 to 80</th>
<th>80 to 90</th>
<th>90 to 100</th>
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<tbody>
<tr>
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<tr>
<td>Unit Weight [kips/ft³]</td>
<td>0.118</td>
<td>0.12</td>
<td>0.114</td>
<td>0.114</td>
<td>0.107</td>
<td>0.107</td>
<td>0.106</td>
<td>0.114</td>
<td>0.118</td>
<td>0.12</td>
<td>0.12</td>
<td>0.117</td>
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<tr>
<td>Saturated Unit Weight [kips/ft³]</td>
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<td>0.12</td>
<td>0.114</td>
<td>0.114</td>
<td>0.107</td>
<td>0.107</td>
<td>0.106</td>
<td>0.114</td>
<td>0.118</td>
<td>0.12</td>
<td>0.12</td>
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<td>Material Type</td>
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<tr>
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<td>0.125</td>
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<td>0.019</td>
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<td>0.035</td>
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<td>0.021</td>
<td>0.017</td>
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</tbody>
</table>
### Wick Drains

#### Wick Drain Region 1

Installation Stage: Stage 1 = 0 mon  
Cross-Section Shape: Circular  
Equivalent Drain Diameter: 0.209  
Drain Spacing: 5  
Drain Length: 80  
Drain Pattern: Triangular  
Ratio of diameter of smear zone to diameter of drain: 4.5  
Ratio of undisturbed to smear zone permeability: 3.33

#### Coordinates

<table>
<thead>
<tr>
<th>X [ft]</th>
<th>Y [ft]</th>
</tr>
</thead>
<tbody>
<tr>
<td>76.6079</td>
<td>-73.2778</td>
</tr>
<tr>
<td>164</td>
<td>-114</td>
</tr>
<tr>
<td>251.329</td>
<td>-73.2778</td>
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<tr>
<td>278</td>
<td>-1.42109e-014</td>
</tr>
<tr>
<td>237.278</td>
<td>87.3291</td>
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<td>164</td>
<td>114</td>
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</tbody>
</table>

#### Query Points

<table>
<thead>
<tr>
<th>Point #</th>
<th>(X,Y) Location</th>
<th>Number of Divisions</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>164, 0</td>
<td>Auto: 73</td>
</tr>
<tr>
<td>2</td>
<td>278, -1.42109e-014</td>
<td>Auto: 73</td>
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<tr>
<td>3</td>
<td>164, 114</td>
<td>Auto: 73</td>
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</tbody>
</table>

#### Field Point Grid

Number of points: 300  
Expansion Factor: 2

<table>
<thead>
<tr>
<th>X [ft]</th>
<th>Y [ft]</th>
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<tbody>
<tr>
<td>624.038</td>
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<tr>
<td>-296.038</td>
<td>-391.519</td>
</tr>
<tr>
<td>-296.038</td>
<td>391.519</td>
</tr>
</tbody>
</table>
Rebecca Elizabeth Scherer was born in Metairie, Louisiana on 11 January 1983, the daughter of Harold John Scherer, Jr. and Rose Beyerback Scherer. She obtained her Bachelor’s of Science degree in Civil Engineering from the University of New Orleans in the Spring of 2007. She joined the University of New Orleans engineering graduate program in the Fall of 2007. While earning both her undergraduate and graduate degrees, she was employed as an associate engineer with Eustis Engineering Services, L.L.C.