REPORT OF SUBSURFACE EXPLORATION
AND
GEOTECHNICAL ENGINEERING SERVICES

Hopewell Regional Alternative 4A-1 Light Phase 2 PER
(Primary Site)
Hopewell, Virginia

G E T Project No: WM13-136G
October 15, 2013

Prepared for:

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GET Solutions, Inc.
Geotechnical • Environmental • Testing

October 15, 2013
Rev 10/23/2013

TO: **HDR Engineering, Inc.**
5700 Lake Wright Drive, Suite 300
Norfolk, VA 23502

Attn: Mr. William S. M'Coy, P.E.

RE: Report of Subsurface Exploration and Geotechnical Engineering Services
Hopewell Regional Alternative 4A-1 Light Phase 2 PER (Primary Site)
Hopewell, Virginia
G E T Project No: WM13-136G

Dear Mr. M'Coy:

In compliance with your instructions, we have completed our Subsurface Exploration and Geotechnical Engineering Services for the above referenced project. The results of this study, together with our recommendations, are presented in this report.

Often, because of design and construction details that occur on a project, questions arise concerning subsurface conditions. **G E T Solutions, Inc.** would be pleased to continue its role as Geotechnical Engineer during the project implementation.

Thank you for the opportunity to work with you on this project. We trust that the information contained herein meets your immediate need, and should you have any questions or if we could be of further assistance, please do not hesitate to contact us.

Respectfully Submitted,
G E T Solutions, Inc.

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EXECUTIVE SUMMARY

The project site is located at the Hopewell Wastewater Treatment plant which is situated east of Hummel Ross Road and southwest of the James River in the City of Hopewell, Virginia. The construction at this site is expected to consist of expanding the existing wastewater treatment facility to include a MBBR system which will consist of the MBBR Building and associated support structures. The construction at this site is also expected to include the construction of new utility alignments along with other associated infrastructure components.

Our field exploration program included seven (7) 25 to 65-foot deep Standard Penetration Test (SPT) borings drilled by **G E T Solutions, Inc.** within the footprint of the proposed structures. In addition, two (2) 45 to 55-foot deep Cone Penetration Test (CPT) soundings and two (2) 45 to 55-foot deep Dilatometer soundings were performed within the footprint of the proposed MBBR structure. In depth descriptions of the natural subsurface soil conditions encountered at this site are included in Section 3.2 in the body of the report.

The initial groundwater table was measured to occur at depths ranging from 13 to 23 feet below current grades (2 to 13 feet MSL) at the boring locations. The discrepancy between the measured groundwater elevations is likely the result of varying site elevations at the boring locations, and potential perched conditions.

The following evaluations and recommendations were developed based on our field exploration and laboratory-testing program:

- Field testing program during construction to include subgrade proofrolling, compaction testing, and foundation excavation observations for bearing capacity verification.

- An estimated cut of 6 to 12 inches in depth will be required to remove existing topsoil materials; however, based on our experience with similar site conditions (wooded areas) this average initial cut to remove organic laden soils could extend to 18 inches or more. In addition, this cut is expected to extend deeper in isolated areas to remove deeper deposits of unsuitable soils.

- Some subgrade improvements should be anticipated within the construction areas (undercutting and backfilling with select fill) as a result of potentially unsuitable/unstable cohesive subgrade soils.

- Due to the presence of compressible soft Clay soils, minor settlements are anticipated as a result of the fill and structural loading conditions. Accordingly, the MBBR Building footprint can be surcharged with up to 20 feet of fill (41 feet MSL). It is estimated that the surcharge load should remain in place for about 2 to 3 months to sufficiently consolidate the compressible Clay layer.

- Shallow foundations designed using a net allowable bearing capacity of 2,000 psf (24-inch embedment, 24-inch width).

- Estimated post-construction total and differential settlements up to 1.5-inches and ¾-inch, respectively within the MBBR structure's footprint, unless a Surcharge program is implemented as discussed herein.

- Estimated post-construction total and differential settlements up to 1-inch and ½-inch, respectively within the Blower building, DAF building, Future Expansion and SH Tank building footprints.

- Deep foundation design comprised of driven, SPPC piles can be implemented to support the MBBR Building's foundations and slabs should the estimated settlements associated with shallow foundation construction be considered unacceptable. Design capacities are presented below.

Pile Type	Embedment Depth(1) (ft.)	Allowable Compression Capacity (tons)	Pre-Augering Depth (ft)
12" SPPC	55-60	80-100	5-10

(1) Below the base of slab elevation of 21 feet MSL.
(2) Lateral capacity is based on one-half of the lateral load that produces a 1-inch lateral displacement (free-head condition). Batter piles would enhance lateral capacity.

- On the basis of the results of the soil test borings drilled at this site (65 feet maximum explored depth) and our experience in the project area, it is our opinion that this site should be considered a Site Class "D" in accordance with Table 20.3-1 Site Classification of the ASCE 7-10 Minimum Design Loads for Buildings and Other Structures, Chapter 20 (referenced in the 2012 IBC).

This summary briefly discusses some of the major topics mentioned in the attached report. Accordingly, this report should be read in its entirety to thoroughly evaluate the contents.

1.0 PROJECT INFORMATION

1.1 Project Authorization

G E T Solutions, Inc. has completed our subsurface exploration and geotechnical engineering services for the proposed Hopewell Regional Alternative 4A-1 Light Phase 2 PER (Primary Site) project located in Hopewell, Virginia. Authorization to proceed with the services was provided in the form of an executed geotechnical sub-consultant agreement, dated July 18, 2013, signed by Mr. William S. M'Coy, P.E. with HDR Engineering, Inc.

1.2 Project Location and Description

The project site is located at the Hopewell Wastewater Treatment plant which is situated east of Hummel Ross Road and southwest of the James River in the City of Hopewell, Virginia. Specifically, the project site lies to the east of the existing Clarifier Bed structures and west of the James River. This site is currently moderately to heavily wooded and generally slopes upward from the perimeter of the existing Clarifier Bed structures to the central portion of the MBBR Building and then downward to the river with elevations ranging from approximately 20 to 35 feet (MSL) within the proposed structures' footprints.

The construction at this site is expected to consist of expanding the existing wastewater treatment facility to include a MBBR system which will consist of the MBBR Building and associated support structures. The proposed expansions will consist of four (4) new structures to the east of the existing Clarifier Bed structures. The main structure will consist of a roughly 3-story MBBR Building (FFE = 23 feet MSL and top of tank elevation = 51 feet MSL), approximately 37,500-square feet in plan area (150' x 250'). This structure is expected to be supported over a shallow or deep (SPPC piles) foundation system (to be determined by the client based on acceptable settlement levels). This structure is expected to be of reinforced concrete/steel frame construction with a first floor slab approximately 2 feet thick supported on grade or pile foundations. The maximum loaded contact pressure of the Tank is expected to be on the order of 1,560 psf. The existing site grades within the MBBR building footprint range from approximately 20 to 35 feet MSL with a proposed bottom of slab elevation of 21 feet MSL. As such, cuts of up to 14 feet are expected within the western, northern and central portion of the structure's footprint while fills of up to 1 foot are expected within the eastern portion of the structure's footprint. A sloped embankment fill section, approximately 10 to 15 feet in depth, will be required along a portion of the existing slope in order to provide access around the eastern portion of the MBBR Building. It is anticipated that this embankment fill will be constructed on a minimum 3H:1V grade.

In addition to the main structure, three (3) complimentary structures will be constructed at this site as well along with a future expansion area located to the northwest. These structures will consist of a Blower building, a DAF building and a SH Tank Building. These structures are expected to be roughly one to two stories in height with footprints much smaller than the MBBR structure. The structural elements of these buildings were not known at the time of this reporting; however, they are expected to be of typical CMU/steel frame construction with first floor slabs supported on grade. In addition, the structural loading characteristics are expected to be negligible in relation to the MBBR structure. As such, shallow foundation support is expected to be suitable for these structures. The construction at this site is also expected to include the construction of new utility alignments along with other associated infrastructure components.

It is noted that the MBBR structure's footprint was originally planned to be larger in plan area with a higher finished floor elevation and lower distributed contact pressure. However, preliminary calculations of the magnitude of calculated post-construction settlements were considered unacceptable. Subsequently, the structure's footprint was reduced and the proposed finished floor elevation lowered in order to decrease these expected settlements. At the time of the building re-configuration, additional field services were performed to refine the engineering assessment.

In addition to the subsurface exploration performed at this site, further subsurface exploration was also performed at an alternate site to the north. The data collected from these exploration points (SPT Borings F-8, F-9 and F-10; Cone Penetration Test CPT-3 and Dilatometer Test DMT-3) is not included in the scope of this report; however, the results of these explorations are included in their respective appendices (Appendix III and IX).

If any of the noted information is incorrect or has changed, please inform G E T Solutions, Inc. so that we may amend the recommendations presented in this report, if appropriate.

1.3 Purpose and Scope of Services

The purpose of this study was to obtain information on the general subsurface conditions at the proposed project site. The subsurface conditions encountered were then evaluated with respect to the available project characteristics. In this regard, engineering assessments for the following items were formulated:

1. General assessment of the soils revealed by the borings performed at the proposed development.
2. General location and description of potentially deleterious material encountered in the borings that may interfere with construction progress or structure performance, including existing fills or surficial/subsurface organics.

3. Soil subgrade preparation, including stripping, grading and compaction. Engineering criteria for placement and compaction of approved structural fill material.

4. Construction considerations for fill placement, subgrade preparation, and foundation excavations.

5. Evaluation of the on-site soils for re-use as structural fill.

6. Ground improvement (surcharge) recommendations to provide satisfactory soils for slab-on-grade and shallow foundation construction.

7. Feasibility of utilizing a shallow foundation system for support of the proposed structures. Design parameters required for the foundation systems, including foundation sizes, allowable bearing pressures, foundation levels and expected total and differential settlements.

8. Feasibility of utilizing an alternative deep foundation system consisting of driven concrete piles for support of the proposed MBBR structure. Design parameters required for the deep foundation system including pile types, pile lengths, allowable capacities, expected total and differential settlements, and pile installation and testing criteria.

9. Slope stability analysis for the embankment slopes.

10. Design parameters for the below grade retaining walls.

11. Seismic site class determination in accordance with the 2012 International Building Code.

The scope of services did not include an environmental assessment for determining the presence or absence of wetlands or hazardous or toxic material in the soil, bedrock, surface water, groundwater or air, on or below or around this site. Prior to development of this site, an environmental assessment is advisable.

2.0 FIELD AND LABORATORY PROCEDURES

2.1 Field Exploration

In order to explore the general subsurface soil types and to aid in developing associated foundation design parameters, seven (7) 25 to 65-foot deep Standard Penetration Test (SPT) borings (designated as M-1 through M-4, B-5, D-6 and F-7) were drilled within the proposed structures' footprints. It is noted that a SPT boring was not performed within the SH Tank Building's footprint. In addition, the SPT borings were performed during the first phase of our exploration to evaluate the initial MBBR Building layout along with the associated support structures. Specific information regarding the SPT boring depths, elevations and locations is tabulated below in Table I.

Table I - SPT Boring Schedule

Boring No.	Boring Depth (Feet)	Boring Elevation (MSL)	Associated Structure
M-1	65	30	MBBR Building
M-2	40	27	MBBR Building
M-3	50	20	MBBR Building
M-4	30	15	MBBR Building
B-5	50	31	Blower Building
D-6	25	35	DAF Building
F-7	25	31	Future Expansion

The SPT borings were performed with the use of rotary wash “mud” drilling procedures in general accordance with ASTM D 1586. The tests were performed continuously from the existing ground surface to a depth of 12-feet, and at 5-foot intervals thereafter. The soil samples were obtained with a standard 1.4” I.D., 2” O.D., 30” long split-spoon sampler. The sampler was driven with blows of a 140 lb. hammer falling 30 inches, using an automatic hammer. The number of blows required to drive the sampler each 6-inch increment of penetration was recorded and is shown on the boring logs. The sum of the second and third penetration increments is termed the SPT N-value (uncorrected for automatic hammer). A representative portion of each disturbed split-spoon sample was collected with each SPT, placed in a glass jar, sealed, labeled, and returned to our laboratory for review. Two (2) thin-walled tube samples were obtained from the very soft to soft CLAY stratum, by hydraulically pressing a 3-inch outside diameter Shelby tube into the soils. Specifically, the tube samples were obtained at the location of borings M-2 and M-3 at depths ranging from 28 to 35 feet below the existing site grades. The tubes were sealed to prevent moisture loss and returned to the laboratory for extraction, classification and consolidation testing.

In conjunction with a reconfiguration of the MBBR Building layout due to the previously mentioned unacceptable estimated settlement levels, a second phase of subsurface exploration was deemed necessary. Subsequently, a Cone Penetration rig was mobilized to the project site to perform two (2) 45 to 55-foot deep Cone Penetration Test (CPT) soundings (designated as CPT-1 and CPT-2) and two (2) 45 to 55-foot deep Dilatometer soundings (designated as DMT-1 and DMT-2). Dissipation testing was performed at the location of CPT-2 at a depth of 33 feet below existing site grades. The results of the CPT and Dilatometer testing are presented in Appendix IX.

The SPT, CPT and DMT boring locations were established and staked in the field by a representative of **G E T Solutions, Inc.** The approximate boring locations are shown on the attached “Boring Location Plan” (Appendix I), which was reproduced based on the site plan provided by the client. Please note: based on limited information made available by the Client, the ground surface elevations shown on the Boring Logs and utilized throughout our evaluations are estimated values rather than specific spot elevations. Once a survey is obtained establishing the Boring elevations with certainty, we request the opportunity to review the information as related to our analyses to ensure the continuing applicability of the recommendations made herein.

2.2 Laboratory Testing

Representative portions of all soil samples collected during drilling were sealed in glass jars, labeled and transferred to our laboratory for classification and analysis. The soil classification was performed by a Geotechnical Engineer in accordance with ASTM D2488. A summary of the soil classification system is provided in Appendix II.

Eighteen (18) representative soil samples were selected and subjected to laboratory testing, which included natural moisture, -#200 sieve wash and Atterberg Limit testing and analysis, in order to corroborate the visual classification. These test results are provided in the table (Table II) below and are presented on the "Boring Log" sheets (Appendix III), included with this report.

Table II - Laboratory Test Results

Boring No.	Depth* (Feet)	Natural Moisture (%)	% Passing #200	Atterberg Limits (LL/PL/PI)	USCS Classification
M-1	23-25	24	89	43/17/26	CL
M-1	33-35	22	8	Not Tested	SP-SM
M-1	43-45	29	60	34/21/13	CL
M-2	18-20	2	9	Not Tested	SP-SM
M-2	23-25	25	10	Not Tested	SP-SM
M-2**	33-35	35	59	33/16/17	CL
M-3	13-15	13	7	Not Tested	SP-SM
M-3	18-20	16	74	21/16/5	CL
M-3	23-25	18	18	Not Tested	SM
M-3**	28-30	49	99	63/27/36	CH
M-4	4-6	14	58	29/21/8	CL
M-4	8-10	7	12	Not Tested	SP-SM
M-4	13-15	17	12	Not Tested	SP-SM
M-4	18-20	42	99	59/24/35	CH
B-5	18-20	24	95	52/23/29	CH
B-5	38-40	50	99	52/26/26	CH
D-6	10-12	29	98	58/28/30	CH
F-7	18-20	2	7	Not Tested	SP-SM

**Depth below existing grades.*
***Shelby Tube Sample*

Two (2) one-dimensional consolidation tests were performed on specimens from the Shelby tube samples obtained at the location of borings M-2 and M-3 at depths of 33 to 35 and 28 to 30 feet below the existing site grades, respectively. The consolidation tests were performed at our Virginia Beach laboratory in general accordance with ASTM D 2435. A representative specimen from each of the Shelby tubes was also subjected to natural moisture content, Atterberg Limits, and -#200 sieve testing. A summary of the consolidation test results are provided below in Table III and the comprehensive results are provided in Appendix V.

Table III - Consolidation Test Results

Boring No.	Depth (ft)	Natural Moisture (%)	Overburden Pressure (tsf)	Pre consolidation Pressure P_c (tsf)	C_c	C_r	e_o
M-2*	34	35.0	1.74	-	-	-	-
M-3	29	48.6	1.30	2.30	0.71	0.10	1.435

** Test results are not considered representative of the subsurface soil conditions encountered across the site.*

One (1) thin walled tube soil sample was selected and subjected to unconsolidated-undrained triaxial compression (ASTM D 2850) testing and analysis. A summary of the triaxial compression test results are provided below (Table IV) and the comprehensive triaxial compression test results are provided in Appendix VI.

Table IV - Unconsolidated-Undrained Triaxial Compressive Strength Test Results

Boring No.	Depth* (ft)	USCS Classification	Cohesion c (lbs/ft²)
M-3	28-30	CH	458

**Depth below existing grade.*

3.0 SITE AND SUBSURFACE CONDITIONS

3.1 Site Geology

The project site lies within a major physiographic province called the Atlantic Coastal Plain. Numerous transgressions and regressions of the Atlantic Ocean have deposited marine, lagoonal, and fluvial (stream lain) sediments. The regional geology is very complex, and generally consists of interbedded layers of varying mixtures of sands, silts and clays. Based on our review of existing geologic and soil boring data, the geologic stratigraphy encountered in our subsurface explorations generally consisted of marine deposited sands and clays.

3.2 Subsurface Soil Conditions

The results of our soil test borings indicated the presence of approximately 1 to 4 inches of topsoil material at the boring locations. The topsoil material thickness is expected to varying at other locations throughout the site. It is noted that the topsoil thicknesses included on the boring logs and noted above are not expected to be indicative of the thicknesses that will be encountered across the site as the boring locations were partially cleared for accessibility prior to drilling. Underlying the surficial materials and extending to the boring termination depths ranging from 25 to 65 feet below existing site grades, the natural subsurface soils were generally arranged into four stratums. It is noted that the deeper strata were not encountered at the boring locations drilled to shallower depths.

Blower Building (B-5):

The initial soil layer extended beneath the surficial materials to a depth of 10 feet below the existing site grades. The recovered soils were comprised of SILT (ML) with varying amounts of Clay and Sand and lean to fat CLAY (CL, CH) with varying amounts of Silt and Sand. The Standard Penetration Test (SPT) Results, N-values, recorded within the cohesive soils of this layer ranged from 8 to 36 blows-per-foot, indicating a medium stiff to hard consistency. Deposits of fine organics (grass roots) were sampled within the upper 2 feet of this stratum.

The second soil layer extended beneath the initial soil layer to a depth of 14 feet below the existing site grades. The recovered soils were classified as SAND (SM) with varying amounts of Silt and Clay. The N-values recorded within the granular soils of this layer ranged from 9 to 12 blows-per-foot, indicating a loose to medium dense relative density.

The third soil layer extended beneath the second soil layer to a depth of 24.5 feet below existing grades. The recovered soils were comprised of fat CLAY (CH) with varying amounts of Silt and Sand. The N-values recorded within the cohesive soils of this layer ranged from 9 to 17 blows-per-foot, indicating a stiff to very stiff consistency.

The final soil layer extended beneath the third soil layer to the boring termination depth of 50 feet below existing site grades. The recovered soils were comprised of SAND (SM, SP-SM) with varying amounts of Silt, Clay and marine shell fragments. The N-values recorded within the granular soils of this layer ranged from 11 to 18 blows-per-foot, indicating a medium dense relative density. A deposit of very soft, fat CLAY (CH) with varying amounts of Silt and Sand was sampled at depths ranging from 38 to 43 feet below the existing site grades.

DAF Building (D-6):

The initial soil layer extended beneath the surficial materials to a depth of 4 feet below the existing site grades. The recovered soils were comprised of lean CLAY (CL) with varying amounts of Silt and Sand. The N-values recorded within the cohesive soils of this layer ranged from 7 to 11 blows-per-foot, indicating a medium stiff to stiff consistency. Deposits of fine organics (grass roots) were sampled within the upper 2 feet of this stratum.

The second soil layer extended beneath the initial soil layer to a depth of 8 feet below the existing site grades. The recovered soils were classified as SAND (SC) with varying amounts of Silt and Clay. The N-values recorded within the granular soils of this layer ranged from 15 to 20 blows-per-foot, indicating a medium dense relative density.

The third soil layer extended beneath the second soil layer to a depth of 14 feet below existing grades. The recovered soils were comprised of fat CLAY (CH) with varying amounts of Silt and Sand. The N-values recorded within the cohesive soils of this layer ranged from 17 to 20 blows-per-foot, indicating a very stiff consistency.

The final soil layer extended beneath the third soil layer to the boring termination depth of 25 feet below existing site grades. The recovered soils were comprised of SAND (SM, SP-SM) with varying amounts of Silt and Clay. The N-values recorded within the granular soils of this layer ranged from 11 to 20 blows-per-foot, indicating a medium dense relative density.

Future Expansion (F-7):

The initial soil layer extended beneath the surficial materials to a depth of 10 feet below the existing site grades. The recovered soils were comprised of lean to fat CLAY (CL, CH) with varying amounts of Silt and Sand. The N-values recorded within the cohesive soils of this layer ranged from 7 to 33 blows-per-foot, indicating a medium stiff to hard consistency. Deposits of fine organics (grass roots) were sampled within the upper 2 feet of this stratum.

The second soil layer extended beneath the initial soil layer to a depth of 23 feet below the existing site grades. The recovered soils were classified as SAND (SM, SP-SM) with varying amounts of Silt and Clay. The N-values recorded within the granular soils of this layer ranged from 10 to 16 blows-per-foot, indicating a loose to medium dense relative density.

The final soil layer extended beneath the second soil layer to the boring termination depth of 25 feet below existing site grades. The recovered soils were comprised of fat CLAY (CH) with varying amounts of Silt and Sand. The N-value recorded within the cohesive soils of this layer was 14 blows-per-foot, indicating a stiff consistency.

MBBR Building (M-1 through M-4):

The initial soil layer extended beneath the surficial materials to depths ranging from 7.5 to 28 feet below the existing site grades. The recovered soils were comprised of SILT (ML) with varying amounts of Clay and Sand and lean to fat CLAY (CL, CH) with varying amounts of Silt and Sand. The N-values recorded within the cohesive soils of this layer ranged from 5 to 46 blows-per-foot, indicating a medium stiff to hard consistency. Deposits of medium dense SAND (SM) were sampled at depths ranging from 2 to 13 feet below the existing site grades at boring locations M-1, M-2 and M-3. Deposits of fine organics (grass roots) and wood fragments were sampled at the boring locations within the upper 2 feet of this stratum.

The second soil layer extended beneath the initial soil layer to depths ranging from 18 to 38 feet below the existing site grades. The recovered soils were classified as SAND (SM, SP-SM) with varying amounts of Silt and Clay. The N-values recorded within the granular soils of this layer ranged from 5 to 24 blows-per-foot, indicating a loose to medium dense relative density. A deposit of very stiff, Silty CLAY (CL-ML) with varying amounts of Silt and Sand was sampled within this stratum at depths ranging from 18 to 23 feet below the existing site grades at boring location M-3.

The third soil layer extended beneath the second soil layer to depths ranging from 24 to 48 feet below existing grades. The recovered soils were comprised of lean to fat CLAY (CL, CH) with varying amounts of Silt and Sand. The N-values recorded within the cohesive soils of this layer ranged from Weight-of-Hammer (WOH) to 6 blows-per-foot, indicating a very soft to medium stiff consistency.

The final soil layer extended beneath the third soil layer to the boring termination depths of 30, 40, 50 and 65 feet below existing site grades. The recovered soils were comprised of SAND (SM, SP-SM) with varying amounts of Silt, Clay and marine shell fragments. The N-values recorded within the granular soils of this layer ranged from 10 to 43 blows-per-foot, indicating a loose to dense relative density. A deposit of medium stiff, fat CLAY (CH) was sampled at depths ranging from 53 to 58 feet below the existing site grades at boring location M-1.

The subsurface description is of a generalized nature provided to highlight the major soil strata encountered. The records of the subsurface exploration are included on the “Boring Log” sheets (Appendix III), in the “Generalized Soil Profile” (Appendix IV) and on the “CPT/DMT Test Results” sheets (Appendix IX), which should be reviewed for specific information as to the individual borings. The stratifications shown on the records of the subsurface exploration represent the conditions only at the actual boring locations. Variations may occur and should be expected between boring locations. The stratifications represent the approximate boundary between subsurface materials and the transition may be gradual or occur between sample intervals. It is noted that the topsoil designation references the presence of surficial organic laden soil, and does not represent any particular quality specification. This material is to be tested for approval prior to use.

3.3 Groundwater Information

The groundwater level was recorded at the boring locations and as observed through the wetness of the recovered soil samples during the drilling operations. The initial groundwater table was measured to occur at depths ranging from 13 to 23 feet below current grades (2 to 13 feet MSL) at the boring locations. The discrepancy between the measured groundwater elevations is likely the result of varying site elevations at the boring locations, and potential perched conditions. The groundwater level was not encountered at the location of boring F-7 to the depth explored. The boreholes were backfilled upon completion for safety considerations. As such, the reported groundwater levels may not be indicative of the static groundwater level.

As subsurface soils begin to dry moisture moves upwards through the soil profile by means of capillary action. Based on the subsurface soil composition (soils containing more than 30% of fines by weight), these initial groundwater readings (based on the relative wetness of the soils) could be in part attributed to the capillary action of the soils. As such, if the static groundwater elevation is critical to the design of the proposed structures and site infrastructure it is recommended to install temporary groundwater monitoring wells to substantiate these initial readings.

Groundwater conditions will vary with environmental variations and seasonal conditions, such as the frequency and magnitude of rainfall patterns, as well as man-made influences, such as existing swales, drainage ponds, underdrains and areas of covered soil (paved parking lots, sidewalks, etc.). Seasonal groundwater fluctuations of $\pm$ 2 feet are common in the project's area; however, greater fluctuations have been documented. We recommend that the contractor determine the actual groundwater levels at the time of the construction to determine groundwater impact on the construction procedures.

4.0 EVALUATIONS AND RECOMMENDATIONS

Our recommendations are based on the previously discussed project information, our interpretation of the soil test borings and laboratory data, and our observations during our site reconnaissance. If the proposed construction should vary from what was described, we request the opportunity to review our recommendations and make any necessary changes.

The anticipated loads associated with the MBBR Building are expected to result in a stress increase within the very soft to medium stiff CLAY (CL, CH) stratum. This stress increase is expected to induce consolidation settlements that will occur over a longer time span than typical post construction settlements based on short-term elastic compression. The total estimated settlements reported herein are anticipated to occur as approximately 70% to 75% short-term compression (during construction to within approximately 3 months of full service load application) and approximately 25% to 30% longer term consolidation occurring within approximately 1 to 1.5 years of completion.

4.1 Clearing and Grading

The proposed construction area should be cleared by means of removing the topsoil material, trees, associated root mat and any other unsuitable materials. It is estimated that a cut of up to 6 to 12 inches in depth will be required to remove the topsoil and root mat material. As such, based on our experience with similar site conditions (wooded areas) this initial cut to remove organic laden soils could extend to 18 inches or more. This cut is expected to extend deeper in isolated areas to remove deeper deposits of organic soils, or unsuitable soils, which become evident during the clearing particularly in the wooded areas of the project site. It is recommended that the clearing operations extend laterally at least 5 feet beyond the perimeter of the proposed construction areas.

Following the initial clearing, the resulting exposed subgrade will generally be comprised of lean CLAY (CL) containing an appreciable amount of fines (Silt and Clay). Accordingly, combinations of excess surface moisture from precipitation ponding on the site and the construction traffic, including heavy compaction equipment, may create pumping and general deterioration of the bearing capabilities of the surface soils. Therefore, undercutting to remove loose/soft soils in isolated areas should be expected. The extent of the undercut will be determined in the field during construction based on the outcome of the field testing procedures (subgrade proofroll). In this regard, and in order to reduce undercutting, care should be exercised during the grading and construction operations at the site.

Inherently wet subgrade soils combined with potential poor site drainage make this site particularly susceptible to subgrade deterioration. Thus, grading operations at this site will be more economical if performed during the drier months of the year (historically April through November). This should minimize these potential problems, although they may not be eliminated. If grading is attempted during the winter months, undercutting of wet soils should be anticipated. However, during the drier months of the year, wet soils could be dried by disking or other drying procedures to achieve moisture contents necessary to achieve adequate degrees of compaction.

Similar projects have required significant improvements to stabilize or bridge unstable subgrade soils, which tend to deteriorate when exposed to construction traffic and moisture. The subgrade improvements typically consist of additional cuts of up to 1-foot and replacement with structural fill to improve poor subgrade soil conditions. The project's budget should include an allowance for subgrade improvements (undercut and backfill with structural fill or aggregate base particularly in the pavement areas).

4.2 Subgrade Preparation

Following the clearing operation, the exposed subgrade soils should be densified with a large static drum roller. After the subgrade soils have been densified, they should be evaluated by **G E T Solutions, Inc.** for stability. Accordingly, the subgrade soils should be proofrolled to check for pockets of loose material hidden beneath a crust of better soil.

Several passes should be made by a large rubber-tired roller or loaded dump truck over the construction areas, with the successive passes aligned perpendicularly. The number of passes will be determined in the field by the Geotechnical Engineer depending on the soils conditions. Any pumping and unstable areas observed during proofrolling (beyond the initial cut) should be undercut and/or stabilized at the directions of the Geotechnical Engineer. The project's budget should include an allowance for subgrade improvements (undercut and backfill with structural fill).

4.3 Structural Fill and Placement

Following the approval of the natural subgrade soils by the Geotechnical Engineer, the placement of the fill required to establish the design grades may begin. Any material to be used for structural fill should be evaluated and tested by **G E T Solutions, Inc.** prior to placement to determine if they are suitable for the intended use. Suitable structural fill material should consist of sand or gravel containing less than 25% by weight of fines (SP, SM, SW, GP, GW), having a liquid limit less than 20 and plastic limit less than 6, and should be free of rubble, organics, clay, debris and other unsuitable material.

The subsurface soils encountered at the boring locations do not appear to meet the criteria recommended in this report for reuse as structural fill, but may be used as fill within green areas or as surcharge fill. Further classification testing (natural moisture content, gradation analysis, and Proctor testing) should be performed in the field during construction to evaluate the suitability of excavated soils for reuse as fill within building and pavement areas.

All structural fill should be compacted to a dry density of at least 95 percent of the Standard Proctor maximum dry density (ASTM D698). In general, the compaction should be accomplished by placing the fill in maximum 10-inch loose lifts and mechanically compacting each lift to at least the specified minimum dry density. A representative of **G E T Solutions, Inc.** should perform field density tests on each lift as necessary to assure that adequate compaction is achieved.

Backfill material in utility trenches within the construction areas should consist of structural fill (as previously above), and should be compacted to at least 95 percent of ASTM D698. This fill should be placed in 4 to 6 inch loose lifts when hand compaction equipment is used.

Care should be used when operating the compactors near existing structures to avoid transmission of the vibrations that could cause settlement damage or disturb occupants. In this regard, it is recommended that the vibratory roller remain at least 25 feet away from existing structures; these areas should be compacted with small, hand-operated compaction equipment.

4.4 Suitability of On-site Soils

Based on the laboratory testing program and visual classifications, the shallow subsurface SILT (ML), CLAY (CL, CH) and SAND (SC, SM) soils encountered at the boring locations, do not appear to meet the criteria recommended in this report (Section 4.3) for reuse as structural fill. As such, it will likely be necessary to import structural fill to expedite the utility backfilling. Further classification testing (natural moisture content, gradation analysis, and Proctor testing) should be performed in the field during construction to evaluate the suitability of excavated soils for reuse as backfill within building and utility areas.

4.5 Settlement Discussion (MBBR Building)

As previously mentioned, the expected fill and structural loading conditions (foundations and slab) associated with the proposed MBBR structure are expected to result in a net stress increase within the soft to medium stiff CLAY (CL, CH) stratum (elevation ranging from -3 to -18 feet MSL, with a typical thickness of 5 to 10 feet), which is expected to induce consolidation settlements. The magnitude and duration of the settlement associated with these loading conditions is important to planning the construction. We evaluated settlement under conditions of maximum loading across what is considered among the least favorable Tank profiles.

We selected a maximum floor distributed loading of 1,560 psf along with a maximum fill height of 1 foot under the structure (along with adjacent embankment construction) and evaluated them with the appropriate subsurface information from the Soil Test Borings, In-Situ Testing (CPT/DMT) and laboratory testing. Based on the consolidation test results, the compressible soils located at the location of boring M-3 were considered representative of subsurface conditions where the settlement would be the greatest and the time to substantial completion of settlement would be the longest. The magnitude and duration of the settlement at locations with smaller loads are expected to be less than the values calculated for these locations. The subsurface soil parameters used during our analysis of this section were estimated by using the SPT boring data, the results of laboratory classification and consolidation testing, In-Situ Testing (CPT/DMT), and our experience with sites in the vicinity of the project.

For evaluation of settlement potential, we utilized the GeoStudio software program SIGMA/W™ for finite element modeling and analysis of applied stress and resulting deformation. Based on our analysis, a maximum total settlement magnitude ranging from about 1.75 to 2.5 inches is expected to occur within the subsurface soils as a result of the structural loads and fill required to achieve the design grade elevations. The total estimated settlements reported herein are anticipated to occur as approximately 70% to 75% short term compression (during construction to within approximately 3 months of full service load application) and approximately 25% to 30% longer term consolidation occurring within approximately 1 to 1.5 years of completion.

It is estimated that, with proper site preparation, the maximum resulting post construction total settlement of the proposed Tank structure should be up to approximately 1.75 to 2.5 inches. The maximum differential settlement magnitude is expected to be less than about ¾-inch within an approximate 50 foot span across the structure width. A depiction of the anticipated settlement profile is included with the comprehensive Finite Element Analysis - Stress Deformation (Appendix VII). The settlements were estimated on the basis of the results of the field penetration tests and consolidation testing. Careful field control will contribute substantially towards minimizing the settlements.

These settlement magnitudes are based on the provided structural characteristics (maximum floor loading of 1,560 psf, site grading as indicated and maximum 1 foot of fill expected to achieve the finished floor elevation. If any of the this information is incorrect or has changed, G E T Solutions, Inc. must be notified so that we may amend the recommendations presented in this report, if appropriate.

4.6 Surcharge (MBBR Building)

Should the estimated settlements associated with shallow foundation construction provided in Section 4.5 be considered unacceptable; ground improvements by means of surcharging portions of the building area can be implemented. Surcharging is a process where a temporary weight is placed on the construction area so that the subsurface soils can settle (consolidate) prior to the foundation construction. It is well suited for clayey or other low permeability soils that require long periods of time to compress.

It is noted that cuts are expected in the western and northern portions of the building footprint and these areas (existing elevation of approximately 28 feet MSL or higher) will not require surcharge. The cut soils from these areas can be placed on the southeastern portion of the building footprint (fill areas) as part of the surcharge program.

It is recommended to surcharge the building area with 15 to 20 feet of fill (in addition to the structural fill needed to achieve the design grade elevations), bringing the top of the surcharge to an elevation of 36 to 41 feet MSL. The upper crest of the surcharge soil (edge of the top of the fill) should extend to at least the design edge of the building, then sloping at an approximate angle of 2H:1V. The surcharge soil should have a minimum in-place dry density of at least 115 pcf. It is estimated that the surcharge load, applied to an elevation of approximately 41 feet MSL, should remain in place for about 2 months to sufficiently consolidate the compressible Clay layer, with the objective in mind to reduce the remaining long-term settlement to be experienced by the structure to less than approximately 0.5 to 0.75 inch. The surcharge height can be modified to accommodate different construction schedules (i.e. lesser surcharge height resulting in longer surcharge time and vice versa). Table V on the following page illustrates Recommended Surcharge Heights, Surcharge Duration, Short-term (surcharge period) Minimum Slope Stability Safety Factor, Total Settlement, Surcharge Period Settlement and Resulting Post-Construction Settlement for selected options:

Table V – Surcharge Program Variables

Surcharge Height (ft)	Recommended Surcharge Duration	Short-Term Slope Stability Safety Factor	Total Estimated Settlement (in)	Surcharge Period Settlement (in)	Post-Construction Settlement (in)
---	---	---	1.75 – 2.5	---	1.75 – 2.5
20	2 mo.	1.26	2.0 – 2.5	1.5 – 2.0	0.50
15	3 mo.	1.46	1.75 – 2.5	1.75 – 2.0	0.50

In order to accurately determine when the surcharge load can be removed, it is recommended to install six (6) settlement platforms. The settlement platforms should be placed directly on the subgrade following the clearing procedures. Then, following the installation of the settlement platforms, elevations must be obtained (zero/baseline readings) prior to the placement of any fill material. It is recommended to install the building pad structural fill material to the finish floor elevation, with each lift compacted to at least 95% of ASTM D698. Then the contractor can proceed with the surcharge placement.

During the surcharge placement activities, elevation readings should be obtained daily. Following the completion of the permanent fill and surcharge placement, the readings should be obtained twice a week. The settlement platform readings should be performed to the nearest .001 foot and should be provided to the structural and the geotechnical engineer for their analyses. All readings should be performed under the direction of a licensed surveyor. Once the consolidation of the Clay material is determined to be substantially complete, the surcharge soil can be removed at the direction of the geotechnical engineer.

4.6.1 Settlement Platform Description

Settlement platforms are surface displacement reference platforms placed on the prepared ground surface at predetermined locations typically prior to select fill and surcharge fill placement. Settlement platforms shall consist of 3-foot square plates to which risers are attached. The risers are extended as the fill is placed. Settlement platforms are monitored by optical survey methods to determine vertical displacements occurring during and after the surcharge soil layer construction.

The base plate should be made from 1/8-inch thick steel plate conforming to the requirements of ASTM A 36. The inner riser pipe should be 1 ½-inch diameter steel pipe conforming to the requirements of ASTM A 53, welded and of standard weight. The casing should consist of 3-inch diameter PVC pipe. The maximum length of the riser pipe and casing sections should be limited to 5 feet. Centralizing spacers should also be provided to maintain the alignment of the exterior casing.

4.6.2 Settlement Platform Installation

The settlement platforms should be installed on a 4-inch thick sand base on the existing ground surface. The riser pipe should be marked in 1-foot increments and labeled at 5-foot increments to indicate the distance above the platform extending up through the fill. The sand base should be tamped to provide a firm, level, and unyielding bearing surface for the base plate. The original ground surface must be stripped of all vegetation prior to placement of the sand base. Spacers should be provided between the riser pipe and the casing at a minimum of 3-foot intervals to ensure concentricity. A container, approximately 2.5 feet in diameter and 3 feet high, with both ends open, should be placed around the initial length of casing pipe. This container should be backfilled with tamped clean sand or fine gravel to support the pipe in a vertical position during the fill placement until the fill is carried above the platform.

The casing pipe should be marked by flags to clearly show its location and to warn equipment operators and others of its location. The contractor should maintain the flags during the entire length of the contract and replace those flags that are missing. At no time should the settlement platform risers extend higher than 5 feet above the fill surface elevation.

4.7 Shallow Foundation Design Recommendations (Appurtenant Structures)

Provided that the construction procedures are properly performed, the proposed structure can be supported by shallow spread footings bearing upon firm natural soil or well compacted structural fill material. The footings can be designed using a net allowable soil pressure of 2,000 pounds per square foot (psf). In using net pressures, the weight of the footings and backfill over the footings, including the weight of the floor slab, need not be considered. Hence, only loads applied at or above the finished floor need to be used for dimensioning the footings. In order to develop the recommended bearing capacity of 2,000 pounds per square foot (psf), the base of the footings should have an embedment of at least 24 inches beneath finished grades and wall footings should have a minimum width of 24 inches. In addition, isolated square column footings are recommended to be a minimum of 3 feet by 3 feet in area for bearing capacity consideration. The recommended 24-inch footing embedment is considered sufficient to provide adequate cover against frost penetration to the bearing soils.

4.8 Settlements (Shallow Foundations - Blower, DAF, Future Expansion and SH Tank Buildings)

It is estimated that, with proper site preparation, the maximum resulting post construction total settlement of the proposed buildings' foundations should be up to 1 inch. The maximum differential settlement magnitude is expected to be less than ½ -inch between adjacent footings (wall footings and column footings of widely varying loading conditions). The settlements were estimated on the basis of the results of the field penetration tests. Careful field control will contribute substantially towards minimizing the settlements.

4.9 Foundation Excavations

In preparation for shallow foundation support, the footing excavations should extend into firm natural soil or well-compacted structural fill. All foundation excavations should be observed by **G E T Solutions, Inc.** At that time, the Geotechnical Engineer should also explore the extent of excessively loose, soft, or otherwise unsuitable material within the exposed excavations. Also, at the time of footing observations, the Geotechnical Engineer may find it necessary to make hand auger borings or use a hand penetration device in the bases of the foundation excavations.

If pockets of unsuitable soils requiring undercut are encountered in the footing excavations, the proposed footing elevation should be re-established by means of backfilling with “flowable fill” or a suitable structural fill material compacted to a dry density of at least 98 percent of the Standard Proctor maximum dry density (ASTM D 698), as described in Section 4.3 of this report, prior to concrete placement. This construction procedure will provide for a net allowable bearing capacity of 2,000 psf.

Immediately prior to reinforcing steel placement, it is suggested that the bearing surfaces of all footing and floor slab areas be compacted using hand operated mechanical tampers, to a dry density of at least 95% of the Standard Proctor maximum dry density (ASTM D 698) as tested to a depth of 12 inches, for bearing capacity considerations. In this manner, any localized areas, which have been loosened by excavation operations, should be adequately recompacted. The compaction testing in the base of the footings may be waived by the Geotechnical Engineer, where firm bearing soils are observed during the footing inspections.

Soils exposed in the bases of all satisfactory foundation excavations should be protected against any detrimental change in condition such as from physical disturbance, rain or frost. Surface run-off water should be drained away from the excavations and not be allowed to pond. If possible, all footing concrete should be placed the same day the excavation is made. If this is not possible, the footing excavations should be adequately protected.

4.10 SPPC Pile Foundation Recommendations (MBBR Building)

Should the estimated settlements associated with shallow foundation construction provided in Section 4.5 be considered unacceptable and the surcharge program described in Section 4.6 be deemed undesirable, the proposed structure’s foundations and slabs can be supported by a deep foundation system. The deep foundation system should be comprised of driven SPPC (square pre-cast pre-stressed concrete) piles. The following sections describe the pile capacity analyses and provide our recommendations for static axial compressive pile capacities, pile testing program, and pile construction criteria. In addition, we have provided estimates of potential settlement.

4.10.1 Axial Compression Capacity Recommendations

We conducted pile capacity analyses using static formulas with coefficients recommended by Geoffrey Myerhoff and George Sowers. The analyses include the contributions of shaft friction and end bearing to the pile capacity. The piles are expected to derive their capacity from a combination of shaft friction and end bearing in the deeper Sand layers at the depth presented in Table VI below.

The soil materials typically exhibit time-dependent strength characteristics; consequently shaft friction and end bearing support tend to increase from initial installation through a process termed “soil setup”. Essentially, the dynamics of driving piles will cause excess pore pressures to develop, thereby decreasing driving resistance during initial pile installation. The pile capacities developed during driving are usually much lower than the design values. Once driving is complete, the excess pore pressures dissipate with time (and soil setup occurs) and the bearing capacity of the pile increases. Based upon our experience with similar projects, 5 to 7 days is usually required for pore pressures to dissipate and soil setup to occur.

For the reasons previously described, it will not be possible to confirm pile capacities with a simple driving criteria such as number of hammer blows per foot of advanced pile. Instead, driving criteria will likely consist of a target tip elevation and/or certain embedded length in a bearing material with specified driving resistance. The specified driving resistance should be based on a Wave Equation Analysis of the contractor’s selected hammer.

Table VI provides our recommended pile type for the structure’s foundations and slabs. The allowable capacity for the piles includes a safety factor of at least 2.0 to allow for a pile load test program that relies primarily on dynamic testing. The capacity of a group of piles spaced at least 3 pile diameters apart, center to center, can be taken as the sum of the individual capacities with no reduction factor. If closer pile spacing is anticipated, the geotechnical engineer should be contacted to evaluate the efficiency of the specific pile group. The order lengths and tip elevations will be adjusted based on the results of the test piles and load test programs.

Table VI - SPPC Pile Recommendations

Pile Type	Embedment Depth[1] (ft.)	Allowable Compression Capacity (tons)	Pre-Augering Depth (ft)
12” SPPC	55-60	80-100	5-10

(1) Below the base of slab elevation of 21 feet MSL.
(2) Lateral capacity is based on one-half of the lateral load that produces a 1-inch lateral displacement (free-head condition). Batter piles would enhance lateral capacity.

It is noted that some piles could potentially encounter driving refusal at shallower depths (medium dense Sand between roughly 15 and -5 feet MSL). We recommend pre-augering the pile locations prior to driving to the depth (below grade) shown in the table above. This is necessary to help in minimizing the effects of vibrations from the driving effort on adjacent buildings and to reduce the potential for pile breakage. Following the pre-augering, the piles should be installed and advanced by driving with an impact hammer to their design tip elevations. If for some reason during construction, pile driving "capacity" is encountered before the piles reach their design tip elevations, the Geotechnical Engineer should be retained to review driving records and field reports to determine whether the pile can adequately support the design loads. If the pile driving hammer is not properly matched to the pile type, size and subsurface conditions, it may reach practical refusal before the pile reaches the design tip elevation, or the required capacity.

4.10.2 Pile Group Settlement

Based on the results of load tests performed on piles driven in similar soils conditions, it is anticipated that the total butt settlements (including elastic shortening) will not exceed about ½-inch, which is the settlement necessary to mobilize the soil/pile capacity in combination with the pile group settlements due to the stress increase in the underlying soils.

4.10.3 Test Piles

We recommend that a test pile program be implemented for the purpose of assisting in the development of tip elevations and to confirm that the contractor's equipment and installation methods are acceptable. The test program should involve at least ten (10) test piles to provide an indication of various driving and/or installation conditions. The test pile locations should be established by the Geotechnical Engineer based on the structural characteristics. It is important to note the relationship between the required testing and our design assumptions. We chose safety factors based upon the recommended pile testing program. We expect that the pile testing program will include primarily dynamic evaluation with a Pile Driving Analyzer (PDA).

The piles should be driven using the drive system submitted by the contractor and approved by the geotechnical engineer. Test pile lengths should be at least ten feet longer than anticipated production pile lengths (i.e. 65-70 feet) to ensure that the required capacity is developed, to allow for refinement of estimated capacities, and for dynamic and static testing reasons.

The indicator piles installed during the Test Pile Program, which satisfy the geotechnical engineer's requirements for proper installation, may also be used as permanent production piles.

The contractor should include in his equipment submittal a Wave Equation Analysis (using GRLWEAP™ software) modeling the behavior of the test piles during driving, or what is termed a "Drivability Study." The primary intent of the Wave Equation Analysis is to estimate the feasibility of the contractor's proposed pile driving system with respect to installing the piles. Since the results of the Wave Equation Analyses are dependent on the chosen hammer, the pile type and length, and the subsurface conditions, it is likely that at least one Wave Equation Analysis per hammer will be required.

Pile driving equipment should not be mobilized for the test piles until the Wave Equation Analyses have been submitted and approved by the geotechnical engineer. If the contractor's proposed pile driving system is rejected, subsequent submittals of alternative drive systems should also include appropriate Wave Equation Analyses that are subject to the approval of the geotechnical engineer. The Wave Equation Analyses are also used to estimate:

- Compressive and tensile stresses experienced by the modeled pile during driving
- The total number of blows required to install the pile
- Driving resistance (in terms of blows per foot) within the various soil strata the pile is embedded in
- Driving time

The results of the WEAP analyses are highly dependent on the many input parameters related to the soil conditions, static pile capacity estimates, as well as specific characteristics associated with different makes and models of pile driving hammers.

4.10.4 Dynamic Testing

Dynamic testing was developed as a method of improving upon the reliability of the wave equation and other dynamic predictions by physically measuring the acceleration and strain of a pile during driving. The use of dynamic pile testing has permitted the possibility of checking the driving stresses in the pile and the hammer performance during pile driving. It is also possible to estimate the static capacity of the pile based upon the strain and acceleration measurements taken during pile driving.

The test pile installation should be monitored by the Geotechnical Engineer using the PDA, an electronic device that records driving stresses and pile/soil interactions, among other things. The PDA results will confirm that the pile driving system (hammer type/energy, cushion type/ thickness, etc.) can successfully install the piles without over stressing them in compression or tension. It is essential the test pile re-strikes be monitored with the PDA.

No sooner than 7 days after installation, all of the test piles should be re-struck while being monitored with the PDA. This test establishes the "static capacity" of the pile. The initial hammer blow during re-strike activities is critical to the quality of dynamic data with respect to capacity interpretation. The contractor should make every effort to insure an initial high-energy blow of the hammer. After several blows during re-strike activities, pore pressures increase, soil setup diminishes, and ultimately pile capacities (as recorded by the PDA) decrease. Loss of estimated static capacity following repeated hammer blows is the reason the initial blows are critical.

The dynamic data recorded by the PDA during restrike testing should be further refined by using CAPWAP® analysis. CAPWAP® analysis, not the initial assessment of capacity determined by the PDA, should be the basis of static pile capacity estimates. Interpretation of CAPWAP® data, in the context of the soils subsurface conditions and previous static pile capacity estimates, should allow the Geotechnical Engineer to estimate ultimate pile capacities and recommend appropriate production pile lengths.

Our previous experience with the PDA indicates that a significant cost savings may be realized if the PDA is properly utilized to monitor the installation of test piles, confirm pile capacity in production installations, and monitor potentially damaging stresses during driving. The use of the PDA permits the confirmation of allowable compression and uplift capacities and pile integrity on several piles for a cost similar to or less than that of a single full-scale static load test. We recommended the owner retain the services of the Geotechnical Engineer to perform the dynamic testing, not the installation contractor, to avoid possible conflicts of interest.

4.10.5 Establishing Pile Driving Criteria

Prior to driving production piles, the geotechnical engineer should establish the criteria for pile installation. The criteria will be based on the data collected during monitoring of the test pile installation and the subsequent re-striking. The intent of establishing driving criteria is to facilitate installation of the production piles without damage and to provide a means of establishing when piles have achieved the design capacities. The driving criteria may include: hammer type, hammer energy, ram weight, pile cushion and thickness, hammer cushion type and thickness, required tip elevations and driving resistance necessary to achieve capacities, and possibly pre-drilling recommendations (if the test pile results warrant the need).

4.10.6 Allowable Driving Stresses

Guidelines from the Pre-stressed Concrete Institute (PCI), American Society of Civil Engineers (ASCE), and the Association of State Highway Transportation Officials (AASHTO) indicate that maximum compressive stresses, imposed on driven precast concrete piles during installation, should be less than the following equation: 0.85 x f'_c (concrete compressive strength, psi) - f_{pe} (effective pre-stressing after losses from relaxation). The three groups differ on the maximum tensile stresses. PCI recommends 6 x square root of f'_c + f_{pe} ; ASHTO and ASCE recommend 3 x square root f'_c + $f_{pe.}$ We recommend using the consensus value for the maximum compressive stress, and the ASCE/AASHTO recommended value for the maximum tensile stress.

4.10.7 Hammer Types and Energies

In comparing hammers of equal energy, the Prestressed Concrete Institute (PCI) states that hammers with heavier rams and lower impact velocities are less likely to cause damaging stresses in concrete piles. Hammers with proportionally higher ram weights and short stroke heights (low impact velocities) are usually air, steam and hydraulic driven, and not diesel fueled. It has been our experience that air, steam and hydraulic hammers are more appropriate for the installation of precast concrete piles than similarly sized (in terms of energy) diesel hammers. We recommend that the contractor use an air, steam or hydraulic driven hammer whose ram weight is roughly equal to 0.5 to 1.0 times the weight of the pile itself. The actual determination of an acceptable ram weight should be determined through the results of the Test Pile Program. If the contractor elects to use a diesel hammer, we recommend a critical, detailed review of the contractor's Wave Equation Analysis prior to driving the test piles.

4.10.8 Driven Pile Installation Monitoring

The geotechnical engineer should observe the installation of the test pile and all production piles. The purpose of the geotechnical engineer's observations is to determine if production installations are being performed in accordance with the previously derived Pile Driving Criteria. Continuous driving and installation records should be maintained for all driven piles. Production piles should be driven utilizing the approved system established as a result of the Test Program.

The field duties of the geotechnical engineer (or a qualified engineer's representative) should include the following:

- Being knowledgeable of the subsurface conditions at the site and the project-specific Pile Driving Criteria.

- Being aware of aspects of the installation including type of pile driving equipment and pile installation tolerances.

- Keeping an accurate record of pile installation and driving procedures.

- Documenting that the piles are installed to the proper depth indicative of the intended bearing stratum. Also documenting that appropriate pile splicing techniques are used, if necessary.

- Recording the number of hammer blows for each foot of driving.

- Generally confirming that the pile driving equipment is operating as anticipated.

- Record the energy rating of the hammer.

- Informing the geotechnical engineer of any unusual subsurface conditions or driving conditions.

- Notifying the owner and structural engineer when unanticipated difficulties or conditions are encountered.

- Confirming from visual appearance that the piles are not damaged during installation and observing the piles prior to installation for defective workmanship. The geotechnical engineer should review all driving records prior to pile cap construction.

4.10.9 Adjacent Structures

When considering the suitability of a driven pile foundation, consideration should be given to the integrity of nearby structures. Due to the large amount of energy required to install driven deep foundations, vibrations of considerable magnitude are generated. These vibrations may affect nearby structures. These structures can, due to their proximity, be detrimentally affected by the construction unless proper protection measures are taken. In addition, experience has shown that these construction features will often lead to the conclusion that damage to adjacent property has taken place, even though none has occurred. It is therefore recommended that a thorough survey of the adjacent property be made prior to starting construction. This will help to better evaluate real claims and refute groundless nuisance claims. The survey should include, but not be limited to, the following:

1. Visually inspect adjacent structures, noting and measuring all cracks and other signs of distress. Take photographs as needed.
2. Visually inspect adjacent pavements, noting and measuring any significant cracks, depressions, etc. Take photographs as needed.
3. Establish several benchmarks along foundation walls on adjacent structures. Both vertical and horizontal control should be employed.
4. Determine if equipment in any adjacent building is sensitive to vibration, and if so, establish proper control and monitoring system.

4.11 Embankment Fill and Placement

Following the clearing operation, the placement of the fill required to establish the design grades may begin. The embankments shall be constructed in accordance with Section 303 of the 2007 VDOT Road and Bridge Specifications. Fill material shall conform to the requirements of AASHTO Standard M57. As such, fill materials classified as A-1, A-2-4, A-2-5 or A-3 can be used for embankment fill. Accordingly, suitable embankment fill material may consist of sand or gravel containing less than 35% by weight of fines (SP, SM, SC, SW, GP, GW), having a liquid limit less than 40 and Plastic Index (PI) less than 10, and should be free of rubble, organics, clay, debris and other unsuitable material. Any material to be used for embankment fill should be evaluated and tested by **G E T Solutions, Inc.** prior to placement to determine if they are suitable for the intended use.

All embankment fill should be tested in accordance with Section 303 of the 2007 VDOT Road and Bridge Specifications and Section 314.01 of the VDOT Materials Division MOI, as stipulated in the Special Provisions for “Density Control of Embankments and Backfill”.

The moisture content of the select fill should be within +/- 2% of the optimum moisture content at the time of placement. In general, the compaction should be accomplished by placing the fill in maximum 8-inch loose lifts and mechanically compacting each lift to a minimum of 95% of the Standard Proctor maximum dry density. A qualified inspector should perform field density tests in accordance with Section 314.01 of the VDOT Materials Division MOI to assure that adequate compaction is achieved. Care should be used when operating the compactors near existing structures to avoid transmission of the vibrations that could cause settlement damage or disturb occupants. In this regard, it is recommended that the vibratory roller remain at least 25 feet away from existing structures (to include the existing water lines located adjacent to the west abutment); these areas should be compacted with small, hand-operated compaction equipment.

4.12 Suitability of On-site Soils for Reuse as Embankment Fill

Based on the boring information it appears that the shallow subsurface SAND (SC, SM) soils located in the cut areas will be suitable for reuse. It is noted that these suitable materials are typically located beneath a CLAY (CL, CH) and SILT (ML) cap with varying thicknesses. As such, care should be taken during excavation in order to segregate the suitable soils from the unsuitable soils (soils containing more than 35% by weight of fines) to prevent contamination. Soils located below the water table will be wet at the time of excavation and will need to dry prior to placement. Typical drying methods include stockpiling or placing the wet materials in thin lifts prior to compaction. The goal of these methods is to dry the soil to within +/- 2% of their optimum moisture at the time of compaction.

4.13 Embankment Stability Analysis

The stability analyses for the sloped embankments were conducted using the GeoStudio software program SLOPE/W™ for limit-equilibrium modeling and analysis of Slope Stability. Circular and directional block potential failure surfaces were evaluated using numerous methods of analysis including Bishop, Spencer and Morgenstern-Price. The soil parameters used were estimated from the SPT boring data, the results of laboratory classification and strength testing, In-Situ Testing (CPT/DMT), and from our experience with typical materials for other projects. Stability analyses were performed on the embankment slopes at 3H:1V. Our slope stability analyses indicated that the sloped embankments will maintain a suitable performance when constructed at 3H:1V or flatter. A summary of the slope stability analyses is tabulated below (Table VII). The comprehensive slope stability analysis reports are provided in Appendix VIII.

Table VII - Summary of Slope Stability Analyses

Maximum Fill Height (ft)	Slope Dimensions (H:V)	Minimum Morgenstern-Price Factor of Safety
10-15	3:1	1.91

4.14 Below Grade Walls

It is expected that any below grade walls associated with the new construction will consist of reinforced concrete walls. It is also expected that the retaining structures will be designed as "at rest" members (no movement). The footings for these structures can be designed using a net allowable soil pressure of 2,000 psf and maximum toe pressure of 2,500 psf. Furthermore, the resultant of the soil pressure distribution across the width of the footing should pass through the center third of the footing cross section.

In order to reduce the magnitude of lateral loads being applied to the walls and to promote positive water drainage, it is recommended that a granular backfill be placed directly behind the walls and extend laterally back from the walls a minimum distance of four feet. These granular soils should be a relatively clean, free draining granular material (SAND) classified as SP-SM or better, containing less than 12% passing the No. 200 sieve (0.074 mm). Filter fabric should be installed between the drainage material and the existing site soils to prevent fines from contaminating the drainage material. A "sock drain" may also be necessary to maintain proper drainage behind the wall. The compaction behind these walls should be in the range of 95% to 97% of the Standard Proctor maximum dry density (ASTM Specification D 698). The soils in this zone should not be over-compacted. In order to minimize the potential for wall damage due to excessive compaction, hand operated mechanical tampers should be used to compact the granular materials. Heavy compaction equipment should not be allowed within five feet of the walls.

With regard to the design of the walls to resist lateral earth pressures, the estimated soil parameters provided on the following page (Table VIII) can be used.

4.15 Floor Slab Design (Appurtenant Structures)

The floor slab may be constructed as a slab-on-grade member provided the previously recommended earthwork activities and evaluations are carried out properly. It is recommended that the ground floor slab be directly supported by at least a 4-inch layer of relatively clean, compacted, poorly graded sand (SP) or gravel (GP) with less than 5% passing the No. 200 Sieve (0.074 mm). The purpose of the 4-inch layer is to act as a capillary barrier and equalize moisture conditions beneath the slab.

It is recommended that all ground floor slabs be "floating". That is, generally ground supported and not rigidly connected to walls or foundations. This is to minimize the possibility of cracking and displacement of the floor slabs because of differential movements between the slab and the foundation. It is also recommended that the floor slab bearing soils be covered by a vapor barrier or retarder in order to minimize the potential for floor dampness, which can affect the performance of glued tile and carpet. Generally, use a vapor retarder for minimal vapor resistance protection below the slab on grade. When floor finishes, site conditions or other considerations require greater vapor resistance protection; consideration should be given to using a vapor barrier. Selection of a vapor retarder or barrier should be made by the Architect based on project requirements.

Table VIII - Below Grade Soil Parameters (Borings M-1 through M-3)

Soil Type	Structural Fill (SM, SP-SM, SP)	CLAY (CL, CH)	SAND (SM, SP, SP-SM)	CLAY (CL, CH)	SAND (SM, SP, SP-SM)
Average SPT N-Value	-	18	17	7	19
Depth (feet)	-	0 to 7.5-8	7.5-8 to 18-28	18-28 to 24-33	24-33 to 30-50
Elevation (feet MSL)	-	15 to 20 - 7.5 to 12	7.5 to 12 - -3 to -8	-3 to -8 - -9 to -13	-9 to -13 - -15 to -30
Total Moist Unit Weight (pcf)	115	115	125	105	115
Total Saturated Unit Weight (pcf)	125	125	135	117	135
Total Buoyant Unit Weight (pcf)	63	63	73	55	73
Friction Angle (ϕ) degrees	34	0	38	0	36
Cohesion (c) pcf	0	2500	0	1000	0
Active Soil Pressure, Ka	0.28	0.84	0.24	0.82	0.26
At-Rest Soil Pressure, Ko	0.44	1.1	0.38	1.0	0.41
Passive Soil Pressure, Kp	3.5	1.2	4.2	1.2	3.8
Seismic Active Lateral Earth Pressure* Kae (uniform)	0.43	0.99	0.39	0.97	0.41
Seismic Passive Lateral Earth Pressure* Kae (uniform)	3.925	1.625	4.625	1.625	4.225
Modulus of Subgrade Reaction (pci)(1)	130	105	-	-	-

**Estimated Values - Values will vary depending upon design acceleration events and type of construction.*
(1) – Shallow influence, small-strain behavior

4.16 Seismic Evaluation

On the basis of the results of the soil test borings drilled at this site (65 feet maximum explored depth) and our experience in the project area, it is our opinion that this site should be considered a Site Class “D” in accordance with Table 20.3-1 Site Classification of the ASCE 7-10 Minimum Design Loads for Buildings and Other Structures, Chapter 20 (referenced in the 2012 IBC). However, the seismic evaluation requires soils information associated with the upper 100 feet. If the site classification is critical to the structural design it will be necessary to perform a 100-foot deep CPT boring with shear wave velocity testing to substantiate the site classification.

5.0 CONSTRUCTION CONSIDERATIONS

5.1 Drainage and Groundwater Concerns

It is expected that dewatering may be required for excavations that extend near or below the existing groundwater table. Dewatering above the groundwater level could probably be accomplished by pumping from sumps. Dewatering at depths below the groundwater level may require well pointing.

It would be advantageous to construct all fills early in the construction. If this is not accomplished, disturbance of the existing site drainage could result in collection of surface water in some areas, thus rendering these areas wet and very loose. Temporary drainage ditches should be employed by the contractor to accentuate drainage during construction.

5.2 Site Utility Installation

The base of the utility trenches should be observed by a qualified inspector prior to the pipe and structure placement to verify the suitability of the bearing soils. Based on the results of our field exploration program it is expected that the utilities and structures located at depths greater than 2 to 13 feet MSL will bear in the wet, loose/soft granular/cohesive soils. In these instances the bearing soils will likely require some stabilization to provide suitable bedding. This stabilization is typically accomplished by providing additional VDOT No. 57 stone bedding (typically 12 to 24 inches). In addition, depending on the depth of the utility trench excavation, some means of dewatering may be required to facilitate the utility installation and associated backfilling.

The resulting excavations should be backfilled with structural fill, as described in Section 4.3 of this report. The subsurface soils encountered at the boring locations did not appear to meet the criteria recommended in this report for reuse as structural fill. As such, imported fill may be required to backfill utility excavations within the building areas.

5.3 Excavations

In Federal Register, Volume 54, No. 209 (October, 1989), the United States Department of Labor, Occupational Safety and Health Administration (OSHA) amended its “Construction Standards for Excavations, 29 CFR, part 1926, Subpart P”. This document was issued to better insure the safety of workmen entering trenches or excavations. It is mandated by this federal regulation that all excavations, whether they be utility trenches, basement excavation or footing excavations, be constructed in accordance with the new (OSHA) guidelines. It is our understanding that these regulations are being strictly enforced and if they are not closely followed, the owner and the contractor could be liable for substantial penalties.

The contractor is solely responsible for designing and constructing stable, temporary excavations and should shore, slope, or bench the sides of the excavations as required to maintain stability of both the excavation sides and bottom. The contractor’s responsible person, as defined in 29 CFR Part 1926, should evaluate the soil exposed in the excavations as part of the contractor's safety procedures. In no case should slope height, slope inclination, or excavation depth, including utility trench excavation depth, exceed those specified in local, state, and federal safety regulations.

We are providing this information solely as a service to our client. **G E T Solutions, Inc.** is not assuming responsibility for construction site safety or the contractor's activities; such responsibility is not being implied and should not be inferred.

6.0 REPORT LIMITATIONS

The recommendations submitted are based on the available soil information obtained by **G E T Solutions, Inc.** and the information supplied by the client and their consultants for the proposed project. If there are any revisions to the plans for this project or if deviations from the subsurface conditions noted in this report are encountered during construction, **G E T Solutions, Inc.** should be notified immediately to determine if changes in the foundation recommendations are required. If **G E T Solutions, Inc.** is not retained to perform these functions, **G E T Solutions, Inc.** can not be responsible for the impact of those conditions on the geotechnical recommendations for the project.

The Geotechnical Engineer warrants that the findings, recommendations, specifications or professional advice contained herein have been made in accordance with generally accepted professional geotechnical engineering practices in the local area. No other warranties are implied or expressed.

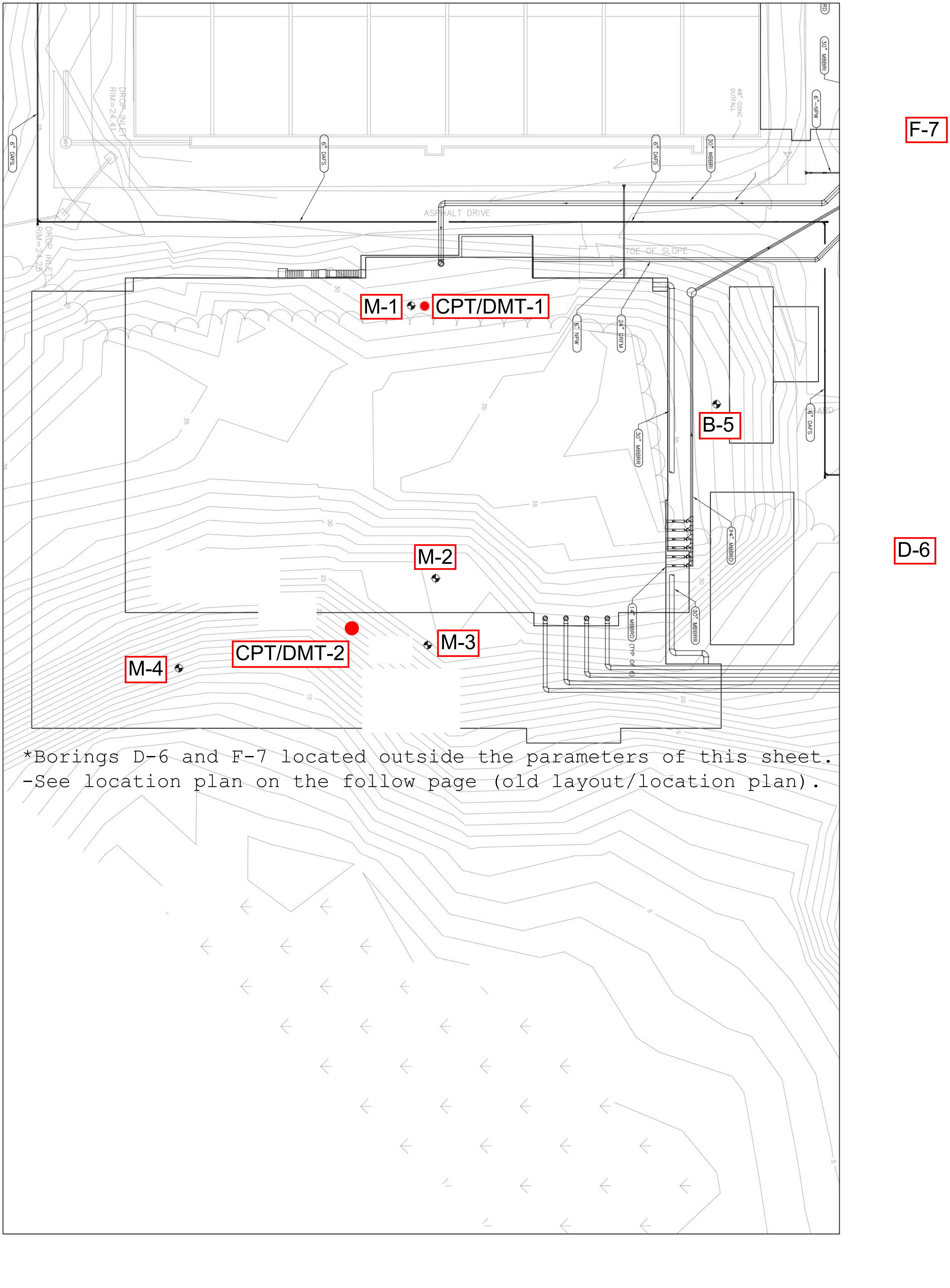
After the plans and specifications are more complete the Geotechnical Engineer should be provided the opportunity to review the final design plans and specifications to assure our engineering recommendations have been properly incorporated into the design documents, in order that the earthwork and foundation recommendations may be properly interpreted and implemented. At that time, it may be necessary to submit supplementary recommendations. This report has been prepared for the exclusive use of HDR Engineering, Inc. and their consultants for the specific application to the proposed Hopewell Regional Alternative 4A-1 Light Phase 2 PER project in Hopewell, Virginia.

APPENDICES

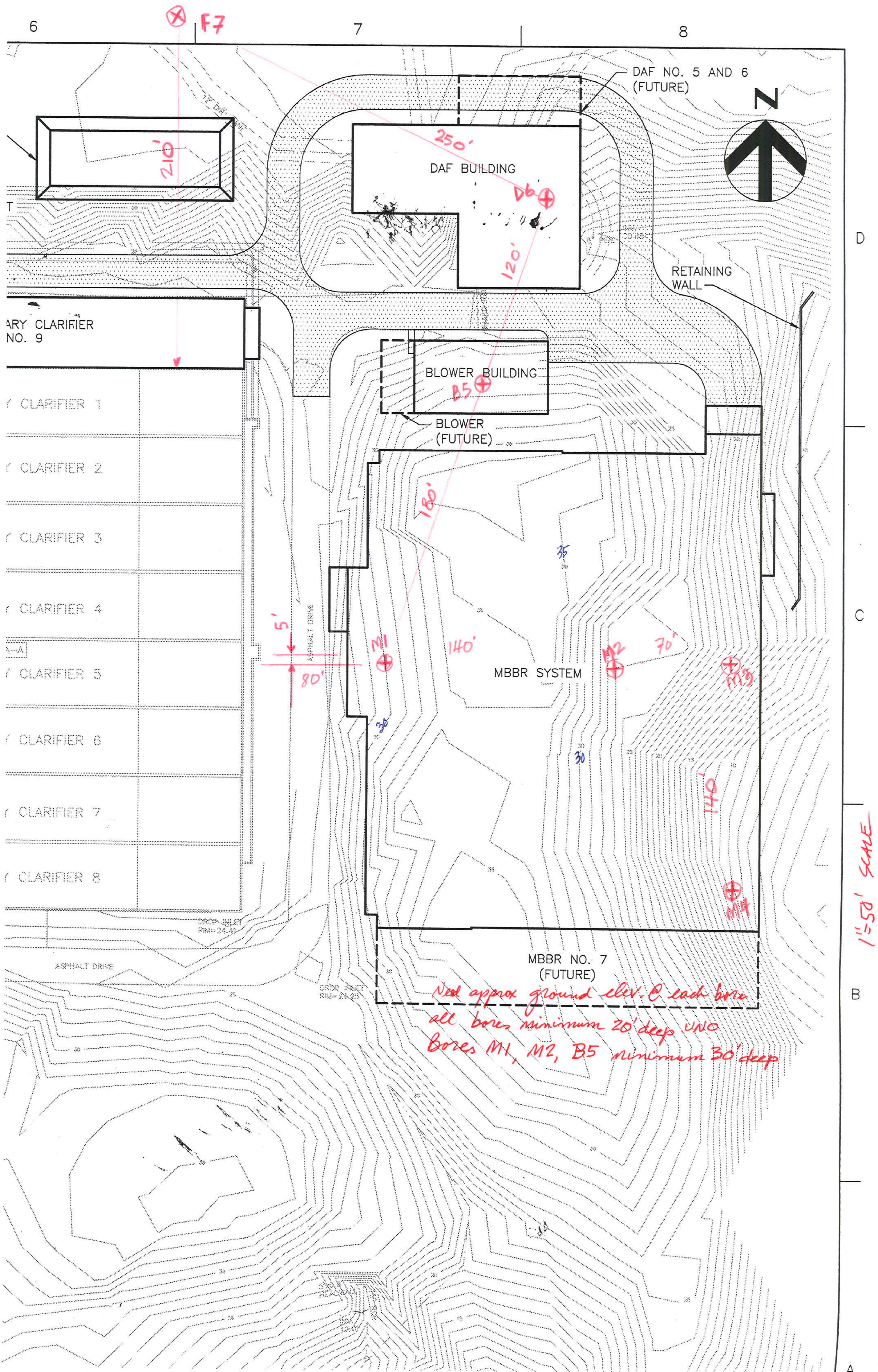
- **I** BORING LOCATION PLAN
- **II** SUMMARY OF SOIL CLASSIFICATION
- **III** BORING LOGS
- **IV** GENERALIZED SOIL PROFILE
- **V** CONSOLIDATION TEST RESULTS
- **VI** TRIAXIAL COMPRESSION TEST RESULTS
- **VII** STRESS-DEFORMATION FINITE ELEMENT ANALYSIS RESULTS
- **VIII** SLOPE STABILITY ANALYSIS RESULTS
- **IX** CPT/DMT TEST RESULTS

APPENDIX I

BORING LOCATION PLAN



*Borings D-6 and F-7 located outside the parameters of this sheet.
-See location plan on the follow page (old layout/location plan).



F7
210'
250'
120'
180'
140'
70'
140'
80'
5'
D6
B5
M1
M2
M3
M4
DAF NO. 5 AND 6
(FUTURE)
DAF BUILDING
RETAINING
WALL
ARY CLARIFIER
NO. 9
BLOWER BUILDING
BLOWER
(FUTURE)
CLARIFIER 1
CLARIFIER 2
CLARIFIER 3
CLARIFIER 4
CLARIFIER 5
CLARIFIER 6
CLARIFIER 7
CLARIFIER 8
ASPHALT DRIVE
MBBR SYSTEM
MBBR NO. 7
(FUTURE)
DROP INLET
Need approx ground elev. @ each bore
all bores minimum 20' deep UNO
Bores M1, M2, B5 minimum 30' deep
1"=50' SCALE

APPENDIX II

SUMMARY OF SOIL CLASSIFICATION

Virginia Beach
204 Grayson Road
Virginia Beach, VA 23462
(757) 518-1703

Williamsburg
1592 Penniman Rd. Suite E
Williamsburg, Virginia 23185
(757) 564-6452

Elizabeth City
504 East Elizabeth St. Suite 2
Elizabeth City, NC 27909
(252) 335-9765

CLASSIFICATION SYSTEM FOR SOIL EXPLORATION

Standard Penetration Test (SPT), N-value

Standard Penetration Tests (SPT) were performed in the field in general accordance with ASTM D 1586. The soil samples were obtained with a standard 1.4" I.D., 2" O.D., 30" long split-spoon sampler. The sampler was driven with blows of a 140 lb. hammer falling 30 inches. The number of blows required to drive the sampler each 6-inch increment (4 increments for each soil sample) of penetration was recorded and is shown on the boring logs. The sum of the second and third penetration increments is termed the SPT N-value.

NON COHESIVE SOILS

(SILT, SAND, GRAVEL and Combinations)

Relative Density

Very Loose	4 blows/ft. or less
Loose	5 to 10 blows/ft.
Medium Dense	11 to 30 blows/ft.
Dense	31 to 50 blows/ft.
Very Dense	51 blows/ft. or more

Particle Size Identification

Boulders	8 inch diameter or more	
Cobbles	3 to 8 inch diameter	
Gravel	Coarse	1 to 3 inch diameter
	Medium	$^{1}/_{2}$ to 1 inch diameter
	Fine	$^{1}/_{4}$ to $^{1}/_{2}$ inch diameter
Sand	Coarse	2.00 mm to $^{1}/_{4}$ inch (diameter of pencil lead)
	Medium	0.42 to 2.00 mm (diameter of broom straw)
	Fine	0.074 to 0.42 mm (diameter of human hair)
Silt		0.002 to 0.074 mm (cannot see particles)

COHESIVE SOILS

(CLAY, SILT and Combinations)

Consistency

Very Soft	2 blows/ft. or less
Soft	3 to 4 blows/ft.
Medium Stiff	5 to 8 blows/ft.
Stiff	9 to 15 blows/ft.
Very Stiff	16 to 30 blows/ft.
Hard	31 blows/ft. or more

Relative Proportions

Descriptive Term	Percent
Trace	0-5
Few	5-10
Little	15-25
Some	30-45
Mostly	50-100

Strata Changes

In the column "Description" on the boring log, the horizontal lines represent approximate strata changes.

Groundwater Readings

Groundwater conditions will vary with environmental variations and seasonal conditions, such as the frequency and magnitude of rainfall patterns, as well as tidal influences and man-made influences, such as existing swales, drainage ponds, underdrains and areas of covered soil (paved parking lots, side walks, etc.).

CLASSIFICATION SYMBOLS (ASTM D 2487 and D 2488)

Coarse Grained Soils

More than 50% retained on No. 200 sieve

GW - Well-graded Gravel
GP - Poorly graded Gravel
GW-GM - Well-graded Gravel w/Silt
GW-GC - Well-graded Gravel w/Clay
GP-GM - Poorly graded Gravel w/Silt
GP-GC - Poorly graded Gravel w/Clay
GM - Silty Gravel
GC - Clayey Gravel
GC-GM - Silty, Clayey Gravel
SW - Well-graded Sand
SP - Poorly graded Sand
SW-SM - Well-graded Sand w/Silt
SW-SC - Well-graded Sand w/Clay
SP-SM - Poorly graded Sand w/Silt
SP-SC - Poorly graded Sand w/Clay
SM - Silty Sand
SC - Clayey Sand
SC-SM - Silty, Clayey Sand

Fine-Grained Soils

50% or more passes the No. 200 sieve

CL - Lean Clay
CL-ML - Silty Clay
ML - Silt
OL - Organic Clay/Silt

Liquid Limit 50% or greater

CH - Fat Clay
MH - Elastic Silt
OH - Organic Clay/Silt

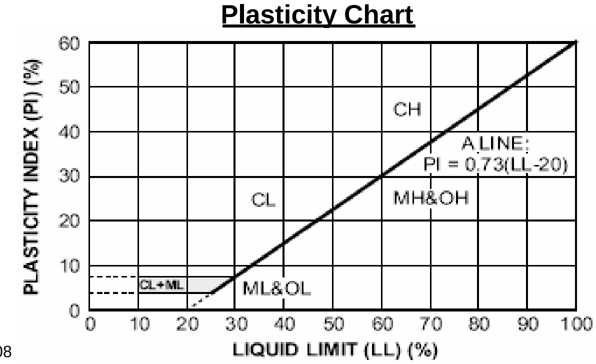
Highly Organic Soils

PT - Peat

Depending on percentage of fines (fraction smaller than No. 200 sieve size), coarse-grained soils are classified as follows:

Less than 5 percent	GW, GP, SW,SP
More than 12 percent	GM, GC, SM, SC
5 to 12 percent	Borderline cases requiring dual symbols

Plasticity Chart



GET Revision 9/25/2008

APPENDIX III

BORING LOGS

GET Solutions, Inc.
Geotechnical • Environmental • Testing

BORING LOG B-5

PROJECT: Hopewell Regional Alternative 4A-1 Light Phase 2 PER
CLIENT: HDR Engineering, Inc.
PROJECT LOCATION: Hopewell, Virginia — **PROJECT NO.:** WM13-136G
BORING LOCATION: See attached boring location plan — **SURFACE ELEVATION:** 31'
DRILLER: GET Solutions, Inc. — **LOGGED BY:** J. Robinson, P.E.
DRILLING METHOD: Rotary wash "mud" — **DATE:** 7-29-13
DEPTH TO WATER - INITIAL*: 18' **AFTER 24 HOURS:** — **CAVING>**

Depth (feet)	Description
0 – 0.08	1" Topsoil
0.08 – 2	Mottled Orange to Gray, Moist, fat CLAY (CH) with trace fine Sand and trace fibrous organics, Medium Stiff
2 – 6	Mottled Orange to Gray, Moist, fine Sandy lean CLAY (CL), Very Stiff to Hard
6 – 10	Mottled Orange to Gray, Moist, fine Sandy SILT (ML), Very Stiff to Hard
10 – 14	Mottled Orange to Gray, Moist, Silty fine SAND (SM), Loose to Medium Dense
14 – 24.5	Mottled Orange to Gray, Moist, fat CLAY (CH) with trace fine Sand, Stiff to Very Stiff
24.5 – 33	Mottled Orange to Gray, Moist, Silty fine to medium SAND (SM), Medium Dense
33 –	Mottled Orange to Gray, Wet, poorly graded fine to coarse SAND (SP-SM) with Silt, Medium Dense

Sample No.	Sample Recovery	Sample Type	Blows Per 6"	N Value	% < #200
1	12	SS	2-3-5-6	8	
2	18	SS	6-11-14-20	25	
3	20	SS	18-16-18-23	34	
4	18	SS	14-17-19-23	36	
5	20	SS	6-7-9-11	16	
6	18	SS	4-6-6-8	12	
7	18	SS	4-4-5-8	9	
8	24	SS	4-7-10-13	17	95
9	24	SS	5-5-8-10	13	
10	12	SS	4-6-10-8	16	
11	12	SS	4-5-6-5	11	

TEST RESULTS: Plastic Limit ├─┤ Liquid Limit; Moisture Content - ●; N-Value - (hatched bar); scale 10 20 30 40 50 60 70

Notes: *It is noted that the topsoil thickness noted above is not expected to be indicative of the thicknesses that will be encountered across the site as the boring locations were cleared for accessibility prior to drilling.

The initial groundwater reading may not be indicative of the static groundwater level.

SS = Split Spoon Sample
ST = Shelby Tube Sample
HA = Hand Auger Sample
BS = Bulk Sample
WOH = Weight of Hammer

Standard Penetration Tests were performed in the field in general accordance with ASTM D 1586.

BORING LOG B-5

PROJECT: Hopewell Regional Alternative 4A-1 Light Phase 2 PER	
CLIENT: HDR Engineering, Inc.	
PROJECT LOCATION: Hopewell, Virginia	**PROJECT NO.:** WM13-136G
BORING LOCATION: See attached boring location plan	**SURFACE ELEVATION:** 31'
DRILLER: GET Solutions, Inc.	**LOGGED BY:** J. Robinson, P.E.
DRILLING METHOD: Rotary wash "mud"	**DATE:** 7-29-13
DEPTH TO WATER - INITIAL*: 18' **AFTER 24 HOURS:**	**CAVING>**

Depth (feet)	Description	Sample No.	Sample Recovery	Sample Type	Blows Per 6"	N Value	% < #200	Test Results
38–43	Mottled Orange to Gray, Wet, fat CLAY (CH) with trace fine Sand, Very Soft	12	24	SS	WOH WOH WOH 2	0	99	
43–50	Mottled Orange to Gray, Wet, poorly graded fine to coarse SAND (SP-SM) with Silt and trace marine shell fragments, Loose to Medium Dense	13	18	SS	8 5 4 6	9		
		14	18	SS	9 9 9 12	18		
50	Boring terminated at 50 ft.							

TEST RESULTS: Plastic Limit ├┤ Liquid Limit; Moisture Content - ●; N-Value - (hatched bar); scale 10 20 30 40 50 60 70

Elevation (MSL) (ft): -10, -15, -20, -25, -30, -35, -40
Depth (meters): 12, 14, 16, 18, 20, 22
Depth (feet): 40, 45, 50, 55, 60, 65, 70, 75

Notes: *It is noted that the topsoil thickness noted above is not expected to be indicative of the thicknesses that will be encountered across the site as the boring locations were cleared for accessibility prior to drilling.

The initial groundwater reading may not be indicative of the static groundwater level.

SS = Split Spoon Sample
ST = Shelby Tube Sample
HA = Hand Auger Sample
BS = Bulk Sample
WOH = Weight of Hammer

Standard Penetration Tests were performed in the field in general accordance with ASTM D 1586.

BORING LOG D-6

PROJECT: Hopewell Regional Alternative 4A-1 Light Phase 2 PER
CLIENT: HDR Engineering, Inc.
PROJECT LOCATION: Hopewell, Virginia | **PROJECT NO.:** WM13-136G
BORING LOCATION: See attached boring location plan | **SURFACE ELEVATION:** 35'
DRILLER: GET Solutions, Inc. | **LOGGED BY:** J. Robinson, P.E.
DRILLING METHOD: Rotary wash "mud" | **DATE:** 7-29-13
DEPTH TO WATER - INITIAL*: 23' **AFTER 24 HOURS:** | **CAVING>**

Elevation (MSL) (ft)	Depth (feet)	Description	Sample No.	Sample Recovery	Sample Type	Blows Per 6"	N Value	% < #200
35	0	3" Topsoil						
	0.25	Mottled Orange to Gray, Moist, lean CLAY (CL) with trace to little fine Sand, Medium Stiff to Stiff	1	18	SS	3 4 3 3	7	
			2	20	SS	4 5 6 6	11	
	4	Mottled Orange to Gray, Moist, Clayey fine to medium SAND (SC), Medium Dense	3	20	SS	7 7 8 7	15	
			4	24	SS	8 9 11 10	20	
	8	Mottled Orange to Gray, Moist, fat CLAY (CH) with trace fine Sand, Very Stiff	5	24	SS	3 7 11 14	18	
			6	24	SS	4 6 11 14	17	98
	14	Mottled Orange to Gray, Moist, Silty fine SAND (SM), Medium Dense	7	24	SS	6 9 11 14	20	
			8	24	SS	5 5 8 10	13	
	23	Mottled Orange to Gray, Wet, poorly graded fine to coarse SAND (SP-SM) with Silt, Medium Dense	9	12	SS	6 5 6 6	11	
10	25	Boring terminated at 25 ft.						

TEST RESULTS
Plastic Limit ⊢⊣ Liquid Limit
Moisture Content - ●
N-Value - (hatched bar)
10 20 30 40 50 60 70

Notes: *It is noted that the topsoil thickness noted above is not expected to be indicative of the thicknesses that will be encountered across the site as the boring locations were cleared for accessibility prior to drilling.

The initial groundwater reading may not be indicative of the static groundwater level.

SS = Split Spoon Sample
ST = Shelby Tube Sample
HA = Hand Auger Sample
BS = Bulk Sample
WOH = Weight of Hammer

Standard Penetration Tests were performed in the field in general accordance with ASTM D 1586.

BORING LOG F-7

PROJECT: Hopewell Regional Alternative 4A-1 Light Phase 2 PER

CLIENT: HDR Engineering, Inc.

PROJECT LOCATION: Hopewell, Virginia — **PROJECT NO.:** WM13-136G

BORING LOCATION: See attached boring location plan — **SURFACE ELEVATION:** 31'

DRILLER: GET Solutions, Inc. — **LOGGED BY:** J. Robinson, P.E.

DRILLING METHOD: Rotary wash "mud" — **DATE:** 7-29-13

DEPTH TO WATER - INITIAL*: N/A **AFTER 24 HOURS:** **CAVING>**

Depth (feet)	Description
0 – 0.08	1" Topsoil
0.08 – 2	Mottled Orange to Gray, Moist, lean CLAY (CL) with trace fine Sand, Medium Stiff
2 – 10	Mottled Orange to Gray, Moist, fat CLAY (CH) with trace fine Sand, Stiff to Hard
10 – 13	Mottled Orange to Gray, Moist, poorly graded fine SAND (SP-SM) with Silt, Loose
13 – 23	Mottled Orange to Gray, Moist, Silty fine to medium SAND (SM), Medium Dense
23 – 25	Mottled Orange to Gray, Moist, fat CLAY (CH) with trace fine Sand, Stiff
25	Boring terminated at 25 ft.

Sample No.	Sample Recovery	Sample Type	Blows Per 6"	N Value	% < #200
1	18	SS	2-3-4-5	7	
2	12	SS	6-7-8-10	15	
3	15	SS	10-12-15-15	27	
4	12	SS	13-16-17-20	33	
5	24	SS	5-8-11-13	19	
6	24	SS	3-4-6-7	10	7
7	24	SS	4-6-7-10	13	
8	18	SS	6-8-8-9	16	
9	24	SS	5-6-8-10	14	

TEST RESULTS: Plastic Limit — Liquid Limit; Moisture Content - ●; N-Value - (hatched bar); scale 10 20 30 40 50 60 70

Notes: *It is noted that the topsoil thickness noted above is not expected to be indicative of the thicknesses that will be encountered across the site as the boring locations were cleared for accessibility prior to drilling.

The initial groundwater reading may not be indicative of the static groundwater level.

SS = Split Spoon Sample
ST = Shelby Tube Sample
HA = Hand Auger Sample
BS = Bulk Sample
WOH = Weight of Hammer

Standard Penetration Tests were performed in the field in general accordance with ASTM D 1586.

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BORING LOG M-1

PROJECT: Hopewell Regional Alternative 4A-1 Light Phase 2 PER
CLIENT: HDR Engineering, Inc.
PROJECT LOCATION: Hopewell, Virginia
PROJECT NO.: WM13-136G
BORING LOCATION: See attached boring location plan
SURFACE ELEVATION: 30'
DRILLER: GET Solutions, Inc.
LOGGED BY: J. Robinson, P.E.
DRILLING METHOD: Rotary wash "mud"
DATE: 7-28-13
DEPTH TO WATER - INITIAL*: 23' **AFTER 24 HOURS:** **CAVING>**

Depth (feet)	Description
0 – 0.25	3" Topsoil
0.25 – 3.5	Mottled Orange to Gray, Moist, lean CLAY (CL) with trace fine Sand and trace fibrous organics, Stiff to Very Stiff
3.5 – 6	Mottled Orange to Gray, Moist, fat CLAY (CH) with trace fine Sand, Hard
6 – 10	Mottled Orange to Gray, Moist, fine Sandy SILT (ML), Very Stiff to Hard
10 – 13	Mottled Orange to Gray, Moist, Silty fine SAND (SM) with Clay, Medium Dense
13 – 28	Mottled Orange to Gray, Moist, lean CLAY (CL) with trace to little fine Sand, Very Stiff
28 – 33	Mottled Orange to Gray, Moist, Silty fine to medium SAND (SM), Medium Dense
33 –	Mottled Orange to Gray, Wet, poorly graded fine to medium SAND (SP-SM) with Silt, Medium Dense

Sample No.	Sample Recovery	Sample Type	Blows Per 6"	N Value	% < #200
1	11	SS	3-4-5-6	9	
2	14	SS	8-10-12-15	22	
3	17	SS	13-16-18-20	34	
4	21	SS	17-23-23-22	46	
5	16	SS	8-8-13-15	21	
6	17	SS	6-9-11-12	20	
7	16	SS	6-9-13-15	22	
8	20	SS	6-11-14-14	25	
9	24	SS	5-7-10-11	17	89
10	14	SS	7-8-9-4	17	
11	17	SS	6-9-8-9	17	8

TEST RESULTS: Plastic Limit ⊢⊣ Liquid Limit; Moisture Content - ●; N-Value - (hatched bar); scale 10 20 30 40 50 60 70

Notes: *It is noted that the topsoil thickness noted above is not expected to be indicative of the thicknesses that will be encountered across the site as the boring locations were cleared for accessibility prior to drilling.

SS = Split Spoon Sample
ST = Shelby Tube Sample
HA = Hand Auger Sample
BS = Bulk Sample
WOH = Weight of Hammer

*The initial groundwater reading may not be indicative of the static groundwater level.

Standard Penetration Tests were performed in the field in general accordance with ASTM D 1586.

GET Solutions, Inc.
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BORING LOG M-1

PROJECT: Hopewell Regional Alternative 4A-1 Light Phase 2 PER
CLIENT: HDR Engineering, Inc.
PROJECT LOCATION: Hopewell, Virginia | **PROJECT NO.:** WM13-136G
BORING LOCATION: See attached boring location plan | **SURFACE ELEVATION:** 30'
DRILLER: GET Solutions, Inc. | **LOGGED BY:** J. Robinson, P.E.
DRILLING METHOD: Rotary wash "mud" | **DATE:** 7-28-13
DEPTH TO WATER - INITIAL*: 23' **AFTER 24 HOURS:** | **CAVING>**

Elevation (MSL) (ft)	Depth (feet)	Description	Sample No.	Sample Recovery	Sample Type	Blows Per 6"	N Value	% < #200
	38	Mottled Orange to Gray, Wet, fine Sandy lean CLAY (CL), Very Soft to Soft	12	24	SS	WOH WOH WOH 1	0	
			13	24	SS	WOH WOH 3 3	3	60
	48	Mottled Orange to Gray, Wet, Silty fine to SAND (SM), Medium Dense	14	20	SS	3 4 9 10	13	
	53	Mottled Orange to Gray, Wet, fat CLAY (CH) with trace fine Sand, Medium Stiff	15	24	SS	4 3 4 3	7	
	58	Mottled Orange to Gray, Wet, poorly graded fine to coarse SAND (SP-SM) with Silt, Loose	16	9	SS	3 5 5 5	10	
	63	Greenish Gray, Wet, Silty fine to medium SAND (SM) with trace Clay, trace marine shell fragments and cemented Sand, Dense	17	24	SS	19 23 20 14	43	
	65	Boring terminated at 65 ft.						

Scale: Elevation -10, -15, -20, -25, -30, -35, -40, -45; Depth (meters) 12, 14, 16, 18, 20, 22; Depth (feet) 40, 45, 50, 55, 60, 65, 70, 75.

TEST RESULTS: Plastic Limit ├─┤ Liquid Limit; Moisture Content - ●; N-Value - (hatched bar); 10 20 30 40 50 60 70

Notes: *It is noted that the topsoil thickness noted above is not expected to be indicative of the thicknesses that will be encountered across the site as the boring locations were cleared for accessibility prior to drilling.

The initial groundwater reading may not be indicative of the static groundwater level.

SS = Split Spoon Sample
ST = Shelby Tube Sample
HA = Hand Auger Sample
BS = Bulk Sample
WOH = Weight of Hammer

Standard Penetration Tests were performed in the field in general accordance with ASTM D 1586.

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BORING LOG M-2

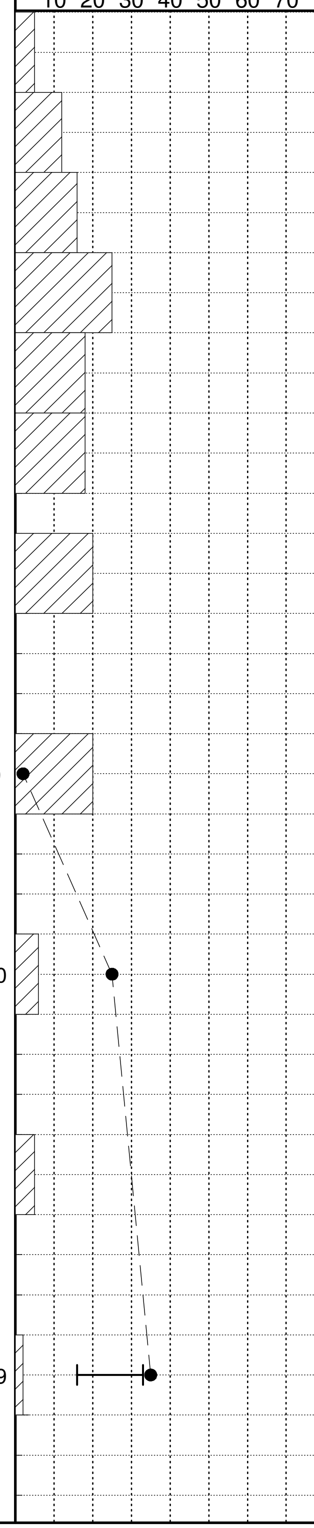
PROJECT: Hopewell Regional Alternative 4A-1 Light Phase 2 PER
CLIENT: HDR Engineering, Inc.
PROJECT LOCATION: Hopewell, Virginia — **PROJECT NO.:** WM13-136G
BORING LOCATION: See attached boring location plan — **SURFACE ELEVATION:** 27'
DRILLER: GET Solutions, Inc. — **LOGGED BY:** J. Robinson, P.E.
DRILLING METHOD: Rotary wash "mud" — **DATE:** 7-28-13
DEPTH TO WATER - INITIAL*: 23' **AFTER 24 HOURS:** **CAVING>**

Depth to bottom (feet)	Description
0.25	3" Topsoil
3	Mottled Orange to Gray, Moist, lean CLAY (CL) with trace fine Sand, Medium Stiff to Stiff
5	Mottled Orange to Gray, Moist, Silty fine to medium SAND (SM), Medium Dense
14	Mottled Orange to Gray, Moist, fat CLAY (CH) with trace fine Sand, Very Stiff
33	Mottled Orange to Gray, Moist to Wet, poorly graded fine to coarse SAND (SP-SM) with Silt, Loose to Medium Dense
	Mottled Orange to Gray, Wet, fat CLAY (CH) with trace fine Sand, Very Soft

Sample No.	Sample Recovery	Sample Type	Blows Per 6"	N Value	% < #200
1	14	SS	2-2-3-3	5	
2	20	SS	4-5-7-7	12	
3	17	SS	5-7-9-11	16	
4	19	SS	9-11-14-15	25	
5	24	SS	6-8-10-11	18	
6	17	SS	6-6-12-12	18	
7	20	SS	8-9-11-12	20	
8	14	SS	7-10-10-10	20	9
9	11	SS	4-3-3-3	6	10
10	4	SS	5-3-2-2	5	
11	24	SS	1-1-1-2	2	59

TEST RESULTS

Plastic Limit ⊢⊣ Liquid Limit
Moisture Content - ●
N-Value -



Notes: *It is noted that the topsoil thickness noted above is not expected to be indicative of the thicknesses that will be encountered across the site as the boring locations were cleared for accessibility prior to drilling.

The initial groundwater reading may not be indicative of the static groundwater level.

SS = Split Spoon Sample
ST = Shelby Tube Sample
HA = Hand Auger Sample
BS = Bulk Sample
WOH = Weight of Hammer

Standard Penetration Tests were performed in the field in general accordance with ASTM D 1586.

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BORING LOG M-2

PROJECT: Hopewell Regional Alternative 4A-1 Light Phase 2 PER
CLIENT: HDR Engineering, Inc.
PROJECT LOCATION: Hopewell, Virginia
PROJECT NO.: WM13-136G
BORING LOCATION: See attached boring location plan
SURFACE ELEVATION: 27'
DRILLER: GET Solutions, Inc.
LOGGED BY: J. Robinson, P.E.
DRILLING METHOD: Rotary wash "mud"
DATE: 7-28-13
DEPTH TO WATER - INITIAL*: 23' **AFTER 24 HOURS:**
CAVING>

Elevation (MSL) (ft)	Depth (meters)	Depth (feet)	Description	Graphic	Sample No.	Sample Recovery	Sample Type	Blows Per 6"	N Value	% < #200	TEST RESULTS (Plastic Limit — Liquid Limit; Moisture Content - ●; N-Value - hatched bar; 10 20 30 40 50 60 70)
	12	38–40	Mottled Orange to Gray, Wet, poorly graded fine to coarse SAND (SP-SM) with Silt, Medium Dense		12	24	SS	4 8 10 10	18		
		40	Boring terminated at 40 ft.								

Elevation scale: -15, -20, -25, -30, -35, -40, -45
Depth (meters) scale: 12, 14, 16, 18, 20, 22
Depth (feet) scale: 40, 45, 50, 55, 60, 65, 70, 75

Notes: *It is noted that the topsoil thickness noted above is not expected to be indicative of the thicknesses that will be encountered across the site as the boring locations were cleared for accessibility prior to drilling.

The initial groundwater reading may not be indicative of the static groundwater level.

SS = Split Spoon Sample
ST = Shelby Tube Sample
HA = Hand Auger Sample
BS = Bulk Sample
WOH = Weight of Hammer

Standard Penetration Tests were performed in the field in general accordance with ASTM D 1586.

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BORING LOG M-3

PROJECT: Hopewell Regional Alternative 4A-1 Light Phase 2 PER

CLIENT: HDR Engineering, Inc.

PROJECT LOCATION: Hopewell, Virginia — **PROJECT NO.:** WM13-136G

BORING LOCATION: See attached boring location plan — **SURFACE ELEVATION:** 20'

DRILLER: GET Solutions, Inc. — **LOGGED BY:** J. Robinson, P.E.

DRILLING METHOD: Rotary wash "mud" — **DATE:** 7-28-13

DEPTH TO WATER - INITIAL*: 18' **AFTER 24 HOURS:** — **CAVING>**

Elevation (MSL) (ft)	Depth (meters)	Depth (feet)
20	0	0
15		5
	2	
10		10
	4	
5		15
0	6	20
-5		25
	8	
-10		30
	10	
-15		35

Depth (feet)	Description	Sample No.	Sample Recovery	Sample Type	Blows Per 6"	N Value	% < #200
0	4" Topsoil						
0.33	Mottled Orange to Gray, Moist, fine Sandy lean CLAY (CL) with trace fibrous organics and trace wood fragements, Stiff	1	17	SS	2 4 6 8	10	
2	Mottled Orange to Gray, Moist, Silty fine to medium SAND (SM), Medium Dense	2	20	SS	6 7 9 11	16	
4	Mottled Orange to Gray, Moist, fat CLAY (CH) with trace fine Sand, Very Stiff to Hard	3	22	SS	10 11 14 16	25	
		4	17	SS	13 14 17 19	31	
8	Mottled Orange to Gray, Moist, poorly graded fine to medium SAND (SP-SM) with Silt, Medium Dense	5	24	SS	9 10 11 12	21	
		6	17	SS	9 9 11 11	20	
		7	14	SS	7 7 6 7	13	7
18	Mottled Orange to Gray, Moist, fine Sandy Silty CLAY (CL-ML), Very Stiff	8	15	SS	8 10 13 17	23	74
23	Mottled Orange to Gray, Wet, Silty fine to medium SAND (SM), Medium Dense	9	14	SS	10 11 13 14	24	18
28	Mottled Orange to Gray, Wet, fat CLAY (CH) with trace fine Sand, Soft	10	24	SS	1 1 2 2	3	99
33	Mottled Orange to Gray, Wet, poorly graded fine to coarse SAND (SP-SM) with Silt, Medium Dense	11	17	SS	9 9 10 7	19	

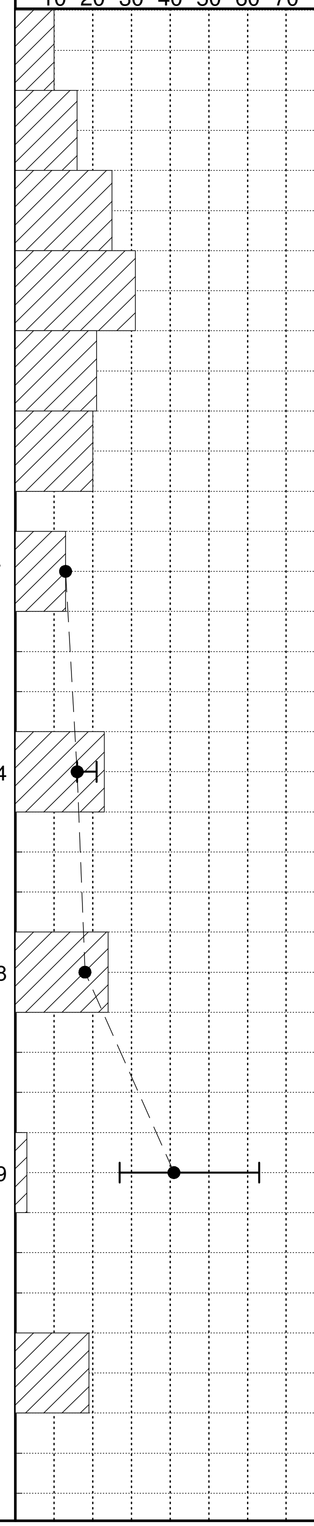
TEST RESULTS

Plastic Limit ├─┤ Liquid Limit

Moisture Content - ●

N-Value -

10 20 30 40 50 60 70



Notes: *It is noted that the topsoil thickness noted above is not expected to be indicative of the thicknesses that will be encountered across the site as the boring locations were cleared for accessibility prior to drilling.

The initial groundwater reading may not be indicative of the static groundwater level.

SS = Split Spoon Sample
ST = Shelby Tube Sample
HA = Hand Auger Sample
BS = Bulk Sample
WOH = Weight of Hammer

Standard Penetration Tests were performed in the field in general accordance with ASTM D 1586.

BORING LOG
M-3

PROJECT: Hopewell Regional Alternative 4A-1 Light Phase 2 PER

CLIENT: HDR Engineering, Inc.

PROJECT LOCATION: Hopewell, Virginia **PROJECT NO.:** WM13-136G

BORING LOCATION: See attached boring location plan **SURFACE ELEVATION:** 20'

DRILLER: GET Solutions, Inc. **LOGGED BY:** J. Robinson, P.E.

DRILLING METHOD: Rotary wash "mud" **DATE:** 7-28-13

DEPTH TO WATER - INITIAL*: 18' **AFTER 24 HOURS:** **CAVING>**

Elevation (MSL) (ft)	Depth (meters)	Depth (feet)	Description	Sample No.	Sample Recovery	Sample Type	Blows Per 6"	N Value	% < #200
	12		Mottled Orange to Gray, Wet, poorly graded fine to coarse SAND (SP-SM) with Silt, Medium Dense	12	12	SS	4 6 8 8	14	
-20		40							
-25		45		13	14	SS	7 12 18 20	30	
	14								
-30		50		14	18	SS	11 9 10 12	19	
			Boring terminated at 50 ft.						
	16								
-35		55							
	18								
-40		60							
-45		65							
	20								
-50		70							
	22								
-55		75							

TEST RESULTS

Plastic Limit ├┤ Liquid Limit

Moisture Content - ●

N-Value -

10 20 30 40 50 60 70

Notes: *It is noted that the topsoil thickness noted above is not expected to be indicative of the thicknesses that will be encountered across the site as the boring locations were cleared for accessibility prior to drilling.

The initial groundwater reading may not be indicative of the static groundwater level.

SS = Split Spoon Sample
ST = Shelby Tube Sample
HA = Hand Auger Sample
BS = Bulk Sample
WOH = Weight of Hammer

Standard Penetration Tests were performed in the field in general accordance with ASTM D 1586.

BORING LOG M-4

GET Solutions, Inc.
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PROJECT: Hopewell Regional Alternative 4A-1 Light Phase 2 PER

CLIENT: HDR Engineering, Inc.

PROJECT LOCATION: Hopewell, Virginia | **PROJECT NO.:** WM13-136G

BORING LOCATION: See attached boring location plan | **SURFACE ELEVATION:** 15'

DRILLER: GET Solutions, Inc. | **LOGGED BY:** J. Robinson, P.E.

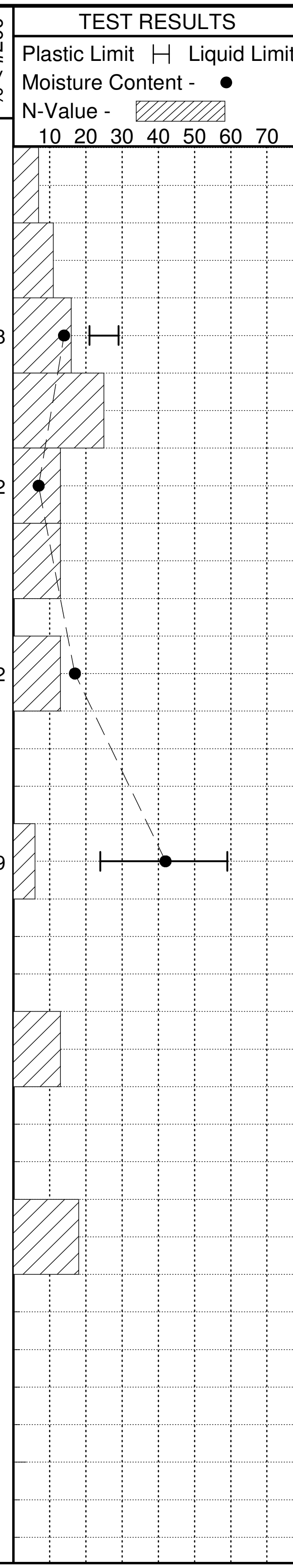
DRILLING METHOD: Rotary wash "mud" | **DATE:** 7-28-13

DEPTH TO WATER - INITIAL*: 13' **AFTER 24 HOURS:** **CAVING>**

Depth (feet)	Description
0 – 0.33	4" Topsoil
0.33 – 7.5	Mottled Orange to Gray, Moist, lean CLAY (CL) with varying amounts of fine Sand, Medium Stiff to Very Stiff
7.5 – 18	Mottled Orange to Gray, Moist to Wet, poorly graded fine to medium SAND (SP-SM) with Silt, Medium Dense
18 – 24	Mottled Orange to Gray, Wet, fat CLAY (CH) with trace fine Sand, Medium Stiff to Stiff
24 – 30	Mottled Orange to Gray, Wet, poorly graded fine to coarse SAND (SP-SM) with Silt, Medium Dense
30	Boring terminated at 30 ft.

Sample No.	Sample Recovery	Sample Type	Blows Per 6"	N Value	% < #200
1	17	SS	1-2-5-7	7	
2	14	SS	5-5-6-9	11	
3	20	SS	6-7-9-10	16	58
4	20	SS	12-12-13-12	25	
5	11	SS	7-6-7-8	13	12
6	14	SS	5-6-7-8	13	
7	24	SS	6-6-7-11	13	12
8	13	SS	3-3-3-3	6	99
9	24	SS	7-8-5-5	13	
10	12	SS	4-10-8-10	18	

TEST RESULTS



Notes: *It is noted that the topsoil thickness noted above is not expected to be indicative of the thicknesses that will be encountered across the site as the boring locations were cleared for accessibility prior to drilling.

The initial groundwater reading may not be indicative of the static groundwater level.

SS = Split Spoon Sample
ST = Shelby Tube Sample
HA = Hand Auger Sample
BS = Bulk Sample
WOH = Weight of Hammer

Standard Penetration Tests were performed in the field in general accordance with ASTM D 1586.

GET Solutions, Inc.

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BORING LOG F-8

PROJECT: Hopewell Regional Alternative 4A-1 Light Phase 2 PER

CLIENT: HDR Engineering, Inc.

PROJECT LOCATION: Hopewell, Virginia — **PROJECT NO.:** WM13-136G

BORING LOCATION: See attached boring location plan — **SURFACE ELEVATION:**

DRILLER: GET Solutions, Inc. — **LOGGED BY:** J. Robinson, P.E.

DRILLING METHOD: Rotary wash "mud" — **DATE:** 9-6-13

DEPTH TO WATER - INITIAL*: 23' — **AFTER 24 HOURS:** — **CAVING>**

Depth (feet)	Description	Sample No.	Sample Recovery	Sample Type	Blows Per 6"	N Value	% < #200	Moisture Content
0 – 0.58	7" Topsoil							
0.58 – 2	Mottled Orange to Gray, Moist, Clayey fine SAND (SC), Very Loose	1	18	SS	1-2-2-4	4		
2 – 6	Mottled Orange to Gray, Moist, Silty fine to medium SAND (SM), Loose to Medium Dense	2	18	SS	3-4-5-4	9		
		3	20	SS	4-5-6-7	11		
6 – 8	Mottled Orange to Gray, Moist, Clayey fine to medium SAND (SC), Medium Dense	4	18	SS	6-7-10-11	17		
8 – 18	Mottled Orange to Gray, Moist, fat CLAY (CH) with trace to little fine Sand, Stiff to Very Stiff	5	24	SS	6-7-12-11	19		
		6	24	SS	8-10-10-15	20		
		7	24	SS	3-5-8-9	13		
18 – 28	Gray, Moist to Wet, Silty fine to coarse SAND (SM) with trace fine Gravel, Medium Dense	8	24	SS	9-10-11-12	21	23	●
		9	12	SS	9-12-17-16	29		
28 – 33	Mottled Orange to Gray, Wet, fat CLAY (CH) with trace fine Sand, Very Soft	10	24	SS	WOH-1-1-1	2		
33 –	Mottled Orange to Gray, Wet, Silty fine to medium SAND (SM), Medium Dense	11	24	SS	3-6-6-4	12		

TEST RESULTS: Plastic Limit — Liquid Limit; Moisture Content - ●; N-Value - (hatched bar); scale 10 20 30 40 50 60 70

Notes: *It is noted that the topsoil thickness noted above is not expected to be indicative of the thicknesses that will be encountered across the site as the boring locations were cleared for accessibility prior to drilling.

The initial groundwater reading may not be indicative of the static groundwater level.

SS = Split Spoon Sample
ST = Shelby Tube Sample
HA = Hand Auger Sample
BS = Bulk Sample
WOH = Weight of Hammer

Standard Penetration Tests were performed in the field in general accordance with ASTM D 1586.

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BORING LOG F-8

PROJECT: Hopewell Regional Alternative 4A-1 Light Phase 2 PER
CLIENT: HDR Engineering, Inc.
PROJECT LOCATION: Hopewell, Virginia
PROJECT NO.: WM13-136G
BORING LOCATION: See attached boring location plan
SURFACE ELEVATION:
DRILLER: GET Solutions, Inc.
LOGGED BY: J. Robinson, P.E.
DRILLING METHOD: Rotary wash "mud"
DATE: 9-6-13
DEPTH TO WATER - INITIAL*: 23' **AFTER 24 HOURS:**
CAVING>

Elevation (MSL) (ft)	Depth (meters)	Depth (feet)	Description	Graphic	Sample No.	Sample Recovery	Sample Type	Blows Per 6"	N Value	% < #200	TEST RESULTS (Plastic Limit / Liquid Limit; Moisture Content; N-Value; 10 20 30 40 50 60 70)
	12	38–43	Mottled Orange to Gray, Wet, fat CLAY (CH) with trace fine Sand, Stiff		12	24	SS	4 4 5 6	9		
	14–18	43–58	Gray/Brown, Wet, Silty fine to medium SAND (SM) with varying amounts of Clay, Loose to Medium Dense		13	22	SS	3 3 3 3	6		
					14	12	SS	5 3 6 5	9		
					15	24	SS	14 12 13 16	25		
	18	58–60	Gray/Brown, Wet, poorly graded fine to medium SAND (SP-SM) with Silt, Dense		16	24	SS	11 18 24 22	42		
		60	Boring terminated at 60 ft.								

Depth scale (meters): 12, 14, 16, 18, 20, 22
Depth scale (feet): 40, 45, 50, 55, 60, 65, 70, 75

Notes: *It is noted that the topsoil thickness noted above is not expected to be indicative of the thicknesses that will be encountered across the site as the boring locations were cleared for accessibility prior to drilling.

*The initial groundwater reading may not be indicative of the static groundwater level.

SS = Split Spoon Sample
ST = Shelby Tube Sample
HA = Hand Auger Sample
BS = Bulk Sample
WOH = Weight of Hammer

Standard Penetration Tests were performed in the field in general accordance with ASTM D 1586.

BORING LOG F-9

PROJECT: Hopewell Regional Alternative 4A-1 Light Phase 2 PER
CLIENT: HDR Engineering, Inc.
PROJECT LOCATION: Hopewell, Virginia
PROJECT NO.: WM13-136G
BORING LOCATION: See attached boring location plan
SURFACE ELEVATION:
DRILLER: GET Solutions, Inc.
LOGGED BY: J. Robinson, P.E.
DRILLING METHOD: Rotary wash "mud"
DATE: 9-5-13
DEPTH TO WATER - INITIAL*: 33' **AFTER 24 HOURS:**
CAVING>

Depth (feet)	Description
0 – 0.33	4" Topsoil
0.33 – 13	Mottled Orange to Gray, Moist, lean CLAY (CL) with varying amounts of fine Sand, Stiff to Hard
13 – 23	Mottled Orange to Gray, Moist, Silty fine SAND (SM), Medium Dense
23 – 28	Mottled Orange to Gray, Moist, lean CLAY (CL) with trace fine Sand, Stiff
28 – 33	Mottled Orange to Gray, Moist, Silty fine to medium SAND (SM), Medium Dense
33 –	Brown, Wet, poorly graded fine to coarse SAND (SP-SM) with Silt and trace fine Gravel, Loose

Sample No.	Sample Recovery	Sample Type	Blows Per 6"	N Value	% < #200
1	8	SS	3-4-6-8	10	
2	10	SS	8-11-14-21	25	
3	10	SS	15-22-23-26	45	
4	18	SS	28-26-27-29	53	
5	16	SS	11-12-13-15	25	
6	13	SS	6-8-11-14	19	
7	10	SS	6-6-9-14	15	
8	21	SS	5-5-7-8	12	
9	24	SS	4-6-7-10	13	99
10	12	SS	6-7-9-8	16	
11	8	SS	4-3-5-7	8	

TEST RESULTS: Plastic Limit ├┤ Liquid Limit; Moisture Content - ●; N-Value - (hatched bar); 10 20 30 40 50 60 70

Notes: *It is noted that the topsoil thickness noted above is not expected to be indicative of the thicknesses that will be encountered across the site as the boring locations were cleared for accessibility prior to drilling.

The initial groundwater reading may not be indicative of the static groundwater level.

SS = Split Spoon Sample
ST = Shelby Tube Sample
HA = Hand Auger Sample
BS = Bulk Sample
WOH = Weight of Hammer

Standard Penetration Tests were performed in the field in general accordance with ASTM D 1586.

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BORING LOG F-9

PROJECT: Hopewell Regional Alternative 4A-1 Light Phase 2 PER
CLIENT: HDR Engineering, Inc.
PROJECT LOCATION: Hopewell, Virginia
PROJECT NO.: WM13-136G
BORING LOCATION: See attached boring location plan
SURFACE ELEVATION:
DRILLER: GET Solutions, Inc.
LOGGED BY: J. Robinson, P.E.
DRILLING METHOD: Rotary wash "mud"
DATE: 9-5-13
DEPTH TO WATER - INITIAL*: 33' **AFTER 24 HOURS:**
CAVING>

Elevation (MSL) (ft)	Depth (meters)	Depth (feet)	Description	Graphic	Sample No.	Sample Recovery	Sample Type	Blows Per 6"	N Value	% < #200	TEST RESULTS
	12	40	Brown, Wet, poorly graded fine to coarse SAND (SP-SM) with Silt and trace fine Gravel, Loose		12	20	SS	2 3 7 13	10		
	14	43–48 (45)	Mottled Orange to Gray, Wet, fat CLAY (CH) with trace fine Sand, Soft		13	24	SS	1 1 3 3	4		
	16	48–53 (50)	Gray, Wet, Silty fine to medium SAND (SM), Medium Dense		14	22	SS	4 5 8 10	13		
	18	53–60 (55, 60)	Mottled Orange to Gray, Wet, Silty fine to medium SAND (SM) with varying amounts of Clay, Loose to Medium Dense		15	24	SS	3 5 9 12	14		
					16	24	SS	3 3 5 4	8		
	20, 22	65, 70, 75	Boring terminated at 60 ft.								

TEST RESULTS: Plastic Limit ├┤ Liquid Limit; Moisture Content - ●; N-Value - (hatched bar); 10 20 30 40 50 60 70

Notes: *It is noted that the topsoil thickness noted above is not expected to be indicative of the thicknesses that will be encountered across the site as the boring locations were cleared for accessibility prior to drilling.

The initial groundwater reading may not be indicative of the static groundwater level.

SS = Split Spoon Sample
ST = Shelby Tube Sample
HA = Hand Auger Sample
BS = Bulk Sample
WOH = Weight of Hammer

Standard Penetration Tests were performed in the field in general accordance with ASTM D 1586.

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BORING LOG F-10

PROJECT: Hopewell Regional Alternative 4A-1 Light Phase 2 PER
CLIENT: HDR Engineering, Inc.
PROJECT LOCATION: Hopewell, Virginia | **PROJECT NO.:** WM13-136G
BORING LOCATION: See attached boring location plan | **SURFACE ELEVATION:**
DRILLER: GET Solutions, Inc. | **LOGGED BY:** J. Robinson, P.E.
DRILLING METHOD: Rotary wash "mud" | **DATE:** 9-5-13
DEPTH TO WATER - INITIAL*: 38' **AFTER 24 HOURS:** | **CAVING>**

Depth (feet)	Description	Sample No.	Sample Recovery	Sample Type	Blows Per 6"	N Value	% < #200
0 – 0.58	7" Topsoil						
0.58 – 10	Mottled Orange to Gray, Moist, lean CLAY (CL) with varying amounts of fine Sand, Medium Stiff to Very Stiff	1	14	SS	2-2-3-4	5	
		2	12	SS	6-8-8-11	16	
		3	10	SS	8-10-12-15	22	
		4	19	SS	15-15-15-18	30	
		5	24	SS	5-8-12-15	20	
10 – 13	Mottled Orange to Gray, Moist, Clayey fine SAND (SC), Medium Dense	6	18	SS	6-9-12-13	21	
13 – 18	Mottled Orange to Gray, Moist, Silty fine SAND (SM), Medium Dense	7	12	SS	8-10-12-17	22	
18 – 33	Mottled Orange to Gray, Moist, fat CLAY (CH) with trace fine Sand, Stiff to Very Stiff	8	24	SS	5-7-10-14	17	
		9	24	SS	5-7-10-13	17	
		10	24	SS	3-4-8-11	12	
33 –	Mottled Orange to Gray, Moist to Wet, Silty fine to medium SAND (SM) with Clay lenses, Medium Dense	11	18	SS	6-9-14-13	23	

TEST RESULTS: Plastic Limit ├┤ Liquid Limit; Moisture Content - ●; N-Value - (hatched bar); scale 10 20 30 40 50 60 70

Notes: *It is noted that the topsoil thickness noted above is not expected to be indicative of the thicknesses that will be encountered across the site as the boring locations were cleared for accessibility prior to drilling.

SS = Split Spoon Sample
ST = Shelby Tube Sample
HA = Hand Auger Sample
BS = Bulk Sample
WOH = Weight of Hammer

The initial groundwater reading may not be indicative of the static groundwater level.

Standard Penetration Tests were performed in the field in general accordance with ASTM D 1586.

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BORING LOG F-10

PROJECT: Hopewell Regional Alternative 4A-1 Light Phase 2 PER
CLIENT: HDR Engineering, Inc.
PROJECT LOCATION: Hopewell, Virginia
PROJECT NO.: WM13-136G
BORING LOCATION: See attached boring location plan
SURFACE ELEVATION:
DRILLER: GET Solutions, Inc.
LOGGED BY: J. Robinson, P.E.
DRILLING METHOD: Rotary wash "mud"
DATE: 9-5-13
DEPTH TO WATER - INITIAL*: 38'
AFTER 24 HOURS:
CAVING>

Elevation (MSL) (ft)	Depth (meters)	Depth (feet)	Description	Graphic	Sample No.	Sample Recovery	Sample Type	Blows Per 6"	N Value	% < #200
	12	40	Mottled Orange to Gray, Moist to Wet, Silty fine to medium SAND (SM) with Clay lenses, Medium Dense		12	16	SS	4 7 9 10	16	
	14	43, 45	Mottled Orange to Gray, Wet, fat CLAY (CH) with trace fine Sand and trace fibrous organics, Medium Stiff to Stiff		13	24	SS	1 2 5 6	7	
	16	50			14	24	SS	2 3 3 5	6	
	18	54, 55, 60	Mottled Orange to Gray, Wet, poorly graded fine to coarse SAND (SP-SM) with Silt, Medium Dense		15	24	SS	2 7 12 14	19	
					16	22	SS	10 12 14 14	26	
	20, 22	60, 65, 70, 75	Boring terminated at 60 ft.							

TEST RESULTS
Plastic Limit — Liquid Limit
Moisture Content - ●
N-Value -
10 20 30 40 50 60 70

Notes: *It is noted that the topsoil thickness noted above is not expected to be indicative of the thicknesses that will be encountered across the site as the boring locations were cleared for accessibility prior to drilling.

The initial groundwater reading may not be indicative of the static groundwater level.

SS = Split Spoon Sample
ST = Shelby Tube Sample
HA = Hand Auger Sample
BS = Bulk Sample
WOH = Weight of Hammer

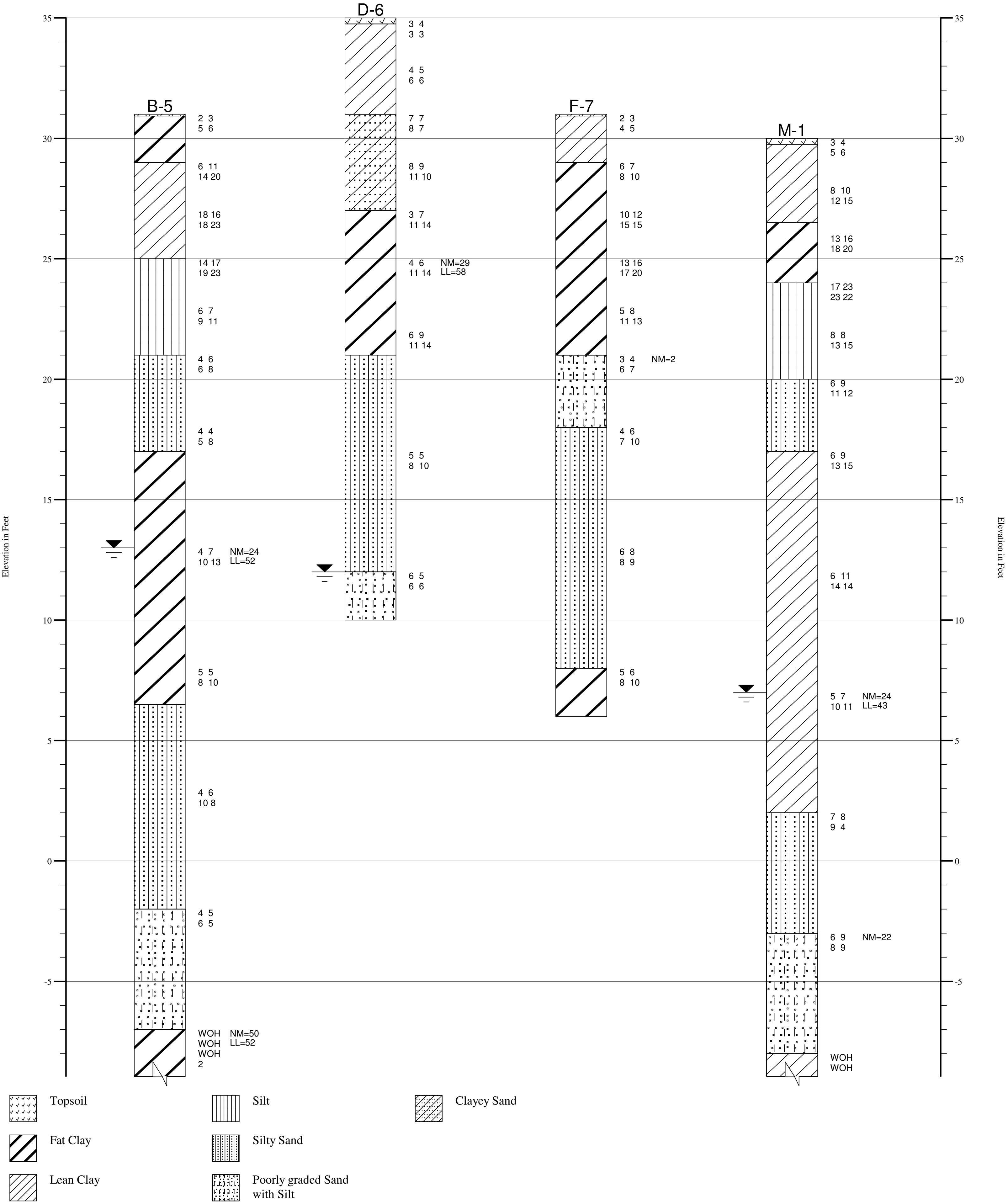
Standard Penetration Tests were performed in the field in general accordance with ASTM D 1586.

APPENDIX IV

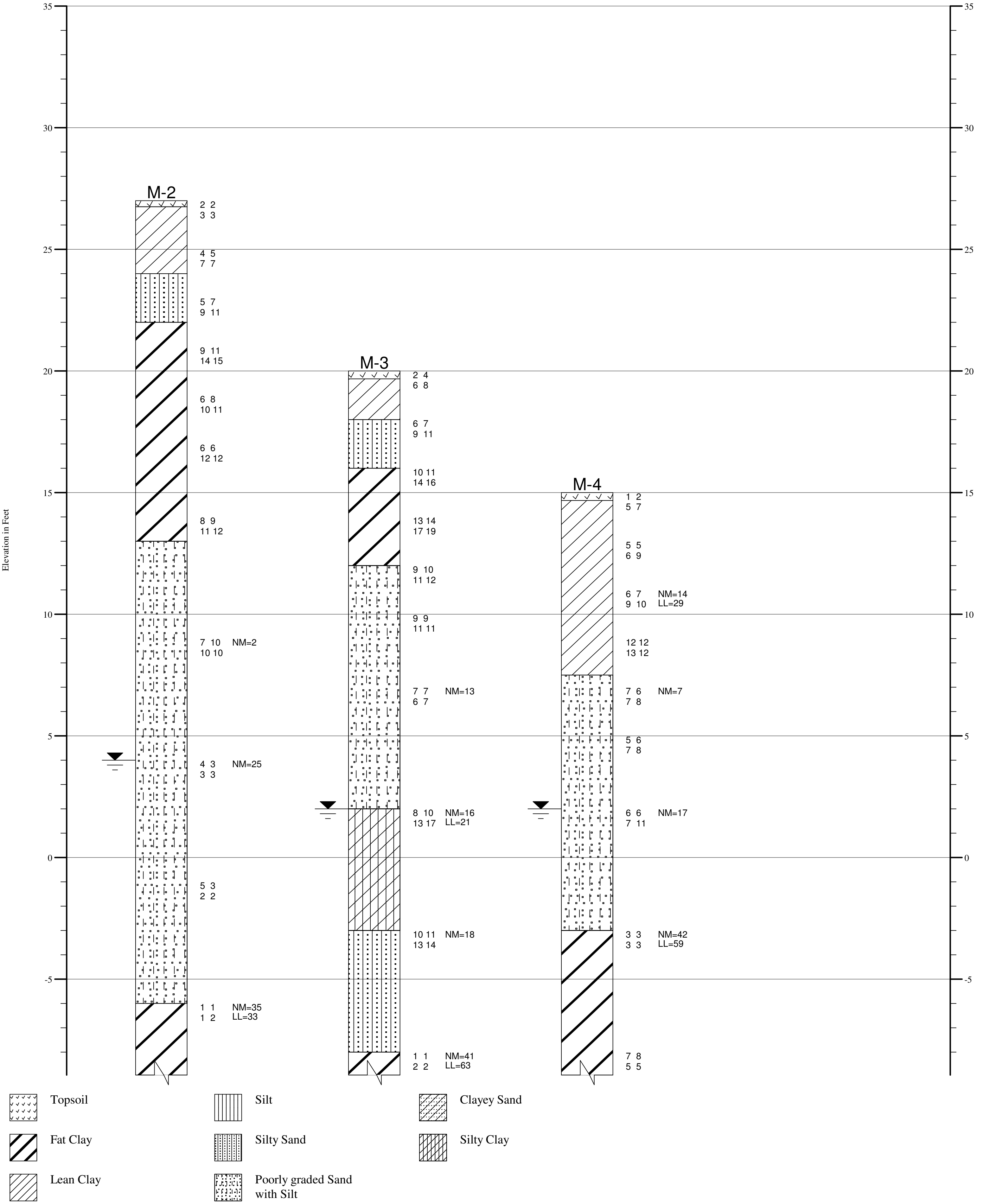
GENERALIZED SOIL PROFILE

LOG OF BORINGS

Hopewell Regional Alternative 4A-1 Light Phase 2 PER



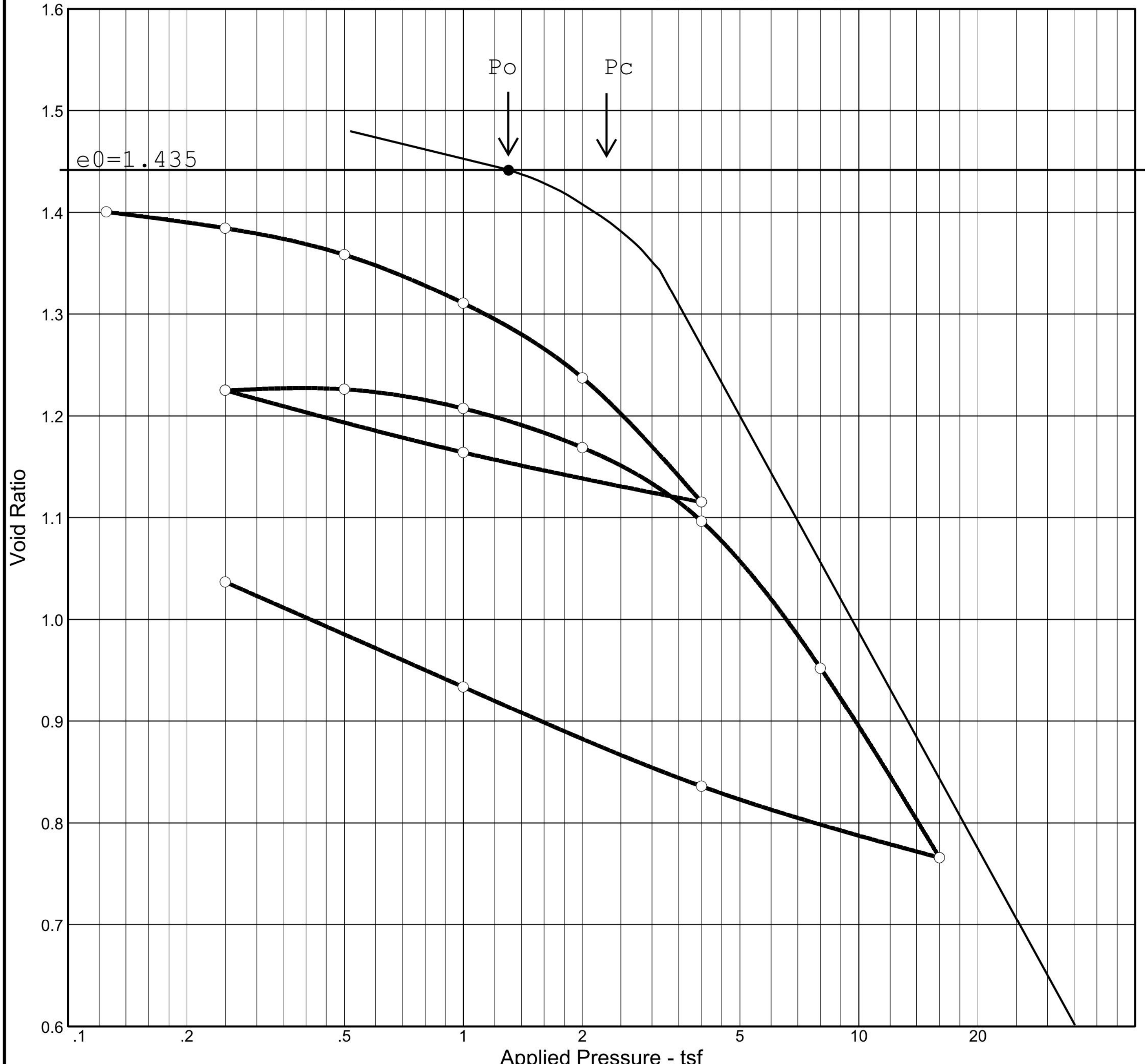
LOG OF BORINGS
Hopewell Regional Alternative 4A-1 Light Phase 2 PER



APPENDIX V

CONSOLIDATION TEST RESULTS

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (tsf)	P_c (tsf)	C_c	C_r	Initial Void Ratio
Saturation	Moisture									
91.7 %	48.6 %	70.5	63	36	2.71	1.30	2.30	0.705	0.10	1.435

MATERIAL DESCRIPTION	USCS	AASHTO
Gray, Fat CLAY	CH	A-7-6(42)

Project No. WM13-136G **Client:** HDR Engineering, Inc.

Project: Hopewell Regional Alternative 4A-1 Light Phase 2 PER

Location: M-3 (28-30 ft.) obtained 7/30/13

CONSOLIDATION TEST REPORT

GET SOLUTIONS, INC.

Remarks:

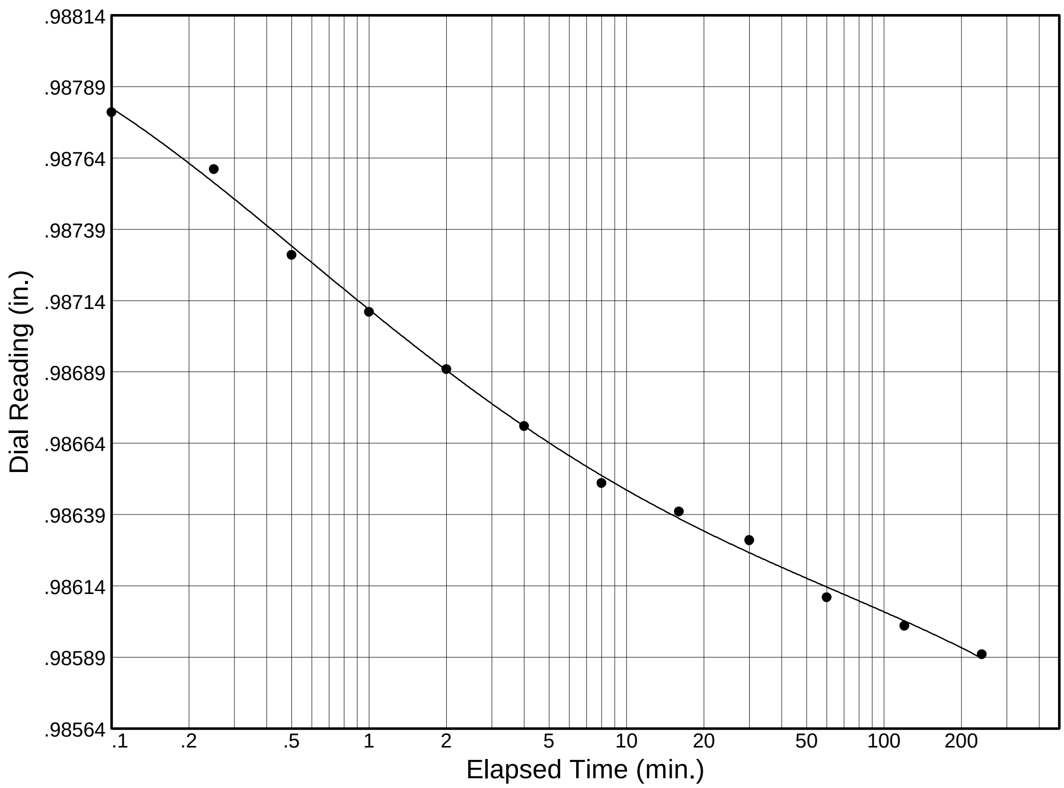
Sample Obtained 7/30/13
Passing #200 Sieve = 99.7%

Figure

Dial Reading vs. Time

Project No.: WM13-136G
Project: Hopewell Regional Alternative 4A-1 Light Phase 2 PER

Location: M-3 (28-30 ft.) obtained 7/30/13



Load No.= 1

Load= 0.13 tsf

D_0 = 0.98823

D_{50} = 0.98742

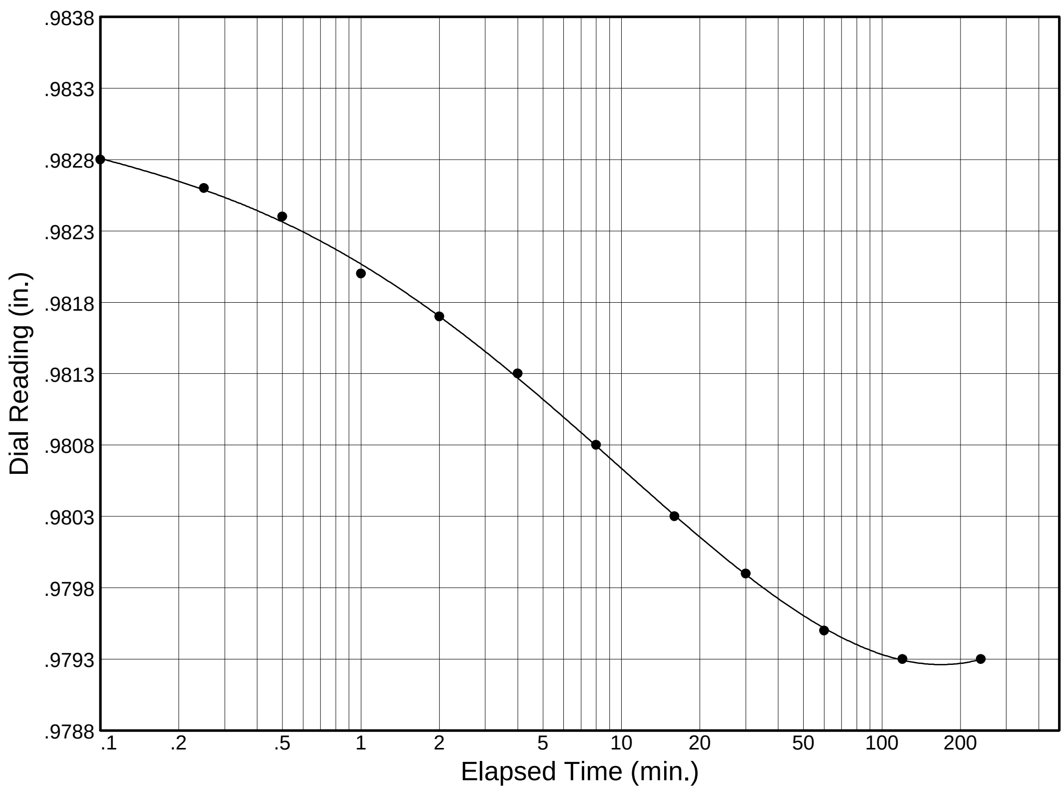
D_{100} = 0.98662

T_{50} = 0.38 min.

C_v @ T_{50}

1.27 ft.^{2}/day

C_α = 0.000



Load No.= 2

Load= 0.25 tsf

D_0 = 0.98285

D_{50} = 0.98107

D_{100} = 0.97930

T_{50} = 5.34 min.

C_v @ T_{50}

0.09 ft.^{2}/day

C_α = 0.000

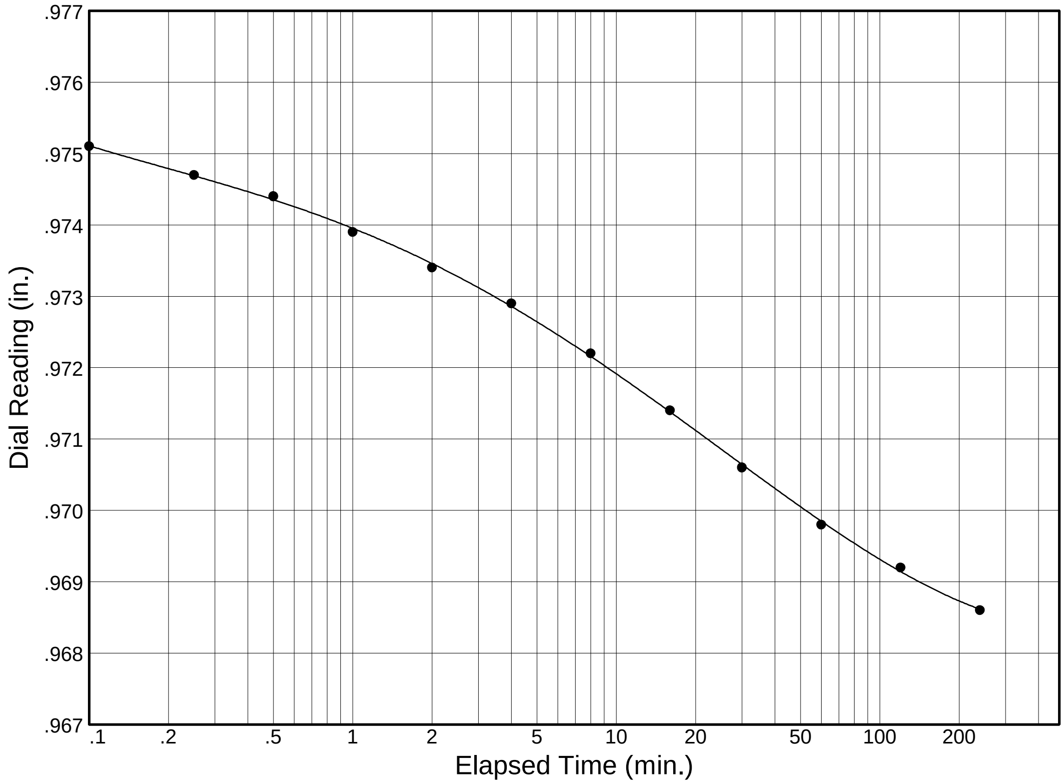
Figure

GET Solutions, Inc.

Dial Reading vs. Time

Project No.: WM13-136G
Project: Hopewell Regional Alternative 4A-1 Light Phase 2 PER

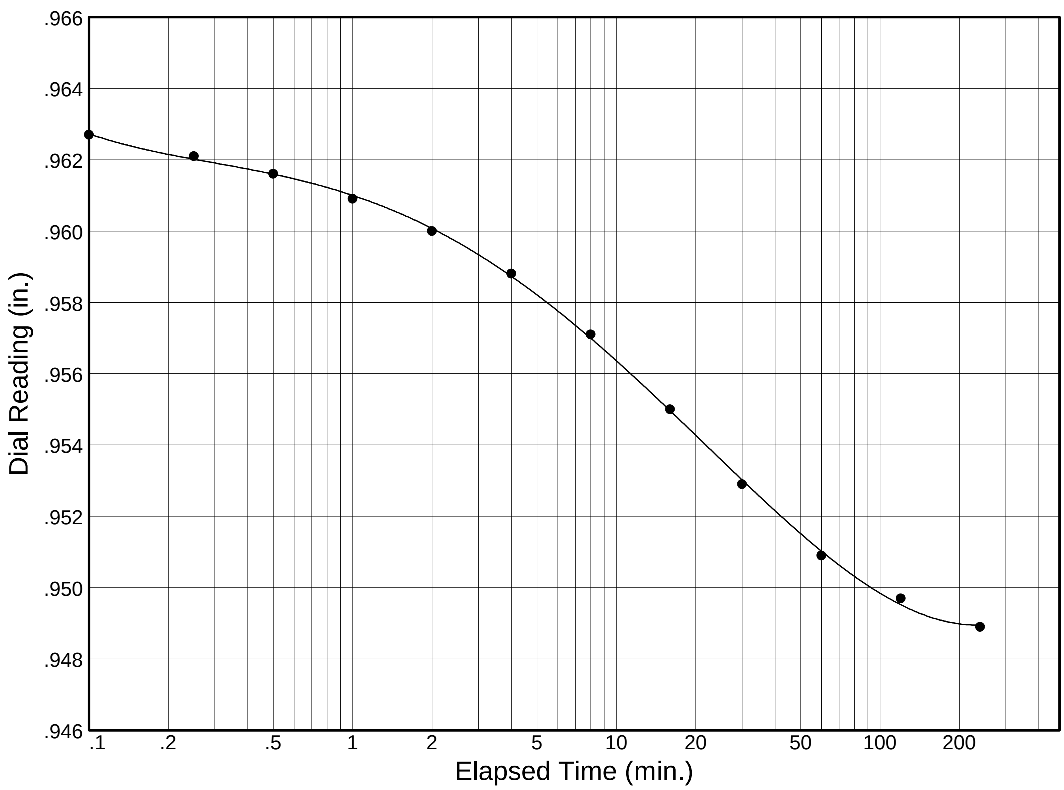
Location: M-3 (28-30 ft.) obtained 7/30/13



Load No.= 3
Load= 0.50 tsf
D_0 = 0.97542
D_{50} = 0.97206
D_{100} = 0.96869
T_{50} = 8.76 min.

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

C_α = 0.003



Load No.= 4
Load= 1.00 tsf
D_0 = 0.96303
D_{50} = 0.95604
D_{100} = 0.94904
T_{50} = 11.19 min.

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

C_α = 0.000

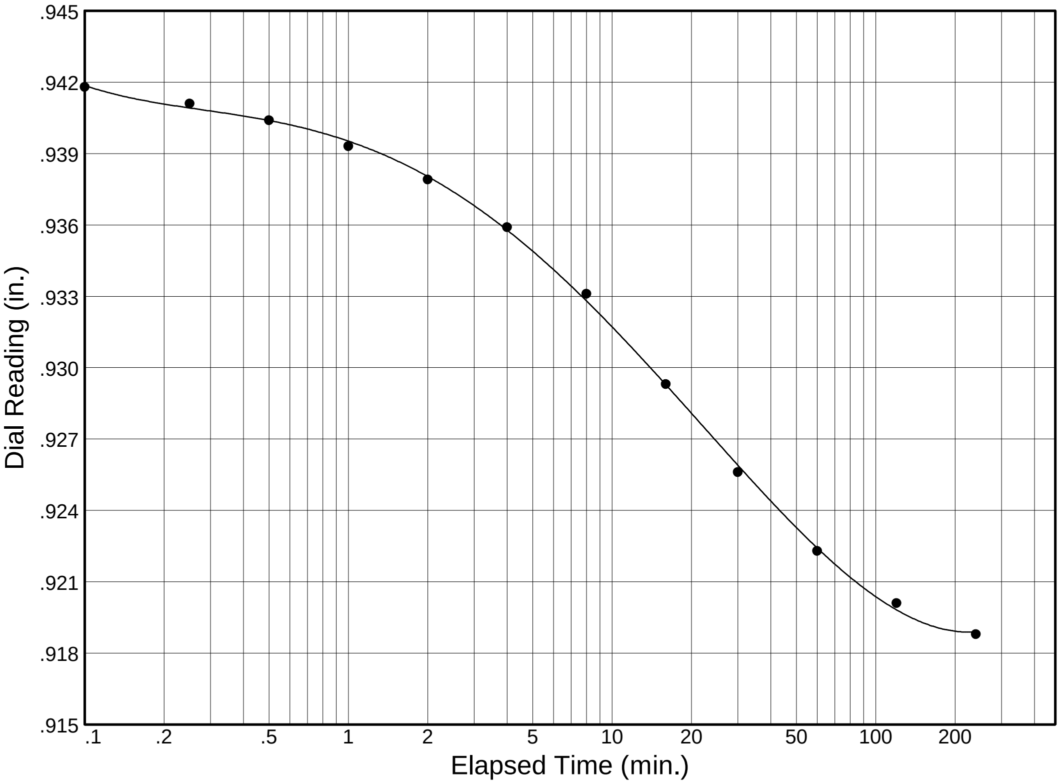
Figure

GET Solutions, Inc.

Dial Reading vs. Time

Project No.: WM13-136G
Project: Hopewell Regional Alternative 4A-1 Light Phase 2 PER

Location: M-3 (28-30 ft.) obtained 7/30/13



Load No.= 5

Load= 2.00 tsf

D_0 = 0.94232

D_{50} = 0.93068

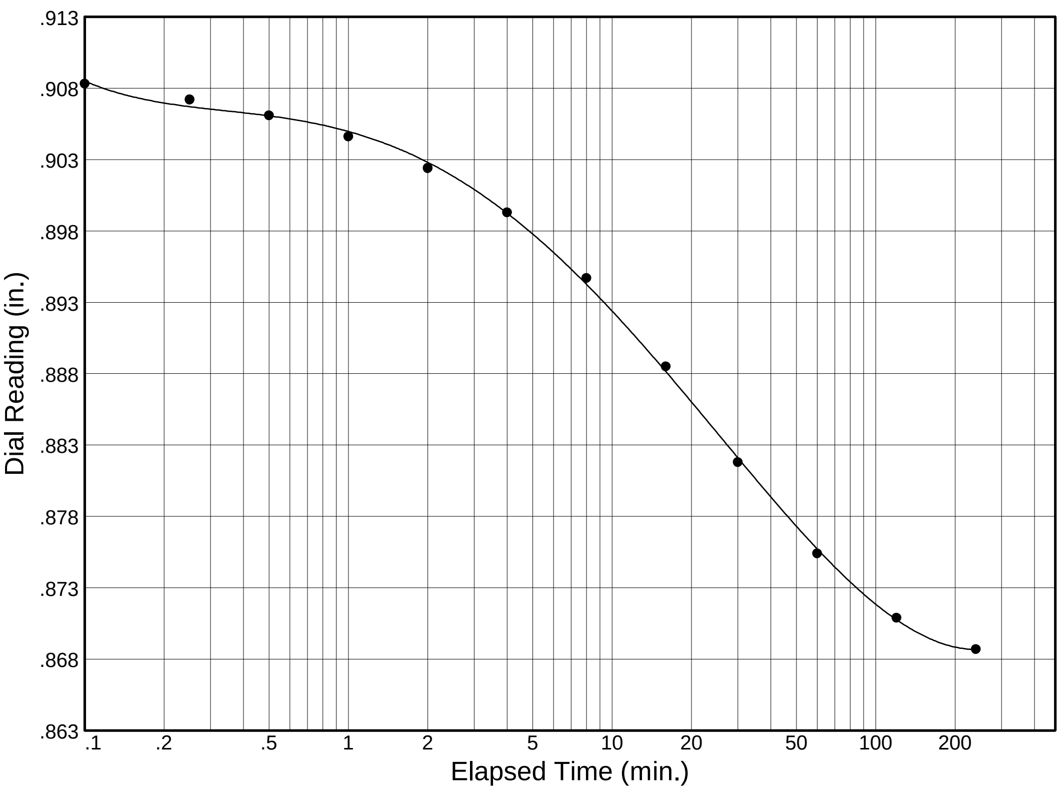
D_{100} = 0.91905

T_{50} = 12.25 min.

C_v @ T_{50}

0.03 ft.^{2}/day

C_α = 0.000



Load No.= 6

Load= 4.00 tsf

D_0 = 0.90852

D_{50} = 0.88858

D_{100} = 0.86865

T_{50} = 15.26 min.

C_v @ T_{50}

0.03 ft.^{2}/day

C_α = 0.000

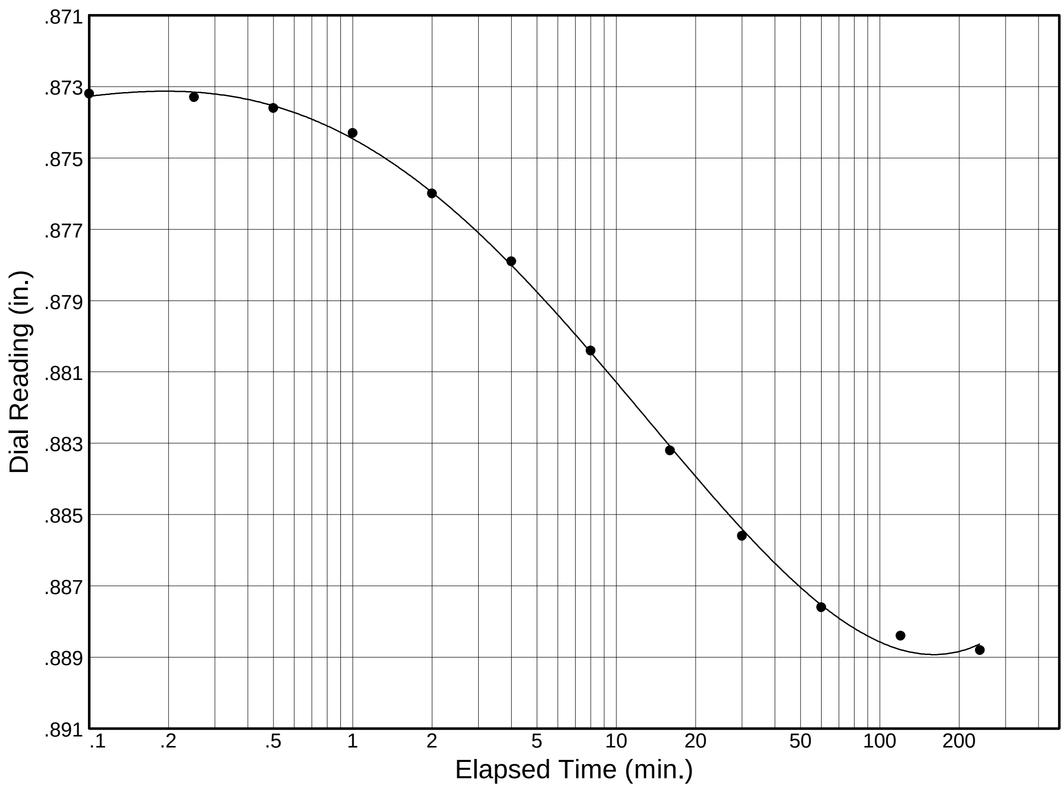
Figure

GET Solutions, Inc.

Dial Reading vs. Time

Project No.: WM13-136G
Project: Hopewell Regional Alternative 4A-1 Light Phase 2 PER

Location: M-3 (28-30 ft.) obtained 7/30/13



Load No.= 7

Load= 1.00 tsf

D_0 = 0.87258

D_{50} = 0.88085

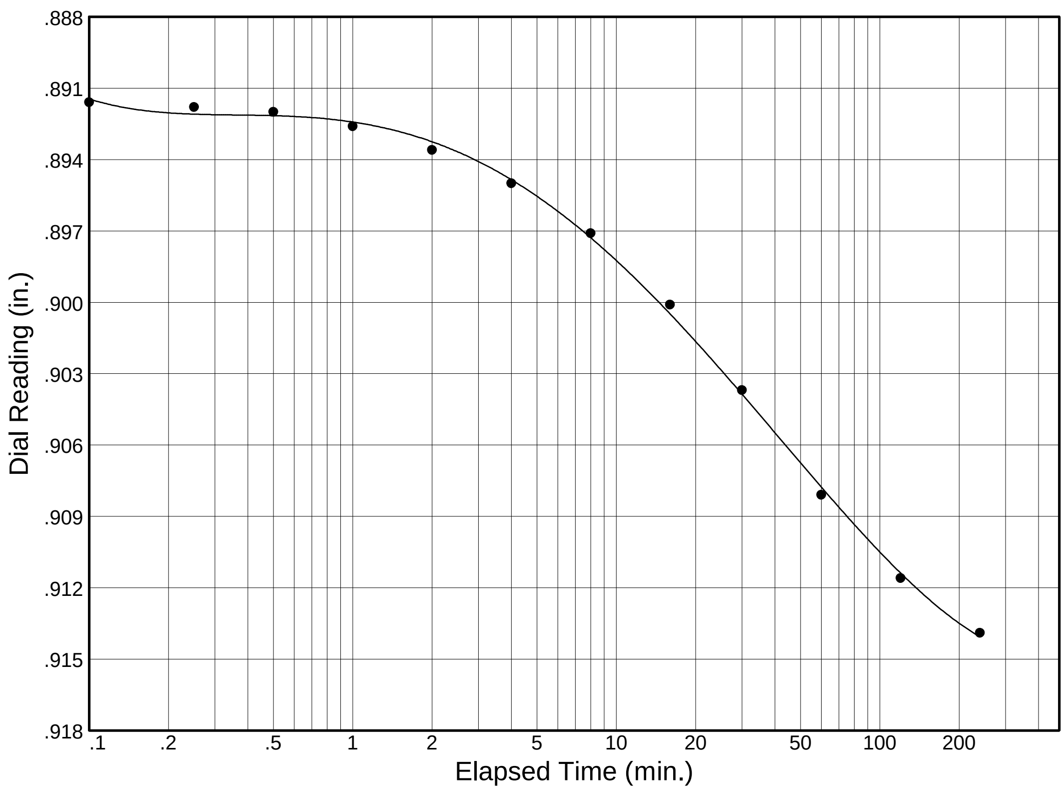
D_{100} = 0.88912

T_{50} = 8.91 min.

C_v @ T_{50}

0.04 ft.^{2}/day

C_α = 0.000



Load No.= 8

Load= 0.25 tsf

D_0 = 0.88989

D_{50} = 0.90168

D_{100} = 0.91347

T_{50} = 20.12 min.

C_v @ T_{50}

0.02 ft.^{2}/day

C_α = 0.001

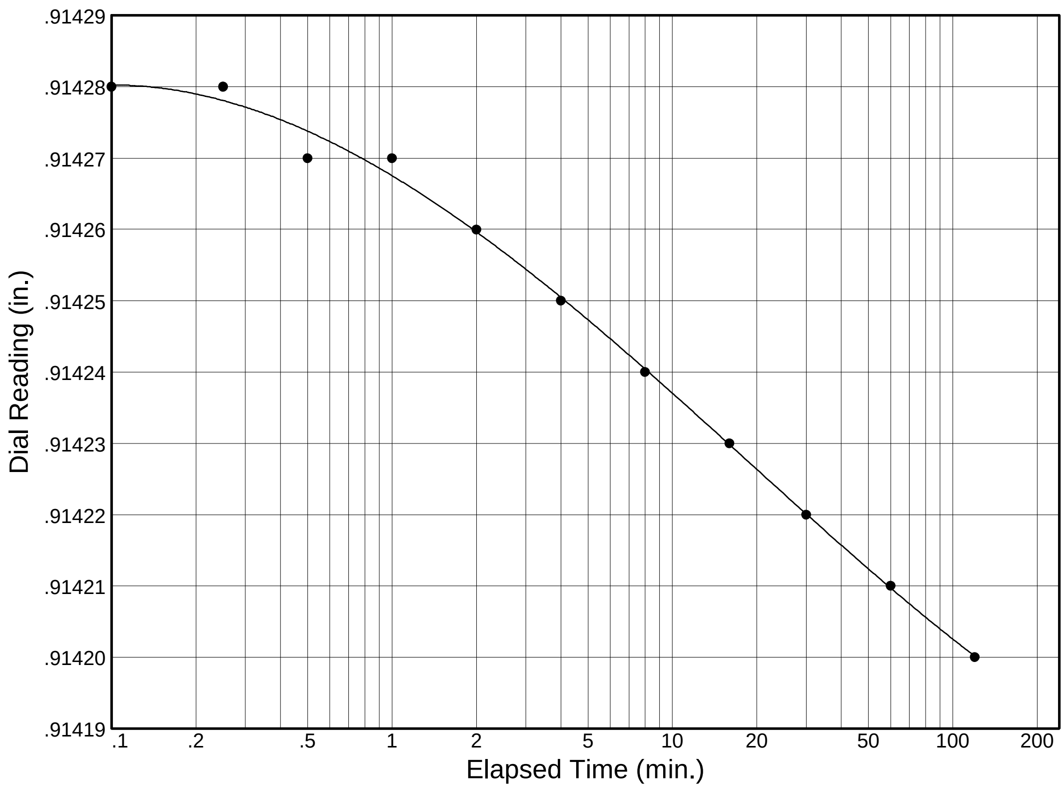
Figure

GET Solutions, Inc.

Dial Reading vs. Time

Project No.: WM13-136G
Project: Hopewell Regional Alternative 4A-1 Light Phase 2 PER

Location: M-3 (28-30 ft.) obtained 7/30/13



Load No.= 9

Load= 0.50 tsf

D_0 = 0.91429

D_{50} = 0.91425

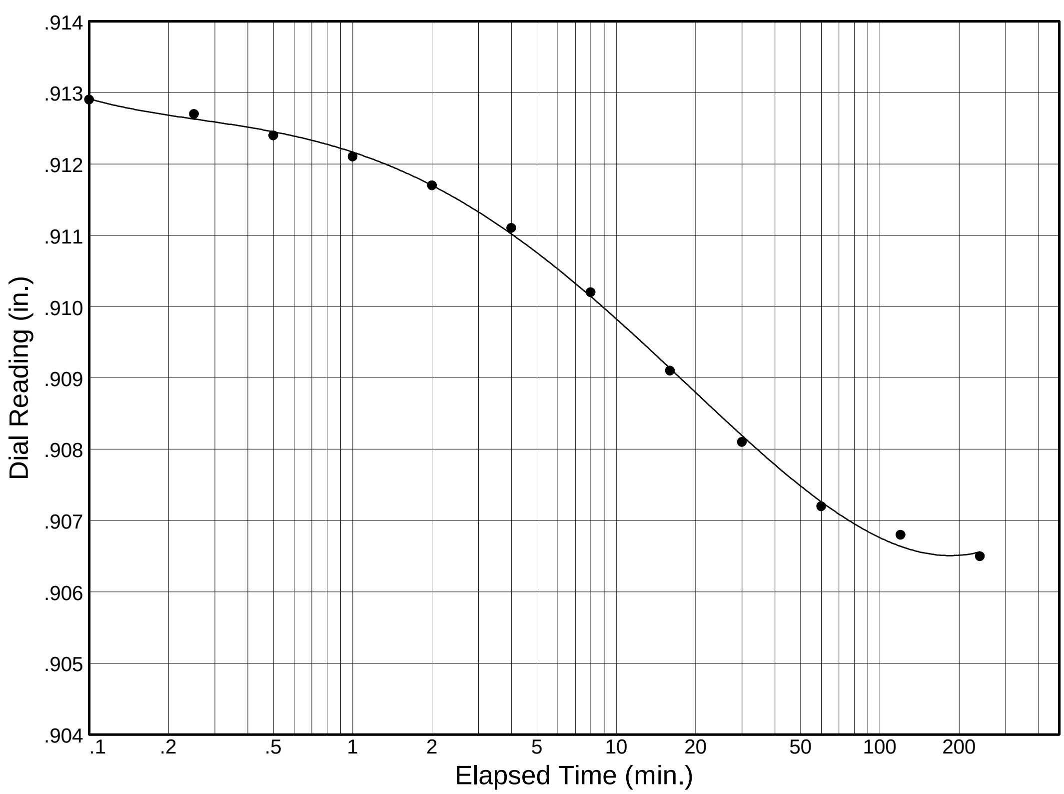
D_{100} = 0.91422

T_{50} = 3.51 min.

C_v @ T_{50}

0.12 ft.^{2}/day

C_α = 0.000



Load No.= 10

Load= 1.00 tsf

D_0 = 0.91309

D_{50} = 0.90980

D_{100} = 0.90651

T_{50} = 10.17 min.

C_v @ T_{50}

0.04 ft.^{2}/day

C_α = 0.000

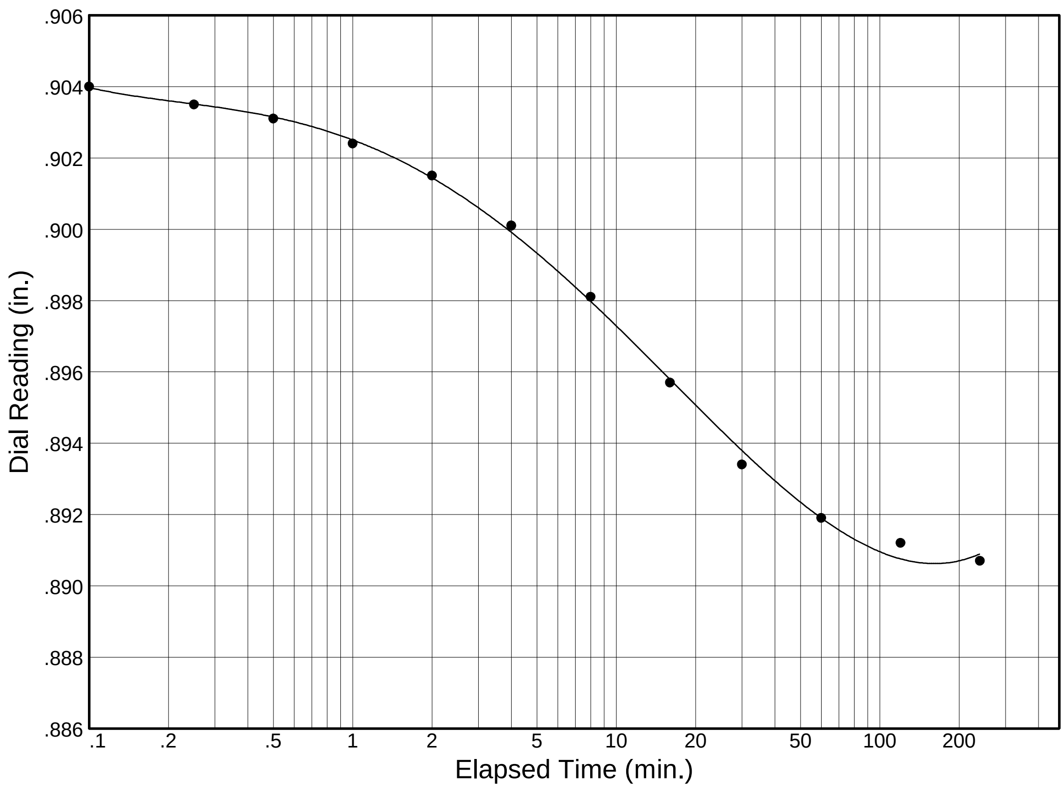
Figure

GET Solutions, Inc.

Dial Reading vs. Time

Project No.: WM13-136G
Project: Hopewell Regional Alternative 4A-1 Light Phase 2 PER

Location: M-3 (28-30 ft.) obtained 7/30/13



Load No.= 11

Load= 2.00 tsf

D_0 = 0.90450

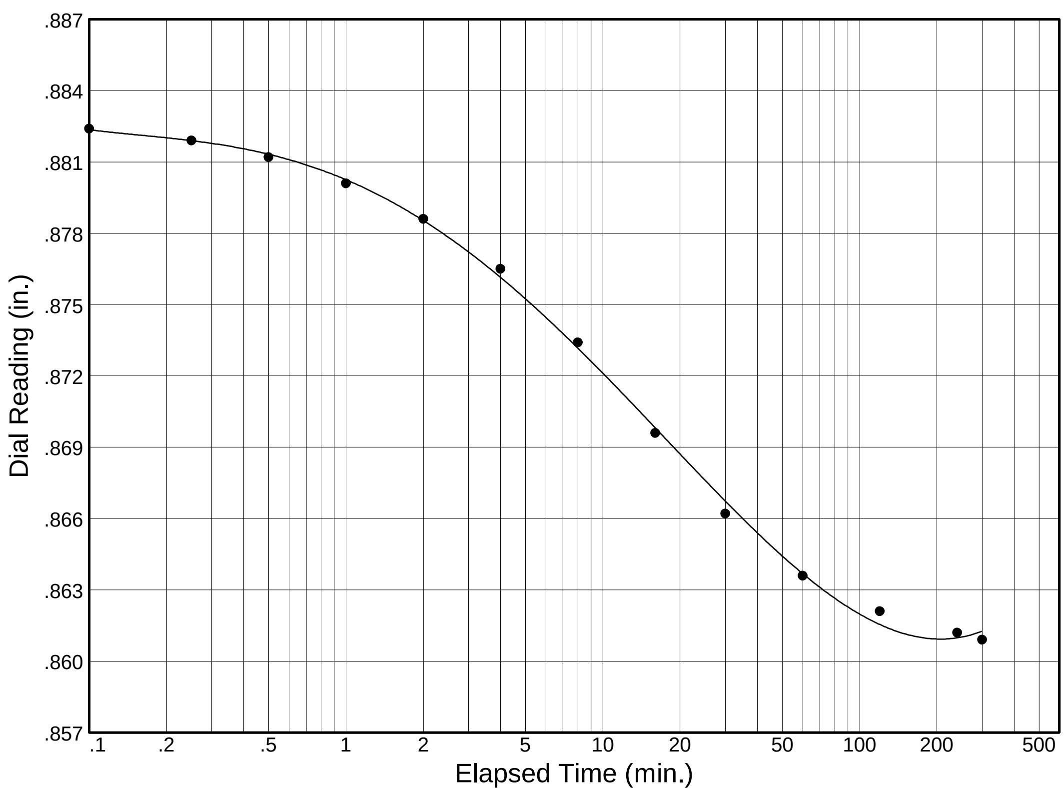
D_{50} = 0.89751

D_{100} = 0.89051

T_{50} = 9.33 min.

C_v @ T_{50}

0.04 ft.^{2}/day



Load No.= 12

Load= 4.00 tsf

D_0 = 0.88316

D_{50} = 0.87232

D_{100} = 0.86148

T_{50} = 9.58 min.

C_v @ T_{50}

0.04 ft.^{2}/day

C_α = 0.002

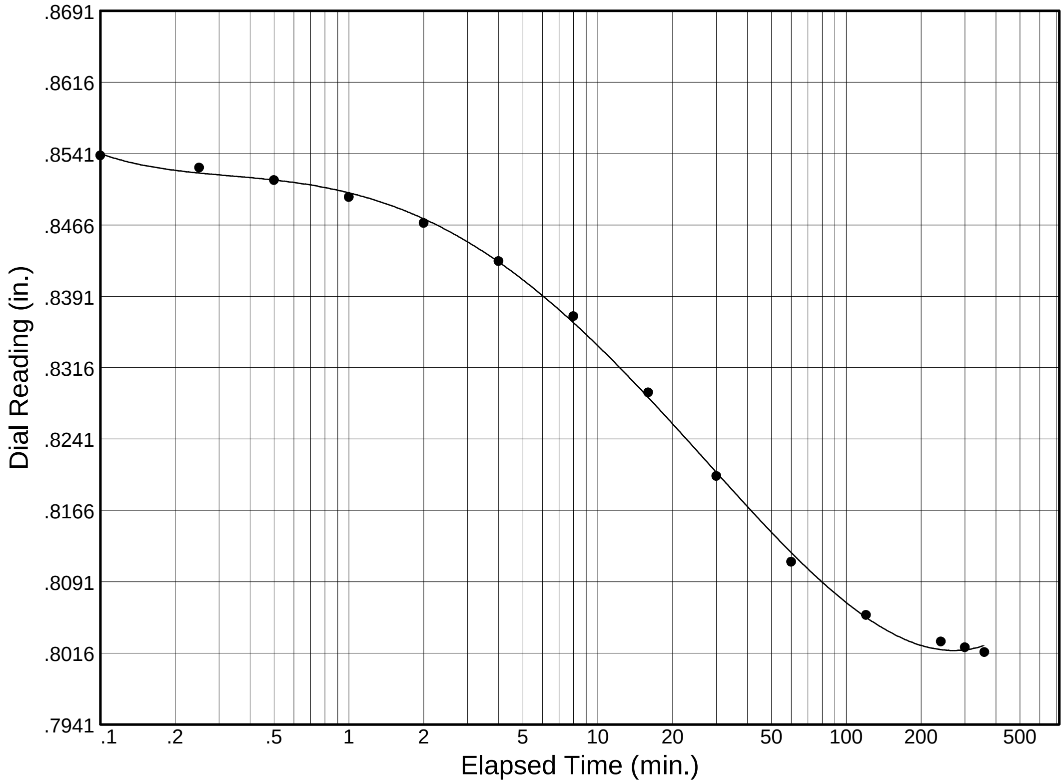
Figure

GET Solutions, Inc.

Dial Reading vs. Time

Project No.: WM13-136G
Project: Hopewell Regional Alternative 4A-1 Light Phase 2 PER

Location: M-3 (28-30 ft.) obtained 7/30/13



Load No.= 13

Load= 8.00 tsf

D_0 = 0.85596

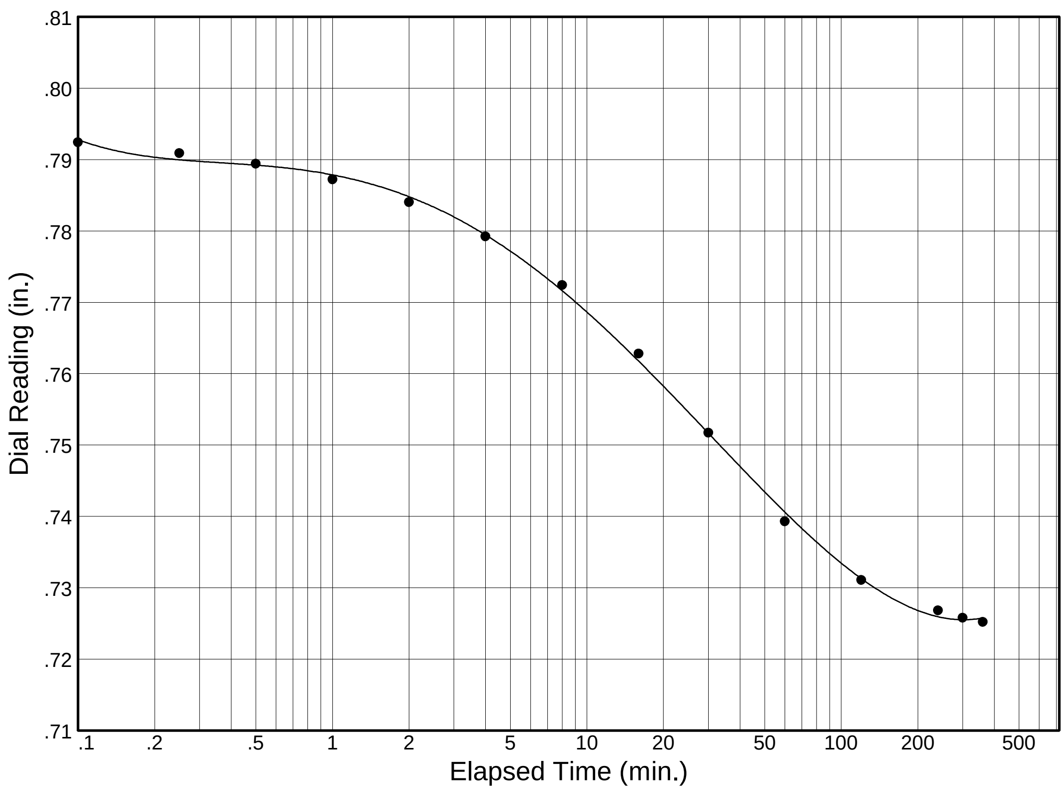
D_{50} = 0.82903

D_{100} = 0.80210

T_{50} = 15.25 min.

C_v @ T_{50}
0.02 $ft.^2$/day

C_α = 0.000



Load No.= 14

Load= 16.00 tsf

D_0 = 0.79488

D_{50} = 0.76037

D_{100} = 0.72587

T_{50} = 17.47 min.

C_v @ T_{50}
0.02 $ft.^2$/day

C_α = 0.004

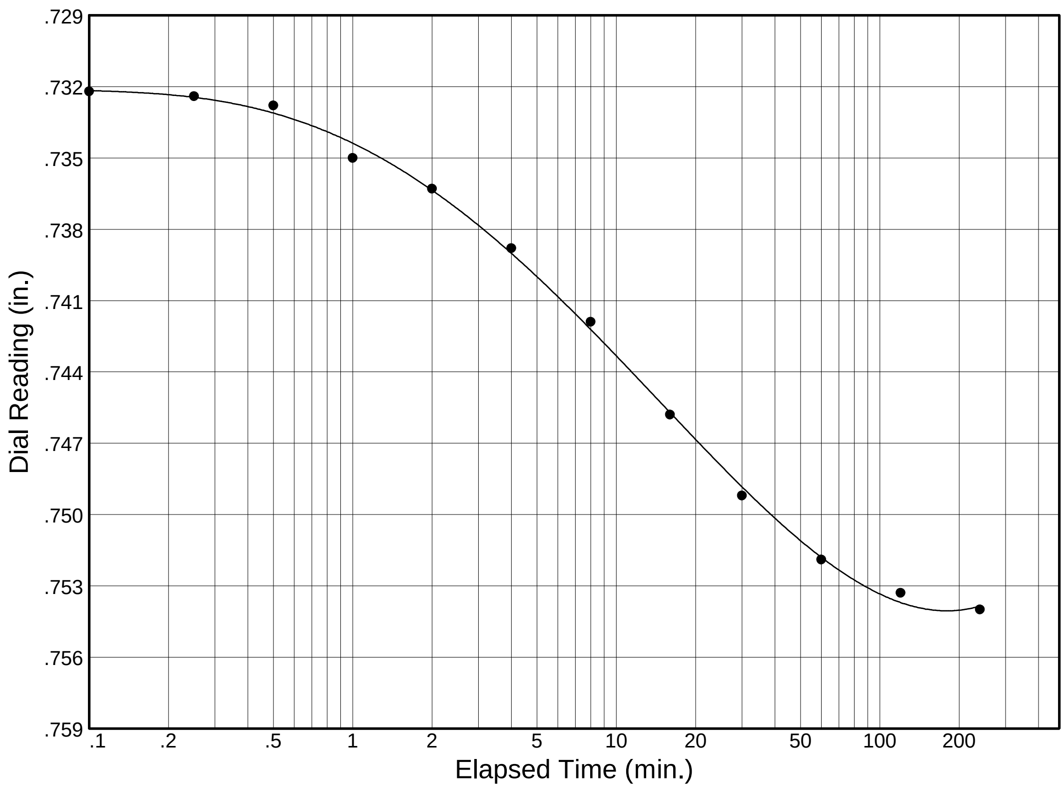
Figure

GET Solutions, Inc.

Dial Reading vs. Time

Project No.: WM13-136G
Project: Hopewell Regional Alternative 4A-1 Light Phase 2 PER

Location: M-3 (28-30 ft.) obtained 7/30/13



Load No.= 15

Load= 4.00 tsf

D_0 = 0.73149

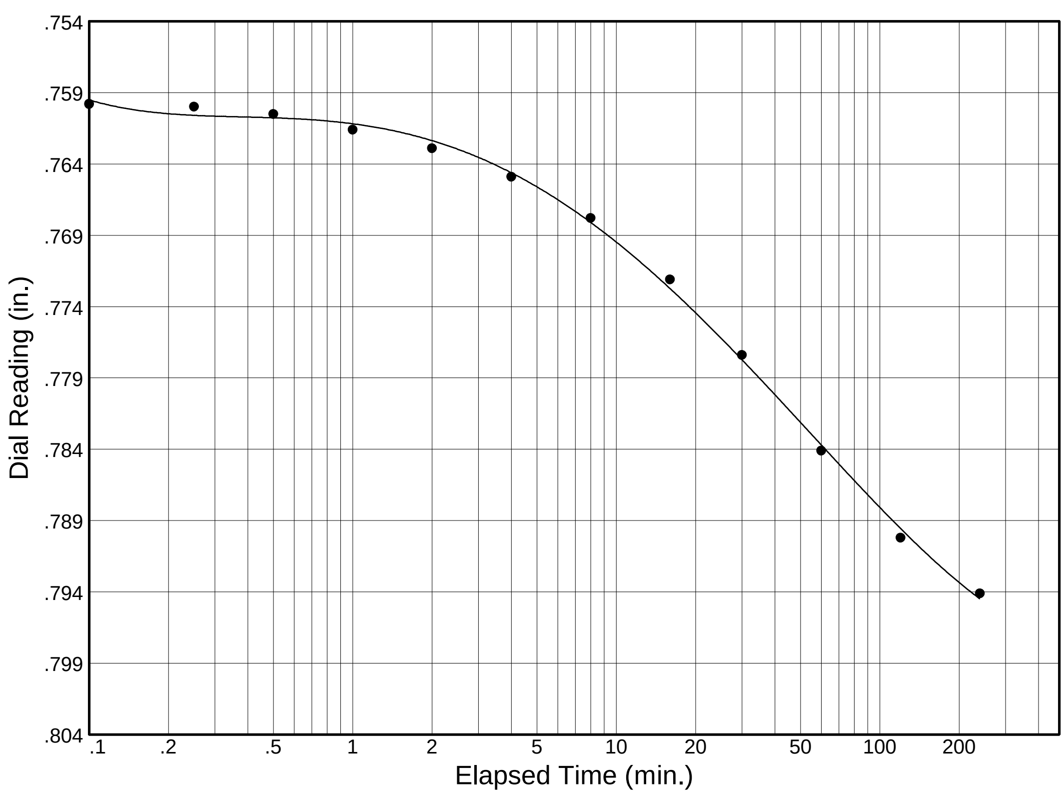
D_{50} = 0.74264

D_{100} = 0.75379

T_{50} = 8.72 min.

C_v @ T_{50}

0.03 ft.^{2}/day



Load No.= 16

Load= 1.00 tsf

D_0 = 0.75806

D_{50} = 0.77612

D_{100} = 0.79419

T_{50} = 24.69 min.

C_v @ T_{50}

0.01 ft.^{2}/day

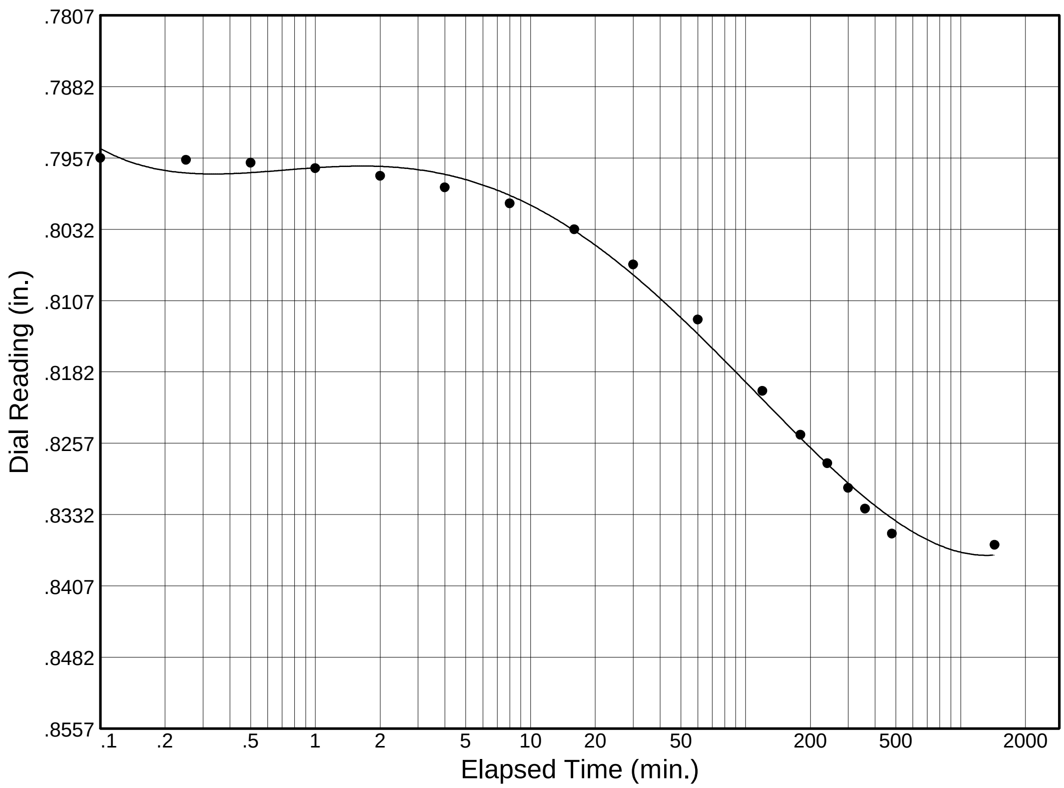
Figure

GET Solutions, Inc.

Dial Reading vs. Time

Project No.: WM13-136G
Project: Hopewell Regional Alternative 4A-1 Light Phase 2 PER

Location: M-3 (28-30 ft.) obtained 7/30/13



Load No.= 17

Load= 0.25 tsf

D_0 = 0.79112

D_{50} = 0.81442

D_{100} = 0.83772

T_{50} = 61.31 min.

C_v @ T_{50}

0.01 ft.^{2}/day

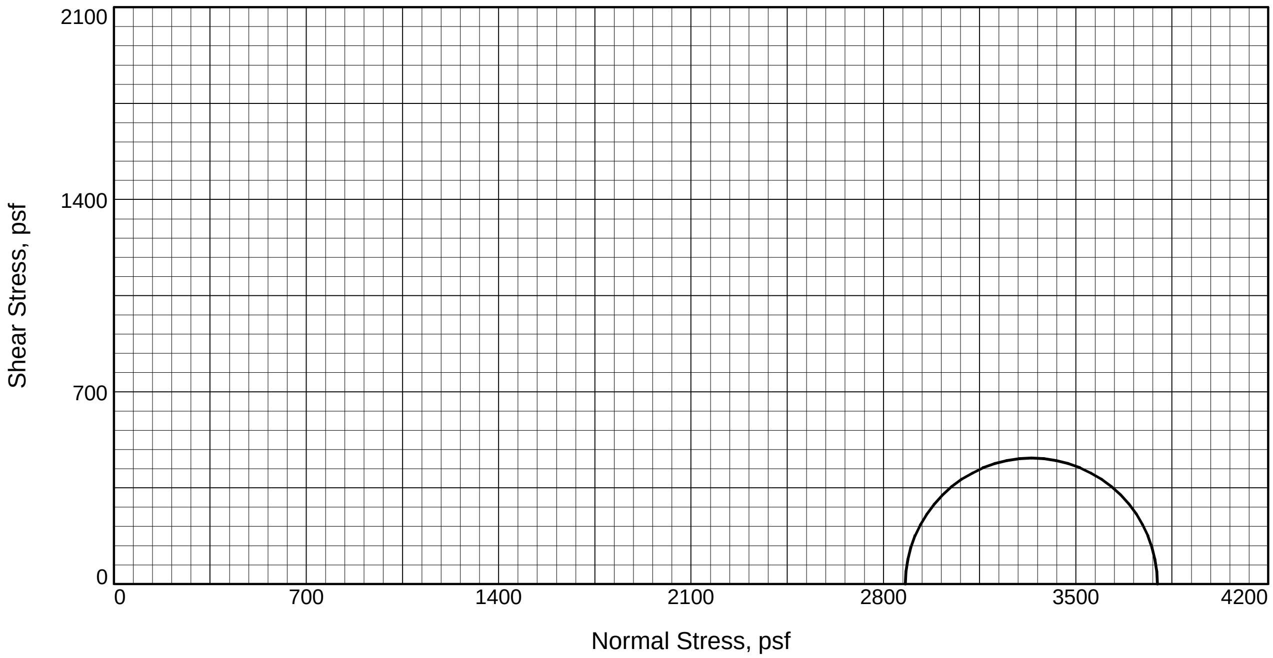
C_α = 0.001

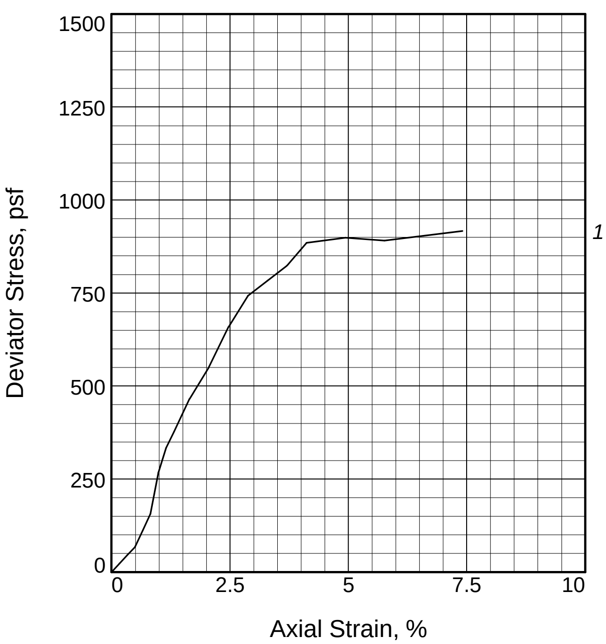
Figure

GET Solutions, Inc.

APPENDIX VI

TRIAXIAL COMPRESSION TEST RESULTS





Sample No.		1
Initial	Water Content, %	48.6
	Dry Density, pcf	71.6
	Saturation, %	96.5
	Void Ratio	1.3636
	Diameter, in.	2.854
	Height, in.	6.070
At Test	Water Content, %	46.5
	Dry Density, pcf	71.6
	Saturation, %	92.3
	Void Ratio	1.3636
	Diameter, in.	2.854
	Height, in.	6.070
Strain rate, in./min.		0.010
Back Pressure, psi		50.00
Cell Pressure, psi		70.00
Fail. Stress, psf		917
Ult. Stress, psf		
σ_1 Failure, psf		3797
σ_3 Failure, psf		2880

Type of Test:
Unconsolidated Undrained

Sample Type:

Description: Gray, Fat CLAY

LL= 63 **PL=** 27 **PI=** 36

Assumed Specific Gravity= 2.71

Remarks:

Figure ________

Client: HDR Engineering, Inc.

Project: Hopewell Regional Alternative 4A-1 Light Phase 2 PER

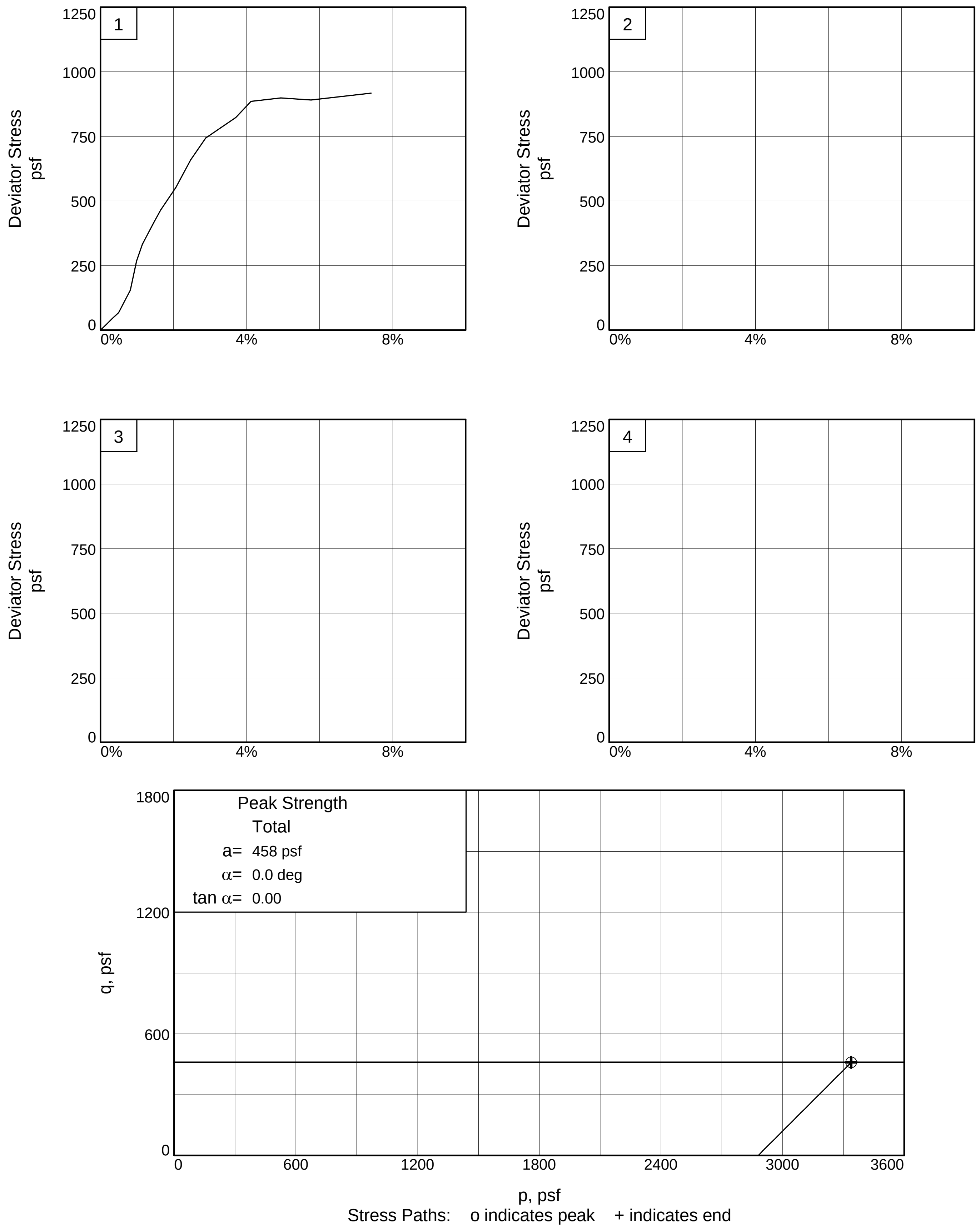
Location: M-3 (28-30 ft.) obtained 7/30/13

Sample Number: M-3 (28-30 ft.) **Depth:** 28-30 ft.

Proj. No.: WM13-136G **Date Sampled:** 7/30/13

TRIAXIAL SHEAR TEST REPORT

GET SOLUTIONS, INC.



Client: HDR Engineering, Inc.

Project: Hopewell Regional Alternative 4A-1 Light Phase 2 PER

Location: M-3 (28-30 ft.) obtained 7/30/13 **Depth:** 28-30 ft. **Sample Number:** M-3 (28-30 ft.)

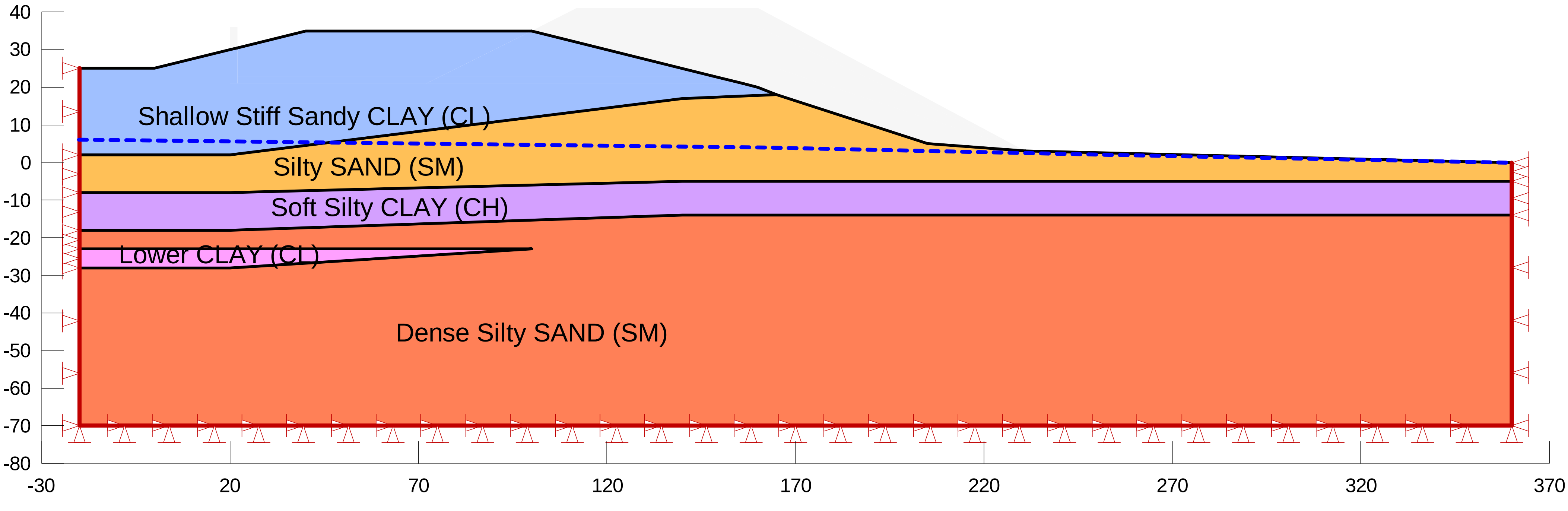
Project No.: WM13-136G **Figure** ________

GET Solutions, Inc.

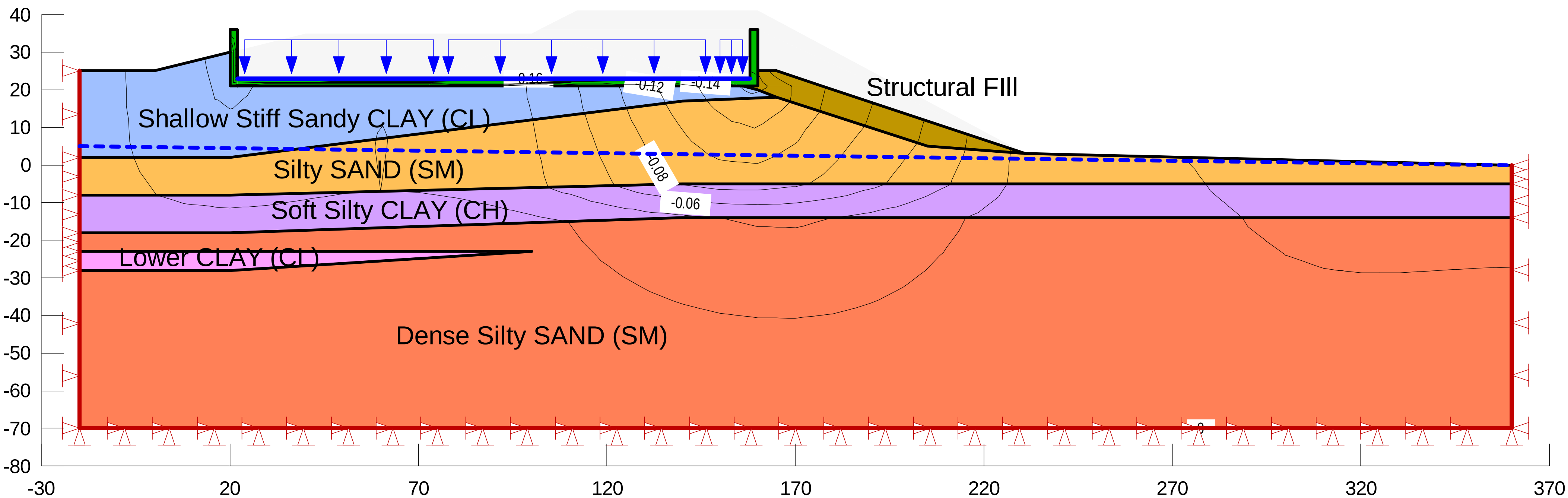
APPENDIX VII

STRESS-DEFORMATION FINITE ELEMENT ANALYSIS RESULTS

Existing Subsurface Profile



Loaded Tank Deformation Profile



Loaded Tank Deformation

File Information

Title: Hopewell - East Alignment
Created By: Bruce Spiro
Last Edited By: Bruce Spiro
Revision Number: 192
File Version: 8.2
Tool Version: 8.12.2.7663
Date: 10/23/2013
Time: 4:00:47 PM
File Name: Hopewell - East.gsz
Directory: C:\Users\bspiro\Documents\Hopewell\Analysis\East\
Last Solved Date: 10/23/2013
Last Solved Time: 4:01:47 PM

Project Settings

Length(L) Units: feet
Time(t) Units: Seconds
Force(F) Units: lbf
Pressure(p) Units: psf
Strength Units: psf
Stiffness Units: psf
Unit Weight of Water: 62.4 pcf
Air Pressure: 101.33 psf
View: 2D
Element Thickness: 1

Analysis Settings

Loaded Tank Deformation

Description: Tank at Elev 21
Kind: SIGMA/W
Parent: Earthwork Phase (Initial)
Method: Load/Deformation
Settings
- Initial Stress: Parent Analysis
- Initial PWP: Water Table
- Exclude cumulative values: No

Control
- Apply Body Force in All Steps: No

Adjust Fill: No
Convergence
Maximum Number of Iterations: 50
Minimum Displacement Difference: 0.001
Significant Digits: 3
Equation Solver: Parallel Direct
Time
Starting Time: 1 sec
Duration: 1 sec
# of Steps: 1
Save Steps Every: 1

Materials

Shallow Stiff Sandy CLAY (CL)

Model: Elastic-Plastic (Effective)
Stress Strain
Effective Young's Modulus (E'): 1,000,000 psf
Cohesion': 2,500 psf
Phi': 0 °
Poisson's Ratio: 0.25
Unit Weight: 125 pcf
Dilation Angle: 0 °

Silty SAND (SM)

Model: Elastic-Plastic (Effective)
Stress Strain
Effective Young's Modulus (E'): 900,000 psf
Cohesion': 0 psf
Phi': 38 °
Poisson's Ratio: 0.25
Unit Weight: 135 pcf
Dilation Angle: 0 °

Soft Silty CLAY (CH)

Model: Elastic-Plastic (Effective)
Stress Strain
Effective Young's Modulus (E'): 200,000 psf
Cohesion': 1,000 psf
Phi': 0 °
Poisson's Ratio: 0.35
Unit Weight: 117 pcf
Dilation Angle: 0 °

Dense Silty SAND (SM)

Model: Elastic-Plastic (Effective)

Stress Strain
Effective Young's Modulus (E'): 1,300,000 psf
Cohesion': 0 psf
Phi': 36 °
Poisson's Ratio: 0.25
Unit Weight: 135 pcf
Dilation Angle: 0 °

Concrete Tank

Model: Linear Elastic (Effective)
Stress Strain
Effective Young's Modulus (E'): 5.8e+008 psf
Unit Weight: 150 pcf
Poisson's Ratio: 0.15

Structural Fill

Model: Elastic-Plastic (Effective)
Stress Strain
Effective Young's Modulus (E'): 800,000 psf
Cohesion': 0 psf
Phi': 34 °
Poisson's Ratio: 0.25
Unit Weight: 125 pcf
Dilation Angle: 0 °

Lower CLAY (CL)

Model: Elastic-Plastic (Effective)
Stress Strain
Effective Young's Modulus (E'): 800,000 psf
Cohesion': 2,000 psf
Phi': 0 °
Poisson's Ratio: 0.3
Unit Weight: 120 pcf
Dilation Angle: 0 °

Boundary Conditions

Fixed X

X: X-Displacement 0

Fixed X/Y

X: X-Displacement 0
Y: Y-Displacement 0

Tank Load 2

X: X-Stress 0

Y: Y-Stress -1560

Initial Water Tables

Initial Water Table 1

Max. negative head: 0

Coordinates

Coordinate 1: (-20, 5) ft

Coordinate 2: (360, 0) ft

Points

	X (ft)	Y (ft)
Point 1	140	17
Point 2	22	23
Point 3	148	23
Point 4	22	30.5
Point 5	22	21
Point 6	158	21
Point 7	160	21
Point 8	160	25
Point 9	158	25
Point 10	20	2
Point 11	20	-8
Point 12	20	-18
Point 13	20	-23
Point 14	20	-28
Point 15	-20	-8
Point 16	-20	-18
Point 17	-20	-23
Point 18	-20	-28
Point 19	140	-5
Point 20	165	25
Point 21	231	3
Point 22	205	5
Point 23	160	20
Point 24	156	21

Point 25	158	23
Point 26	20	30
Point 27	22	36
Point 28	20	36
Point 29	160	36
Point 30	158	36
Point 31	140	-14
Point 32	360	-5
Point 33	360	-14
Point 34	-20	2
Point 35	-20	25
Point 36	0	25
Point 37	20	21
Point 38	40	35
Point 39	100	35
Point 40	100	-23
Point 41	165	18
Point 42	360	0
Point 43	360	-70
Point 44	-20	-70
Point 45	72	21
Point 46	76	23
Point 47	102	36
Point 48	112	41
Point 49	160	41
Point 50	177	21

Lines

	Start Point	End Point	Stress/Strain Boundary
Line 1	39	3	
Line 2	2	4	
Line 3	26	4	
Line 4	2	5	
Line 5	5	37	
Line 6	37	26	

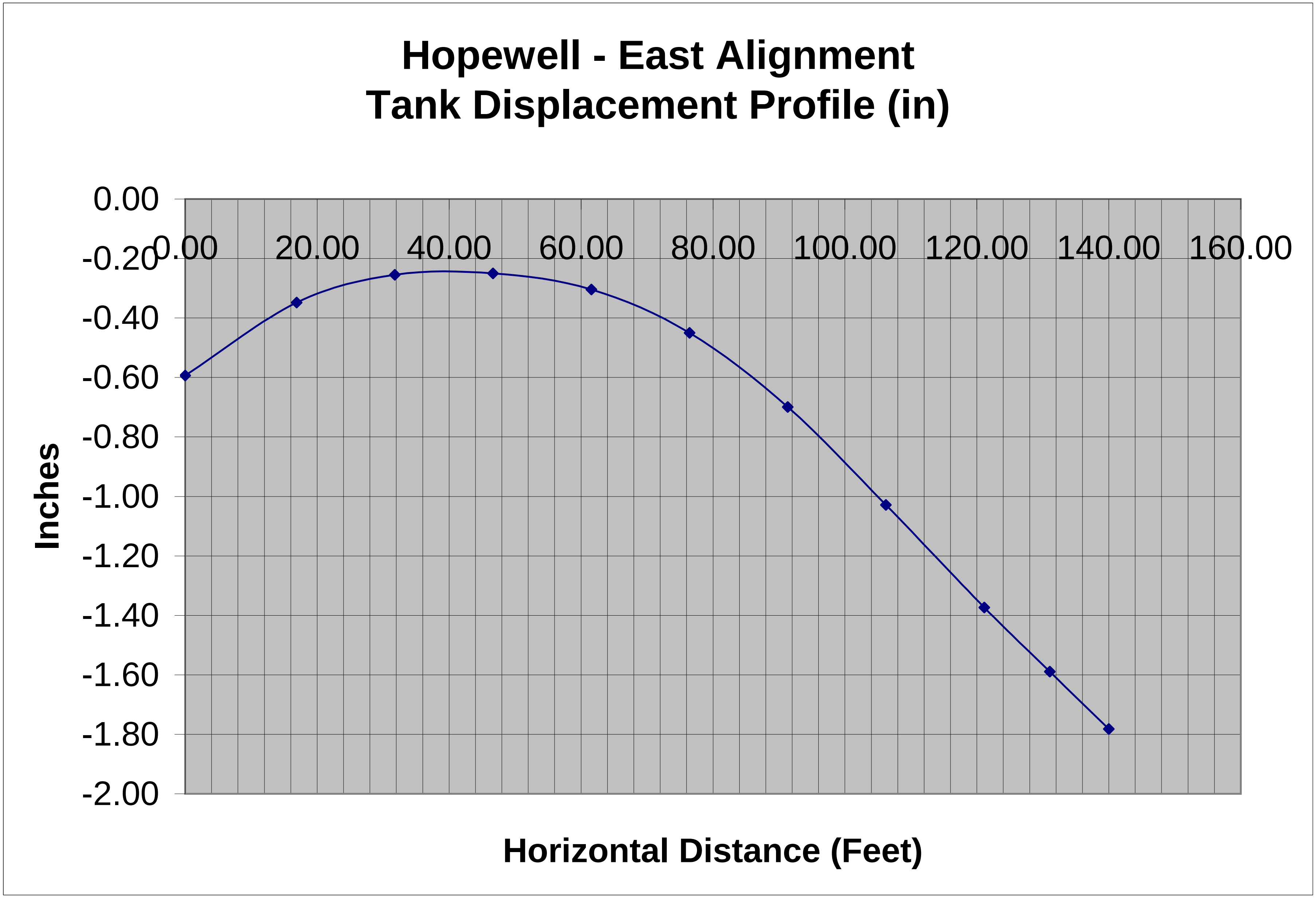
Line 7	3	24	
Line 8	3	25	Tank Load 2
Line 9	25	6	
Line 10	6	24	
Line 11	25	9	
Line 12	9	30	
Line 13	30	29	
Line 14	29	8	
Line 15	8	7	
Line 16	7	6	
Line 17	35	36	
Line 18	36	26	
Line 19	24	23	
Line 20	23	41	
Line 21	41	1	
Line 22	1	10	
Line 23	10	34	
Line 24	34	35	Fixed X
Line 25	8	20	
Line 26	21	22	
Line 27	22	41	
Line 28	21	42	
Line 29	42	32	Fixed X
Line 30	32	19	
Line 31	19	11	
Line 32	11	15	
Line 33	15	34	Fixed X
Line 34	16	15	Fixed X
Line 35	32	33	Fixed X
Line 36	33	31	
Line 37	31	12	
Line 38	12	16	
Line 39	17	13	
Line 40	13	40	
Line 41	40	14	
Line 42	14	18	

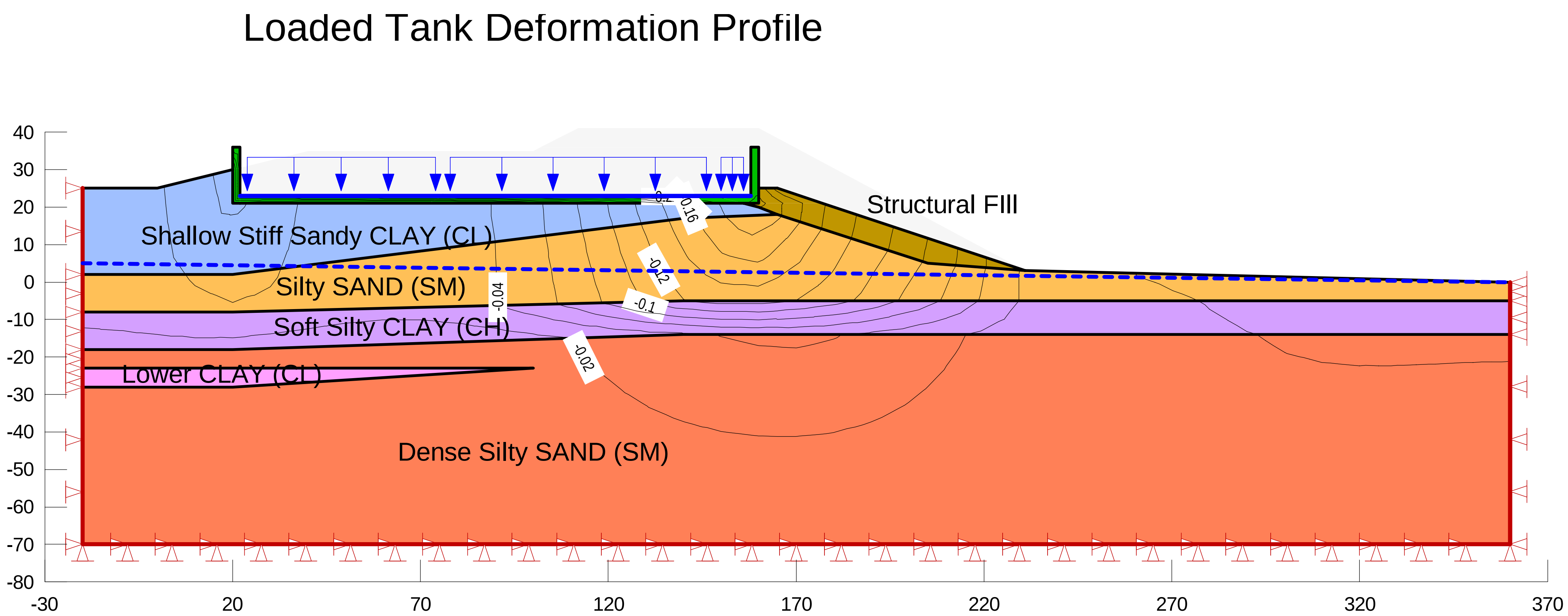
Line 43	18	17	Fixed X
Line 44	33	43	Fixed X
Line 45	43	44	Fixed X/Y
Line 46	44	18	Fixed X
Line 47	17	16	Fixed X
Line 48	28	27	
Line 49	27	4	
Line 50	26	28	
Line 51	24	45	
Line 52	45	5	
Line 53	39	47	
Line 54	47	30	
Line 55	21	29	
Line 56	47	48	
Line 57	48	49	
Line 58	49	21	
Line 59	2	46	Tank Load 2
Line 60	46	45	
Line 61	3	46	Tank Load 2
Line 62	39	46	
Line 63	4	38	
Line 64	38	39	
Line 65	20	50	
Line 66	50	21	
Line 67	50	7	

Regions

	Material	Points	Area (ft²)
Region 1	Concrete Tank	26,4,2,5,37	18.5
Region 2	Concrete Tank	3,25,6,24	12
Region 3	Concrete Tank	6,25,9,30,29,8,7	30
Region 4	Shallow Stiff Sandy CLAY (CL)	35,36,26,37,5,45,24,23,41,1,10,34	2,425.5
Region 5	Silty SAND (SM)	34,10,1,41,22,21,42,32,19,11,15	4,615
Region 6	Soft Silty CLAY (CH)	16,15,11,19,32,33,31,12	3,520
Region 7	Lower CLAY (CL)	17,13,40,14,18	400

Region 8	Dense Silty SAND (SM)	16,12,31,33,43,44,18,14,40,13,17	20,480
Region 9	Concrete Tank	28,27,4,26	11.5
Region 10		39,47,30,9,25,3	465
Region 11		29,8,20,50,21	335.5
Region 12		47,48,49,21,29,30	442.5
Region 13	Concrete Tank	2,46,45,5	104
Region 14	Concrete Tank	46,45,24,3	156
Region 15		46,3,39	432
Region 16		4,38,39,46,2	751.5
Region 17	Structural Fill	7,8,20,50	44
Region 18	Structural Fill	24,6,7,50,21,22,41,23	348


Hopewell - East Alignment
Tank Displacement Profile (in)
0.00
-0.20
-0.40
-0.60
-0.80
-1.00
-1.20
-1.40
-1.60
-1.80
-2.00
0.00
20.00
40.00
60.00
80.00
100.00
120.00
140.00
160.00
Inches
Horizontal Distance (Feet)



Loaded Tank Deformation Profile
Structural FIll
Shallow Stiff Sandy CLAY (CL)
Silty SAND (SM)
Soft Silty CLAY (CH)
Lower CLAY (CL)
Dense Silty SAND (SM)
-0.2
-0.16
-0.12
-0.1
-0.04
-0.02
40
30
20
10
0
-10
-20
-30
-40
-50
-60
-70
-80
-30
20
70
120
170
220
270
320
370

Loaded Tank Deformation

File Information

Title: Hopewell - East Alignment
Created By: Bruce Spiro
Last Edited By: Bruce Spiro
Revision Number: 190
File Version: 8.2
Tool Version: 8.12.2.7663
Date: 10/23/2013
Time: 4:37:00 PM
File Name: Hopewell - East 2.gsz
Directory: C:\Users\bspiro\Documents\Hopewell\Analysis\East\
Last Solved Date: 10/23/2013
Last Solved Time: 4:37:57 PM

Project Settings

Length(L) Units: feet
Time(t) Units: Seconds
Force(F) Units: lbf
Pressure(p) Units: psf
Strength Units: psf
Stiffness Units: psf
Unit Weight of Water: 62.4 pcf
Air Pressure: 101.33 psf
View: 2D
Element Thickness: 1

Analysis Settings

Loaded Tank Deformation

Description: Tank at Elev 21
Kind: SIGMA/W
Parent: Earthwork Phase (Initial)
Method: Load/Deformation
Settings
- Initial Stress: Parent Analysis
- Initial PWP: Water Table
- Exclude cumulative values: No

Control
- Apply Body Force in All Steps: No

Adjust Fill: No
Convergence
Maximum Number of Iterations: 50
Minimum Displacement Difference: 0.001
Significant Digits: 3
Equation Solver: Parallel Direct
Time
Starting Time: 1 sec
Duration: 1 sec
# of Steps: 1
Save Steps Every: 1

Materials

Shallow Stiff Sandy CLAY (CL)

Model: Elastic-Plastic (Effective)
Stress Strain
Effective Young's Modulus (E'): 900,000 psf
Cohesion': 2,500 psf
Phi': 0 °
Poisson's Ratio: 0.25
Unit Weight: 125 pcf
Dilation Angle: 0 °

Silty SAND (SM)

Model: Elastic-Plastic (Effective)
Stress Strain
Effective Young's Modulus (E'): 800,000 psf
Cohesion': 0 psf
Phi': 38 °
Poisson's Ratio: 0.25
Unit Weight: 135 pcf
Dilation Angle: 0 °

Soft Silty CLAY (CH)

Model: Elastic-Plastic (Effective)
Stress Strain
Effective Young's Modulus (E'): 107,500 psf
Cohesion': 1,000 psf
Phi': 0 °
Poisson's Ratio: 0.35
Unit Weight: 117 pcf
Dilation Angle: 0 °

Dense Silty SAND (SM)

Model: Elastic-Plastic (Effective)

Stress Strain

Effective Young's Modulus (E'): 1,300,000 psf

Cohesion': 0 psf

Phi': 36 °

Poisson's Ratio: 0.25

Unit Weight: 135 pcf

Dilation Angle: 0 °

Concrete Tank

Model: Linear Elastic (Effective)

Stress Strain

Effective Young's Modulus (E'): 5.8e+008 psf

Unit Weight: 150 pcf

Poisson's Ratio: 0.15

Structural Fill

Model: Elastic-Plastic (Effective)

Stress Strain

Effective Young's Modulus (E'): 800,000 psf

Cohesion': 0 psf

Phi': 34 °

Poisson's Ratio: 0.25

Unit Weight: 125 pcf

Dilation Angle: 0 °

Lower CLAY (CL)

Model: Elastic-Plastic (Effective)

Stress Strain

Effective Young's Modulus (E'): 800,000 psf

Cohesion': 2,000 psf

Phi': 0 °

Poisson's Ratio: 0.3

Unit Weight: 120 pcf

Dilation Angle: 0 °

Boundary Conditions

Fixed X

X: X-Displacement 0

Fixed X/Y

X: X-Displacement 0

Y: Y-Displacement 0

Tank Load 2

X: X-Stress 0

Y: Y-Stress -1560

Initial Water Tables

Initial Water Table 1

Max. negative head: 0

Coordinates

Coordinate 1: (-20, 5) ft

Coordinate 2: (360, 0) ft

Points

	X (ft)	Y (ft)
Point 1	140	17
Point 2	22	23
Point 3	148	23
Point 4	22	30.5
Point 5	22	21
Point 6	158	21
Point 7	160	21
Point 8	160	25
Point 9	158	25
Point 10	20	2
Point 11	20	-8
Point 12	20	-18
Point 13	20	-23
Point 14	20	-28
Point 15	-20	-8
Point 16	-20	-18
Point 17	-20	-23
Point 18	-20	-28
Point 19	140	-5
Point 20	165	25
Point 21	231	3
Point 22	205	5
Point 23	160	20
Point 24	156	21

Point 25	158	23
Point 26	20	30
Point 27	22	36
Point 28	20	36
Point 29	160	36
Point 30	158	36
Point 31	140	-14
Point 32	360	-5
Point 33	360	-14
Point 34	-20	2
Point 35	-20	25
Point 36	0	25
Point 37	20	21
Point 38	40	35
Point 39	100	35
Point 40	100	-23
Point 41	165	18
Point 42	360	0
Point 43	360	-70
Point 44	-20	-70
Point 45	72	21
Point 46	76	23
Point 47	102	36
Point 48	112	41
Point 49	160	41
Point 50	177	21

Lines

	Start Point	End Point	Stress/Strain Boundary
Line 1	39	3	
Line 2	2	4	
Line 3	26	4	
Line 4	2	5	
Line 5	5	37	
Line 6	37	26	

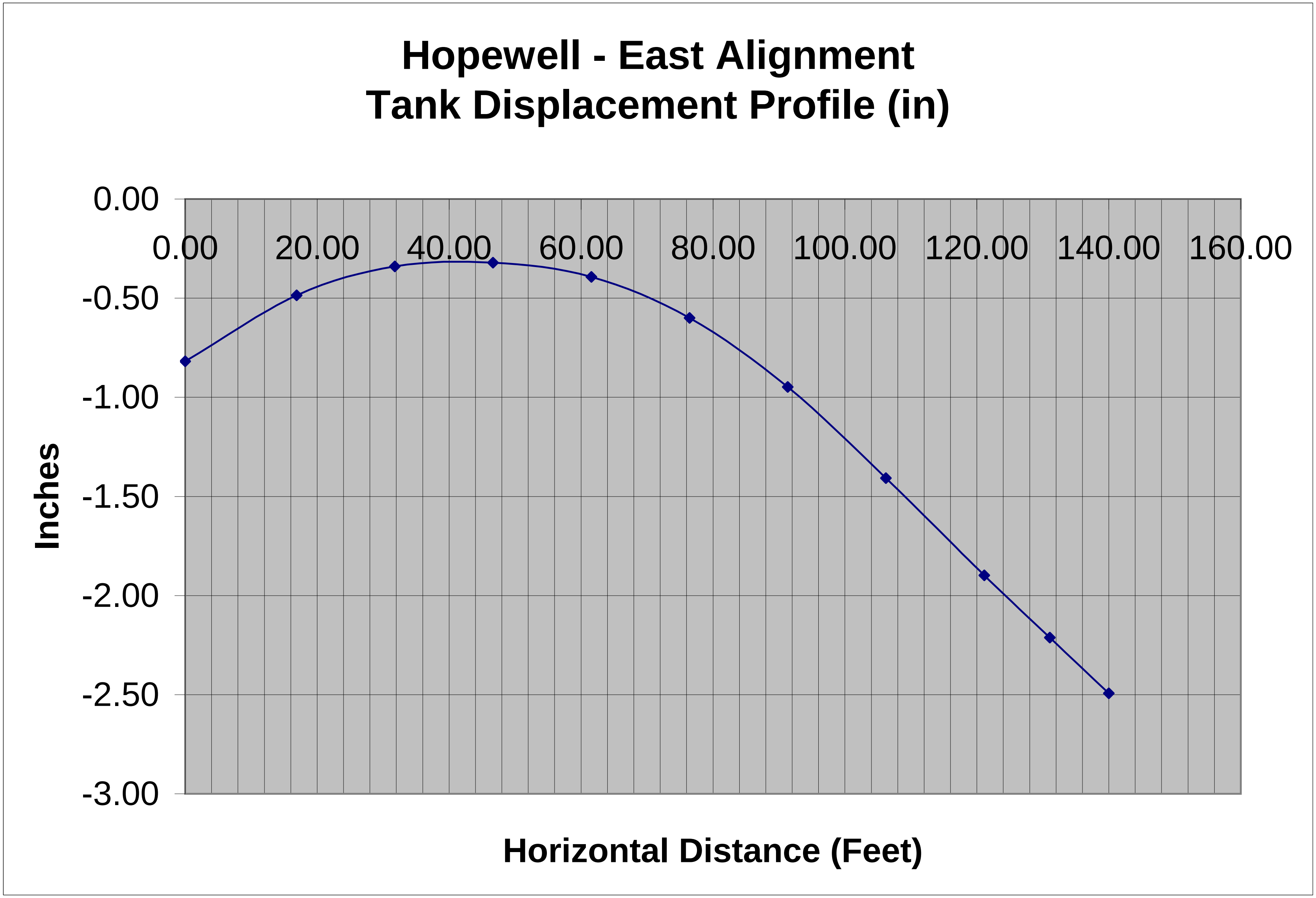
Line 7	3	24	
Line 8	3	25	Tank Load 2
Line 9	25	6	
Line 10	6	24	
Line 11	25	9	
Line 12	9	30	
Line 13	30	29	
Line 14	29	8	
Line 15	8	7	
Line 16	7	6	
Line 17	35	36	
Line 18	36	26	
Line 19	24	23	
Line 20	23	41	
Line 21	41	1	
Line 22	1	10	
Line 23	10	34	
Line 24	34	35	Fixed X
Line 25	8	20	
Line 26	21	22	
Line 27	22	41	
Line 28	21	42	
Line 29	42	32	Fixed X
Line 30	32	19	
Line 31	19	11	
Line 32	11	15	
Line 33	15	34	Fixed X
Line 34	16	15	Fixed X
Line 35	32	33	Fixed X
Line 36	33	31	
Line 37	31	12	
Line 38	12	16	
Line 39	17	13	
Line 40	13	40	
Line 41	40	14	
Line 42	14	18	

Line 43	18	17	Fixed X
Line 44	33	43	Fixed X
Line 45	43	44	Fixed X/Y
Line 46	44	18	Fixed X
Line 47	17	16	Fixed X
Line 48	28	27	
Line 49	27	4	
Line 50	26	28	
Line 51	24	45	
Line 52	45	5	
Line 53	39	47	
Line 54	47	30	
Line 55	21	29	
Line 56	47	48	
Line 57	48	49	
Line 58	49	21	
Line 59	2	46	Tank Load 2
Line 60	46	45	
Line 61	3	46	Tank Load 2
Line 62	39	46	
Line 63	4	38	
Line 64	38	39	
Line 65	20	50	
Line 66	50	21	
Line 67	50	7	

Regions

	Material	Points	Area (ft^2)
Region 1	Concrete Tank	26,4,2,5,37	18.5
Region 2	Concrete Tank	3,25,6,24	12
Region 3	Concrete Tank	6,25,9,30,29,8,7	30
Region 4	Shallow Stiff Sandy CLAY (CL)	35,36,26,37,5,45,24,23,41,1,10,34	2,425.5
Region 5	Silty SAND (SM)	34,10,1,41,22,21,42,32,19,11,15	4,615
Region 6	Soft Silty CLAY (CH)	16,15,11,19,32,33,31,12	3,520
Region 7	Lower CLAY (CL)	17,13,40,14,18	400

Region 8	Dense Silty SAND (SM)	16,12,31,33,43,44,18,14,40,13,17	20,480
Region 9	Concrete Tank	28,27,4,26	11.5
Region 10		39,47,30,9,25,3	465
Region 11		29,8,20,50,21	335.5
Region 12		47,48,49,21,29,30	442.5
Region 13	Concrete Tank	2,46,45,5	104
Region 14	Concrete Tank	46,45,24,3	156
Region 15		46,3,39	432
Region 16		4,38,39,46,2	751.5
Region 17	Structural Fill	7,8,20,50	44
Region 18	Structural Fill	24,6,7,50,21,22,41,23	348

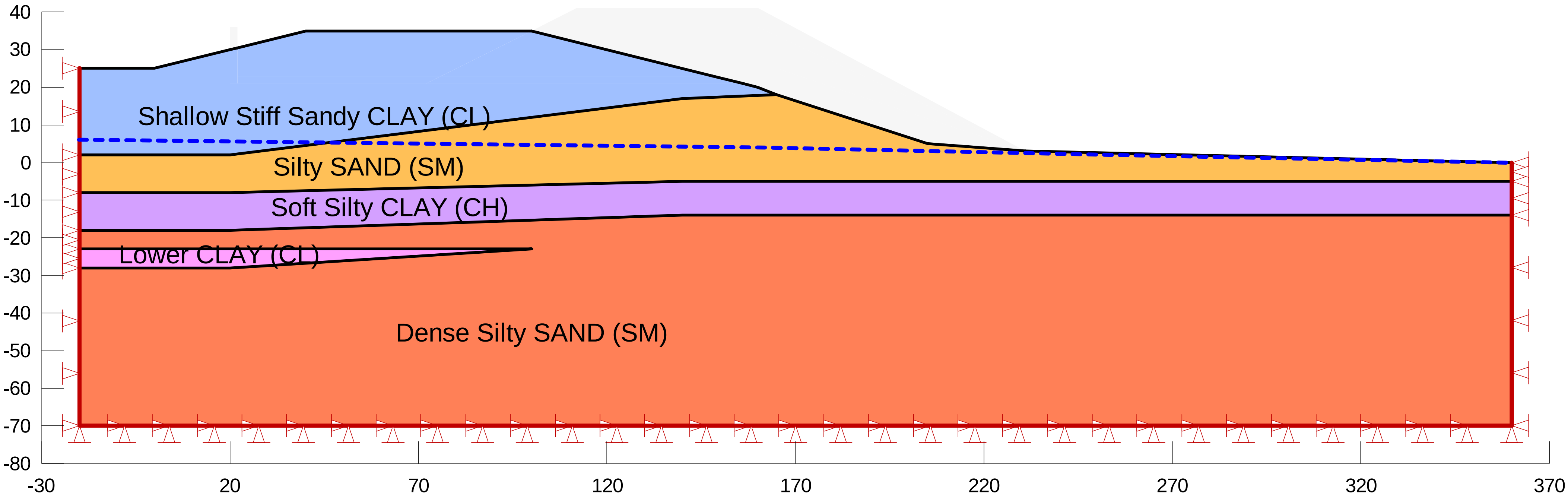


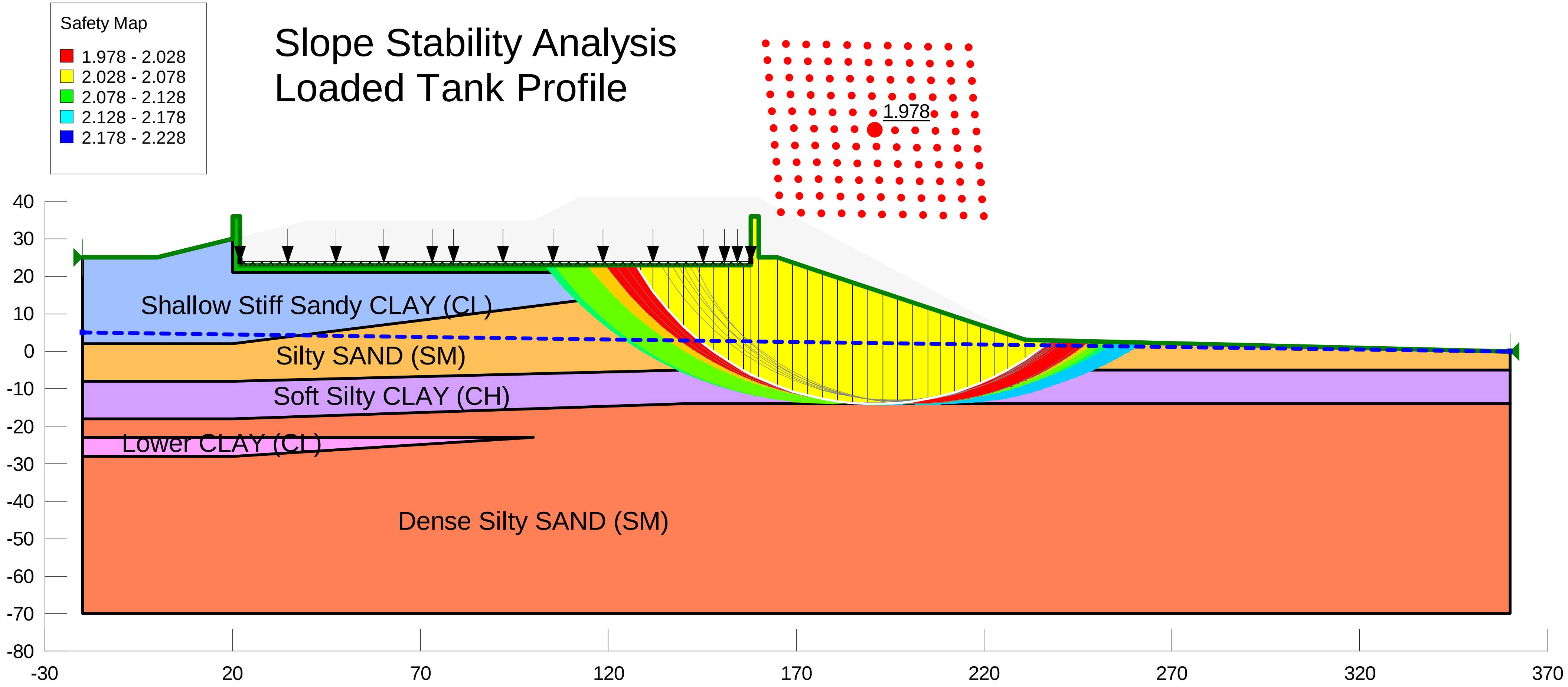
Hopewell - East Alignment
Tank Displacement Profile (in)
0.00
-0.50
-1.00
-1.50
-2.00
-2.50
-3.00
0.00
20.00
40.00
60.00
80.00
100.00
120.00
140.00
160.00
Inches
Horizontal Distance (Feet)

APPENDIX VIII

SLOPE STABILITY ANALYSIS RESULTS

Existing Subsurface Profile





Safety Map
1.978 - 2.028
2.028 - 2.078
2.078 - 2.128
2.128 - 2.178
2.178 - 2.228
Slope Stability Analysis
Loaded Tank Profile
1.978
Shallow Stiff Sandy CLAY (CL)
Silty SAND (SM)
Soft Silty CLAY (CH)
Lower CLAY (CL)
Dense Silty SAND (SM)
40
30
20
10
0
-10
-20
-30
-40
-50
-60
-70
-80
-30
20
70
120
170
220
270
320
370

Loaded Tank Slope Stability (Grid)

File Information

Title: Hopewell - East Alignment
Created By: Bruce Spiro
Last Edited By: Bruce Spiro
Revision Number: 192
File Version: 8.2
Tool Version: 8.12.2.7663
Date: 10/23/2013
Time: 4:00:47 PM
File Name: Hopewell - East.gsz
Directory: C:\Users\bspiro\Documents\Hopewell\Analysis\East\
Last Solved Date: 10/23/2013
Last Solved Time: 4:01:06 PM

Project Settings

Length(L) Units: feet
Time(t) Units: Seconds
Force(F) Units: lbf
Pressure(p) Units: psf
Strength Units: psf
Unit Weight of Water: 62.4 pcf
View: 2D
Element Thickness: 1

Analysis Settings

Loaded Tank Slope Stability (Grid)

Description: Tank at Elev 21
Kind: SLOPE/W
Parent: Loaded Tank Deformation
Method: Morgenstern-Price
Settings
  Side Function
    Interslice force function option: Half-Sine
  Lambda
    Lambda 1: -1
    Lambda 2: -0.8
    Lambda 3: -0.6
    Lambda 4: -0.4

Lambda 5: -0.2
Lambda 6: 0
Lambda 7: 0.2
Lambda 8: 0.4
Lambda 9: 0.6
Lambda 10: 0.8
Lambda 11: 1
PWP Conditions Source: Piezometric Line
Apply Phreatic Correction: No
Use Staged Rapid Drawdown: No
Slip Surface
Direction of movement: Left to Right
Use Passive Mode: No
Slip Surface Option: Grid and Radius
Critical slip surfaces saved: 10
Optimize Critical Slip Surface Location: No
Tension Crack
Tension Crack Option: (none)
F of S Distribution
F of S Calculation Option: Constant
Advanced
Number of Slices: 30
F of S Tolerance: 0.001
Minimum Slip Surface Depth: 0.1 ft
Optimization Maximum Iterations: 2,000
Optimization Convergence Tolerance: 1e-007
Starting Optimization Points: 8
Ending Optimization Points: 16
Complete Passes per Insertion: 1
Driving Side Maximum Convex Angle: 5 °
Resisting Side Maximum Convex Angle: 1 °

Materials

Shallow Stiff Sandy CLAY (CL)

Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion': 2,500 psf
Phi': 0 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Silty SAND (SM)

Model: Mohr-Coulomb
Unit Weight: 135 pcf

Cohesion': 0 psf
Phi': 38 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Soft Silty CLAY (CH)

Model: Mohr-Coulomb
Unit Weight: 117 pcf
Cohesion': 1,000 psf
Phi': 0 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Dense Silty SAND (SM)

Model: Mohr-Coulomb
Unit Weight: 135 pcf
Cohesion': 0 psf
Phi': 36 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Concrete Tank

Model: Mohr-Coulomb
Unit Weight: 150 pcf
Cohesion': 0 psf
Phi': 50 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Structural Fill

Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion': 0 psf
Phi': 34 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Lower CLAY (CL)

Model: Mohr-Coulomb
Unit Weight: 120 pcf
Cohesion': 2,000 psf
Phi': 0 °
Phi-B: 0 °

Pore Water Pressure
Piezometric Line: 1

Slip Surface Grid

Upper Left: (162, 82) ft
Lower Left: (166, 37) ft
Lower Right: (220, 36) ft
Grid Horizontal Increment: 10
Grid Vertical Increment: 10
Left Projection Angle: 0 °
Right Projection Angle: 0 °

Slip Surface Radius

Upper Left Coordinate: (143, 19) ft
Upper Right Coordinate: (226, 1) ft
Lower Left Coordinate: (143, -22) ft
Lower Right Coordinate: (223, -31) ft
Number of Increments: 20
Left Projection: No
Left Projection Angle: 135 °
Right Projection: No
Right Projection Angle: 45 °

Slip Surface Limits

Left Coordinate: (-20, 25) ft
Right Coordinate: (360, 0) ft

Piezometric Lines

Piezometric Line 1

Coordinates

	X (ft)	Y (ft)
Coordinate 1	-20	5
Coordinate 2	360	0

Surcharge Loads

Surcharge Load 1

Surcharge (Unit Weight): 1,560 pcf
Direction: Vertical

Coordinates

	X (ft)	Y (ft)
	22	24
	158	24

Points

	X (ft)	Y (ft)
Point 1	140	17
Point 2	22	23
Point 3	148	23
Point 4	22	30.5
Point 5	22	21
Point 6	158	21
Point 7	160	21
Point 8	160	25
Point 9	158	25
Point 10	20	2
Point 11	20	-8
Point 12	20	-18
Point 13	20	-23
Point 14	20	-28
Point 15	-20	-8
Point 16	-20	-18
Point 17	-20	-23
Point 18	-20	-28
Point 19	140	-5
Point 20	165	25
Point 21	231	3
Point 22	205	5
Point 23	160	20

Point 24	156	21
Point 25	158	23
Point 26	20	30
Point 27	22	36
Point 28	20	36
Point 29	160	36
Point 30	158	36
Point 31	140	-14
Point 32	360	-5
Point 33	360	-14
Point 34	-20	2
Point 35	-20	25
Point 36	0	25
Point 37	20	21
Point 38	40	35
Point 39	100	35
Point 40	100	-23
Point 41	165	18
Point 42	360	0
Point 43	360	-70
Point 44	-20	-70
Point 45	72	21
Point 46	76	23
Point 47	102	36
Point 48	112	41
Point 49	160	41
Point 50	177	21

Regions

	Material	Points	Area (ft^2)
Region 1	Concrete Tank	26,4,2,5,37	18.5
Region 2	Concrete Tank	3,25,6,24	12
Region 3	Concrete Tank	6,25,9,30,29,8,7	30
Region 4	Shallow Stiff Sandy CLAY (CL)	35,36,26,37,5,45,24,23,41,1,10,34	2,425.5
Region 5	Silty SAND (SM)	34,10,1,41,22,21,42,32,19,11,15	4,615

Region 6	Soft Silty CLAY (CH)	16,15,11,19,32,33,31,12	3,520
Region 7	Lower CLAY (CL)	17,13,40,14,18	400
Region 8	Dense Silty SAND (SM)	16,12,31,33,43,44,18,14,40,13,17	20,480
Region 9	Concrete Tank	28,27,4,26	11.5
Region 10		39,47,30,9,25,3	465
Region 11		29,8,20,50,21	335.5
Region 12		47,48,49,21,29,30	442.5
Region 13	Concrete Tank	2,46,45,5	104
Region 14	Concrete Tank	46,45,24,3	156
Region 15		46,3,39	432
Region 16		4,38,39,46,2	751.5
Region 17	Structural Fill	7,8,20,50	44
Region 18	Structural Fill	24,6,7,50,21,22,41,23	348

Current Slip Surface

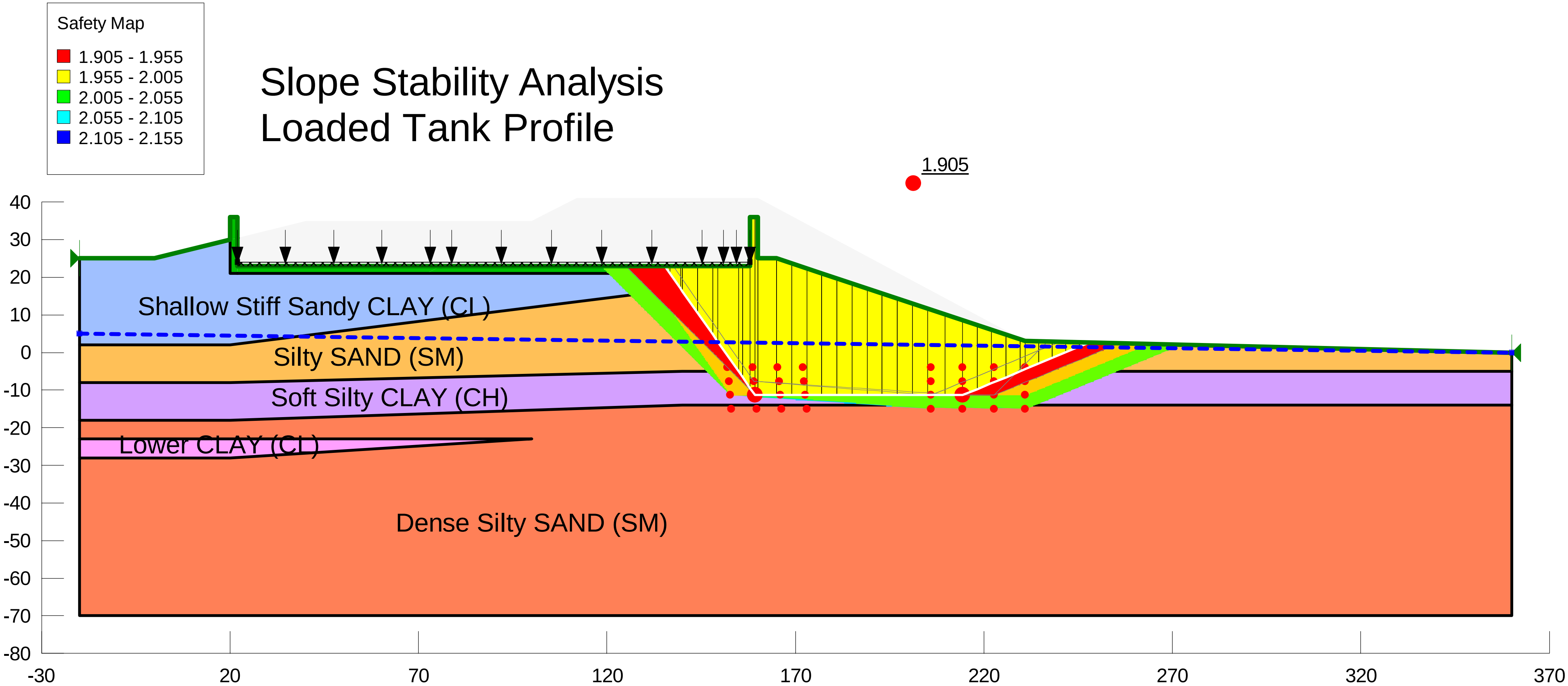
Slip Surface: 1,274
F of S: 1.978
Volume: 2,454.661 ft³
Weight: 319,909.81 lbs
Resisting Moment: 12,304,755 lbs-ft
Activating Moment: 6,218,965.7 lbs-ft
Resisting Force: 139,829.16 lbs
Activating Force: 70,774.633 lbs
F of S Rank: 1
Exit: (237.68906, 2.8444405) ft
Entry: (127.46013, 23) ft
Radius: 73.029548 ft
Center: (191, 59) ft

Slip Slices

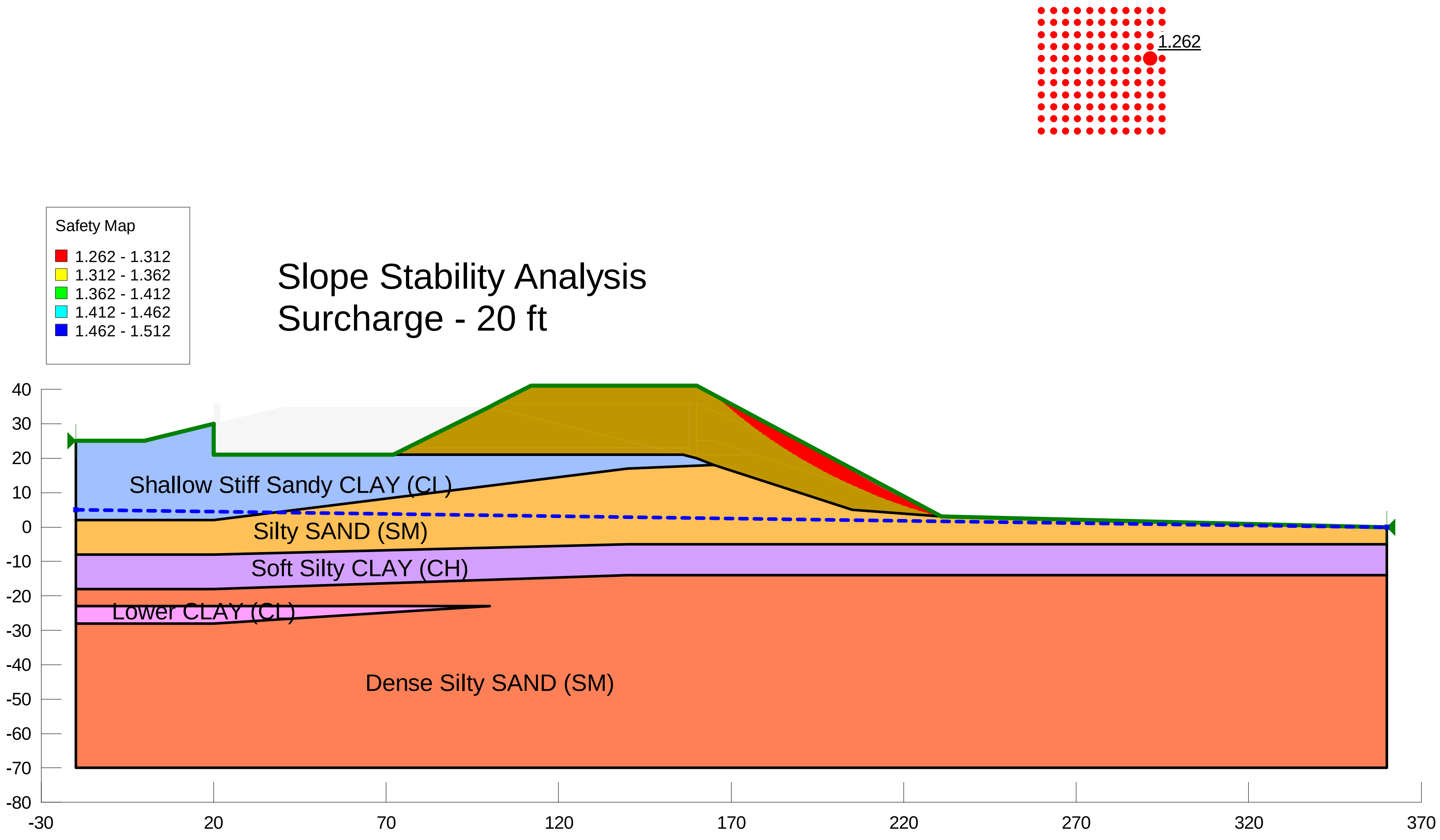
	X (ft)	Y (ft)	PWP (psf)	Base Normal Stress (psf)	Frictional Strength (psf)	Cohesive Strength (psf)
Slice 1	128.04788	22	-1,182.3551	842.2335	1,003.7348	0
Slice 2	130.30494	18.498391	-965.70788	291.46116	0	2,500
Slice 3	133.98069	13.483297	-655.784	1,856.6426	1,450.5682	0

Slice 4	137.99356	8.8491312	-369.90682	2,365.4681	1,848.1062	0
Slice 5	142.1594	4.7831804	-119.61186	2,848.6433	2,225.6041	0
Slice 6	146.1594	1.4049124	87.907854	3,302.0143	2,511.1352	0
Slice 7	149.95613	-1.3582611	257.21257	3,718.6454	2,704.3677	0
Slice 8	153.95613	-3.8922873	412.05159	4,127.2452	2,902.6273	0
Slice 9	157	-5.6222724	517.50348	4,811.798	0	1,000
Slice 10	159	-6.6359445	579.11452	5,316.6179	0	1,000
Slice 11	162.5	-8.1840026	672.83966	3,887.6209	0	1,000
Slice 12	167	-9.9407727	778.76737	4,075.6538	0	1,000
Slice 13	171	-11.206768	854.48128	4,123.1018	0	1,000
Slice 14	175	-12.225789	914.78399	4,148.8329	0	1,000
Slice 15	178.98713	-13.006578	960.23154	4,151.3867	0	1,000
Slice 16	182.96138	-13.558241	991.39228	4,128.7939	0	1,000
Slice 17	186.93564	-13.889196	1,008.7808	4,078.6104	0	1,000
Slice 18	191	-14	1,012.3579	3,995.5282	0	1,000
Slice 19	195.06436	-13.889196	1,002.1067	3,878.2473	0	1,000
Slice 20	199.03862	-13.558241	978.19203	3,726.7578	0	1,000
Slice 21	203.01287	-13.006578	940.50514	3,536.7486	0	1,000
Slice 22	206.76462	-12.284818	892.38698	3,325.6425	0	1,000

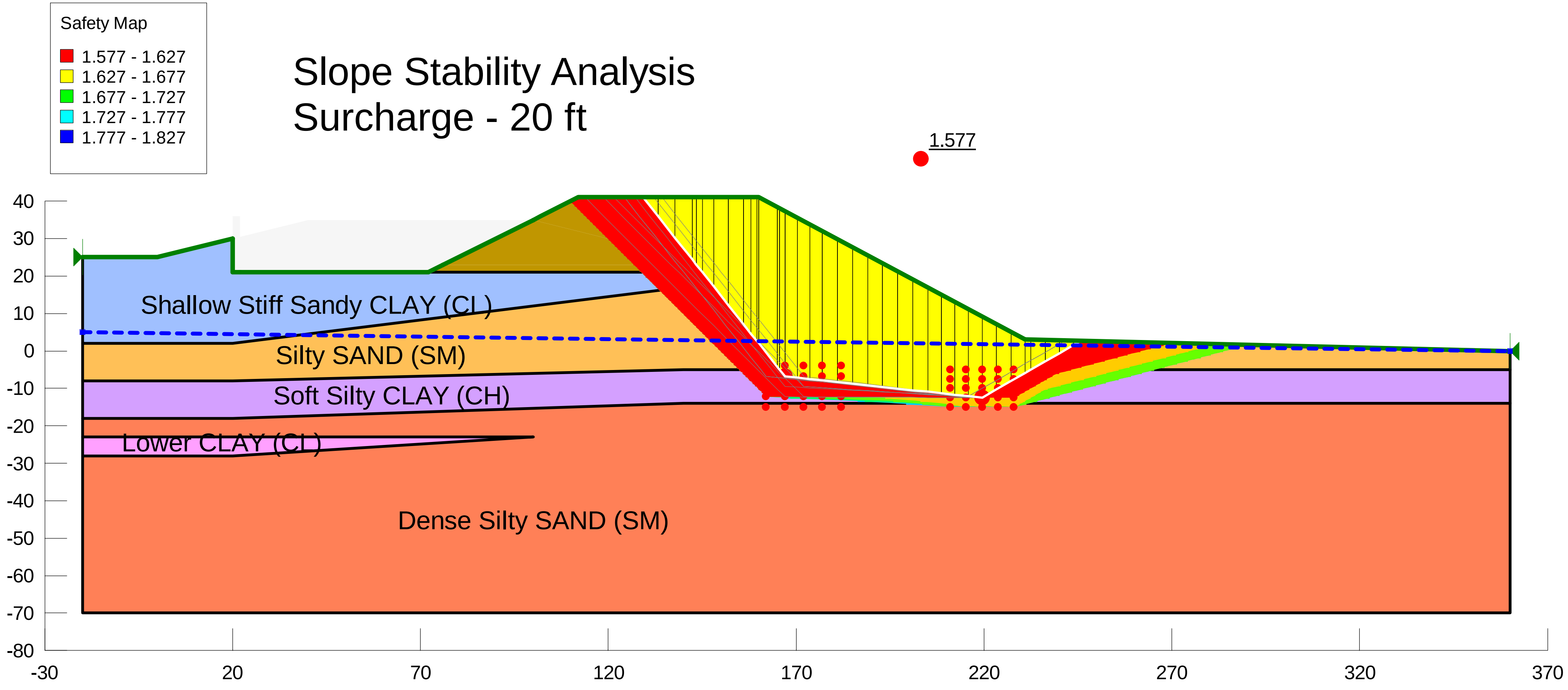
Slice 23	210.29387	-11.411034	834.96513	3,097.2913	0	1,000
Slice 24	213.82312	-10.34673	765.65485	2,835.831	0	1,000
Slice 25	217.35237	-9.0829436	683.8969	2,541.4543	0	1,000
Slice 26	220.88162	-7.608283	588.98038	2,214.9389	0	1,000
Slice 27	224.41086	-5.9083213	480.00507	1,857.6336	0	1,000
Slice 28	228.58774	-3.5504292	329.44316	1,285.7762	747.16929	0
Slice 29	233.59367	-0.23587231	118.50468	577.73834	358.79266	0
Slice 30	236.9382	2.2367772	-38.534684	125.4991	98.05064	0



Safety Map
1.905 - 1.955
1.955 - 2.005
2.005 - 2.055
2.055 - 2.105
2.105 - 2.155
Slope Stability Analysis
Loaded Tank Profile
1.905
Shallow Stiff Sandy CLAY (CL)
Silty SAND (SM)
Soft Silty CLAY (CH)
Lower CLAY (CL)
Dense Silty SAND (SM)
40
30
20
10
0
-10
-20
-30
-40
-50
-60
-70
-80
-30
20
70
120
170
220
270
320
370



1.262
Safety Map
1.262 - 1.312
1.312 - 1.362
1.362 - 1.412
1.412 - 1.462
1.462 - 1.512
Slope Stability Analysis
Surcharge - 20 ft
Shallow Stiff Sandy CLAY (CL)
Silty SAND (SM)
Soft Silty CLAY (CH)
Lower CLAY (CL)
Dense Silty SAND (SM)
40
30
20
10
0
-10
-20
-30
-40
-50
-60
-70
-80
-30
20
70
120
170
220
270
320
370


Safety Map
1.577 - 1.627
1.627 - 1.677
1.677 - 1.727
1.727 - 1.777
1.777 - 1.827
Slope Stability Analysis
Surcharge - 20 ft
1.577
Shallow Stiff Sandy CLAY (CL)
Silty SAND (SM)
Soft Silty CLAY (CH)
Lower CLAY (CL)
Dense Silty SAND (SM)
40
30
20
10
0
-10
-20
-30
-40
-50
-60
-70
-80
-30
20
70
120
170
220
270
320
370

APPENDIX IX

CPT/DMT TEST RESULTS

ConeTec Inc.

Geotechnical and Environmental Site Investigation Contractors

606-S Roxbury Industrial Center, Charles City, VA 23030 • Tel: (804) 966-5696 • Fax: (804) 966-5697
• E-mail: virginia@conetec.com • Website: www.conetec.com

September 11, 2013

Mr. Joe Robinson, P.E.
GET Solutions, Inc.
1592 E. Penniman Road
Williamsburg, VA 23185

Dear Mr. Robinson,

Re: CPTu and DMT Testing
Hopewell Regional Treatment Facility; Hopewell, VA

We are pleased to enclose our data submission for the CPTu and DMT testing that ConeTec performed for you at the above referenced site on September 5th and 6th, 2013.

Three cone penetration tests (CPTu) were completed to depths ranging from approximately 45 feet to 60 feet beneath the existing ground surface. Prior to advancing the cone penetrometer, the soundings were hand augered to a depth of approximately 5 feet. The zero depth refers to the top of the ground surface. A compression model electronic piezo cone penetrometer, with a 15 cm^2 tip and a 225 cm^2 friction sleeve, was used. The cone is designed with an equal end area friction sleeve and a tip end area ratio of 0.80. At the beginning of the sounding, the cone was outfitted with a vacuum-saturated, six millimeter-thick, porous plastic pore pressure element that is located immediately behind the tip (the U_2 location).

The cone was advanced using a 20-ton track-mounted cone penetration rig. As the cone was advanced into the ground, tip resistance (qc), sleeve friction (fs) and dynamic pore water pressure (U) were recorded every five centimeters (approximately every two inches) and are included in the attached file. A tabular output of this data and summary of engineering parameters, is included in the .xls files.

In addition to the CPT tests, three dilatometer tests (DMT) were completed to depths ranging from approximately 45 feet to 55 feet beneath the existing ground surface. Again, prior to advancing the DMT we hand augered to a depth of approximately 5 feet. The zero depth refers to the top of the ground surface. The dilatometers were advanced using a 20-ton, track-mounted rig. At approximately 1 foot intervals, the penetration was halted and the A and B pressure measurements were taken. At select depths, the C readings were measured. The A pressure measurement is defined as the amount of pressure applied to inflate the membrane to be flush with the plane of the dilatometer blade (A-position). The B pressure measurement is defined as the amount of pressure applied to inflate the membrane to extend 1.1 mm beyond the plane of the dilatometer blade. The C reading is defined as the pressure recorded when the membrane returns to the A-position. Upon completion of the DMT's we taped the holes in an effort to determine the ground water table. The soundings collapsed shallow and were dry. For the empirical correlation calculations, we assumed the water table based on the CPT soundings that were completed nearby the DMT soundings.

Enclosed are tabular and graphical output of the measurements taken in the field and estimated geotechnical parameters based on empirical correlations. The estimated geotechnical parameters are provided only as a first estimation. It is the project engineer's responsibility to use the measurements taken in the field to calculate the desired parameters based on correlations proven by his/her own experience. No warranty, expressed or implied, is made to the accuracy of these estimated geotechnical parameters. Refer to the Interpretation Methods section of the report for a more detailed explanation of empirical correlations.

Thank you very much for using ConeTec. It was a pleasure working with you and your staff and we look forward to working with you again in the future. If you have any questions or require additional information, please do not hesitate to contact us.

Best regards,

Alan Sweeney

Alan Sweeney
ConeTec, Inc.

Hopewell Regional Treatment Facility

September 5th and 6th, 2013
13-54061

Table 1: Sounding Information Table

Test Type	Sounding Number	Filename	Depth (ft)	Estimated GWT (ft)	Comments
CPTu	CPT-1	13-54061_CP01	60.0	31	
CPTu	CPT-2	13-54061_CP02	45.9	15	
CPTu	CPT-3	13-54061_CP03	55.8	28	
DMT	DMT-1	DMT-1	55.0	31*	
DMT	DMT-2	DMT-2	45.0	15*	
DMT	DMT-3	DMT-3	55.0	28*	

*Note: the estimated gwt for the DMT soundings is based on the adjacent CPT Soundings.

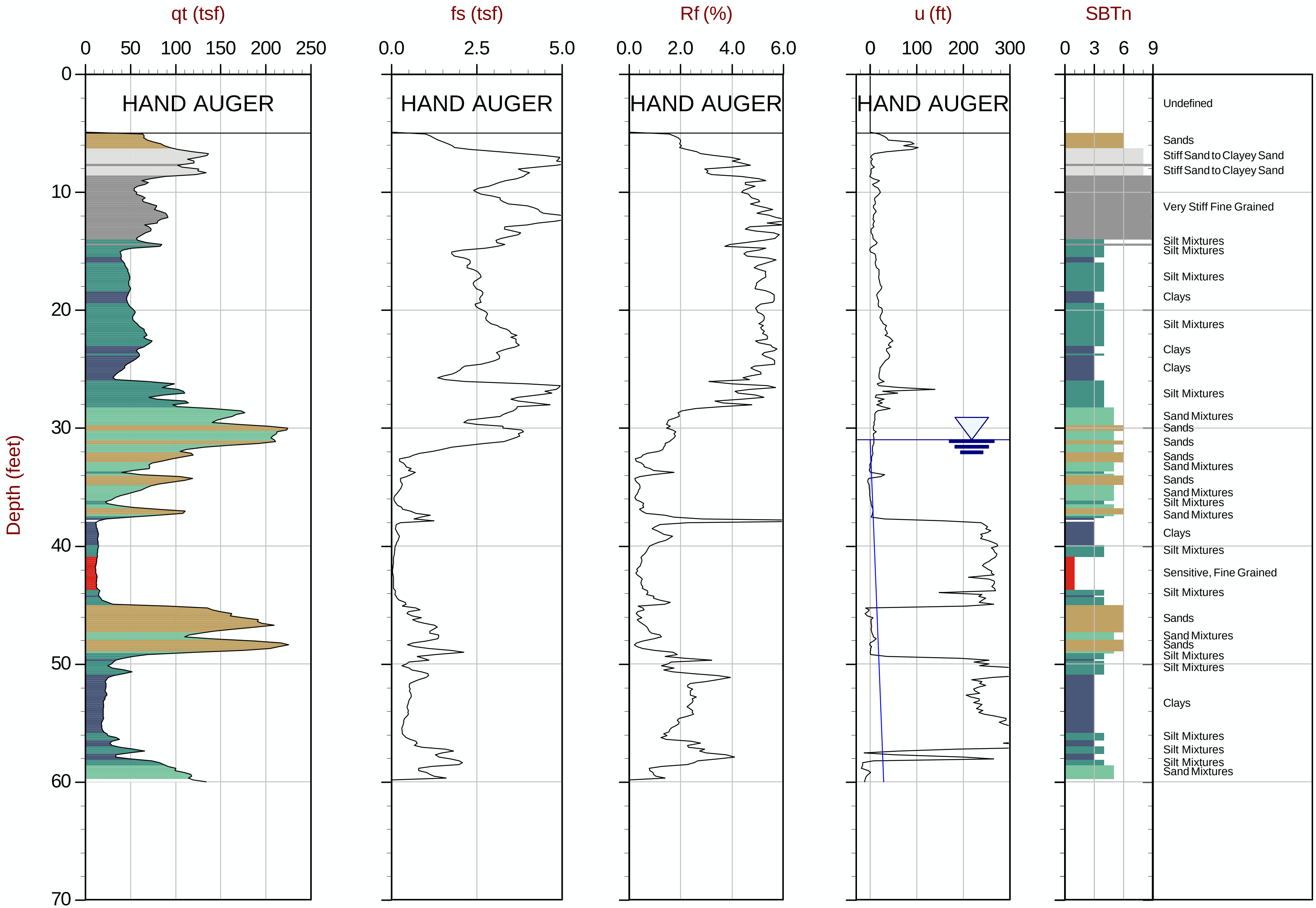
CPTu Plots

CONETEC

GET

Job No: 13-54061
Date: 09:05:13 09:46
Site: Hopewell Regional Treatment Facility

Sounding: CPT-1
Cone: 367:T1500F15U500



Max Depth: 18.300 m / 60.04 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: Every Point

File: 13-54061_CP01.COR
Unit Wt: SBT Chart Soil Zones

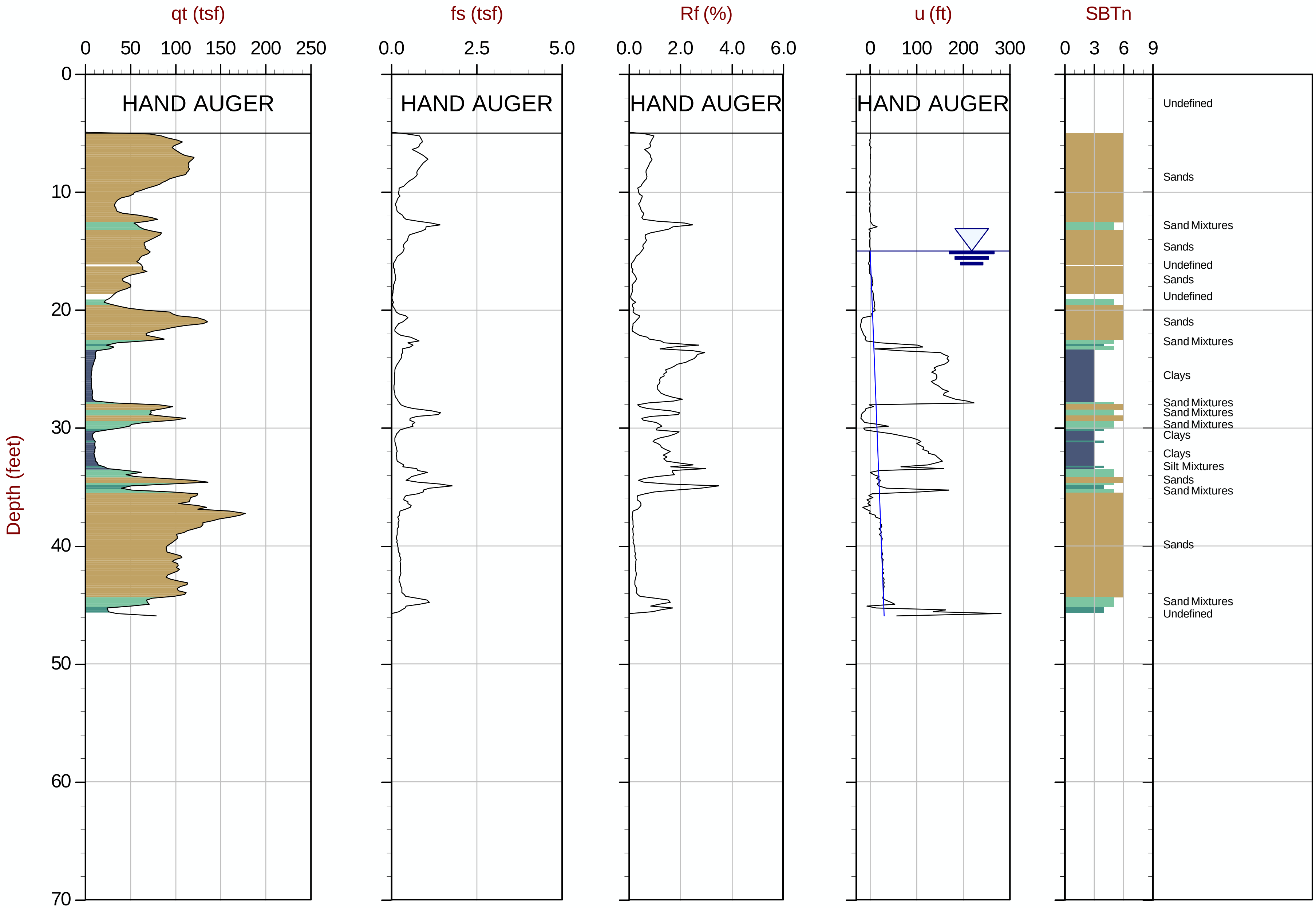
SBT: Lunne, Robertson and Powell, 1997
Coords: N: 37.29518 E: -77.25478
Page No: 1 of 1

ConeTec

GET

Job No: 13-54061
Date: 09:05:13 13:26
Site: Hopewell Regional Treatment Facility

Sounding: CPT-2
Cone: 367:T1500F15U500



Max Depth: 14.000 m / 45.93 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: Every Point

File: 13-54061_CP02.COR
Unit Wt: SBT Chart Soil Zones

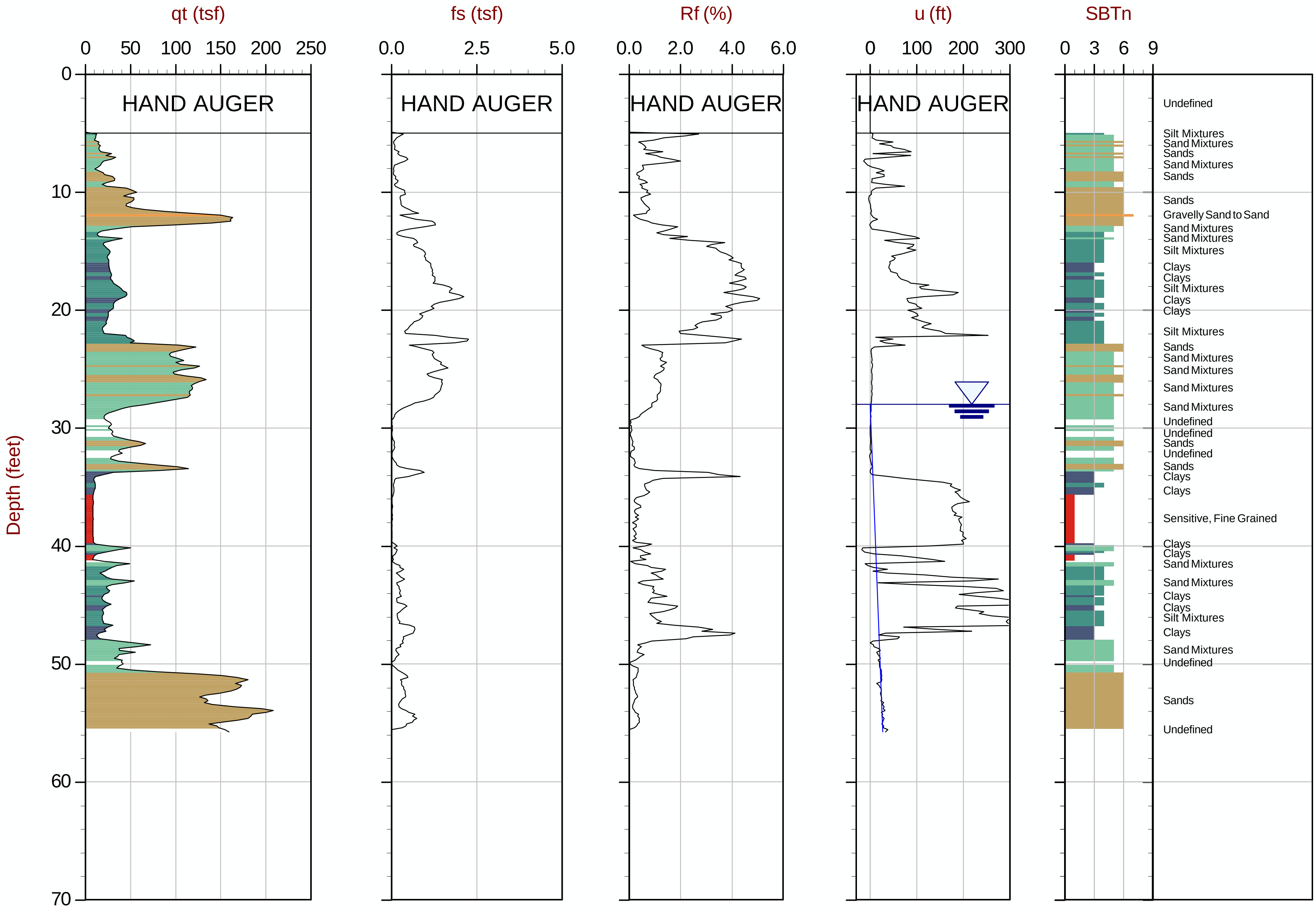
SBT: Lunne, Robertson and Powell, 1997
Coords: N: 37.29514 E: -77.25420
Page No: 1 of 1

CONETEC

GET

Job No: 13-54061
Date: 09:05:13 11:50
Site: Hopewell Regional Treatment Facility

Sounding: CPT-3
Cone: 367:T1500F15U500



Max Depth: 17.000 m / 55.77 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: Every Point

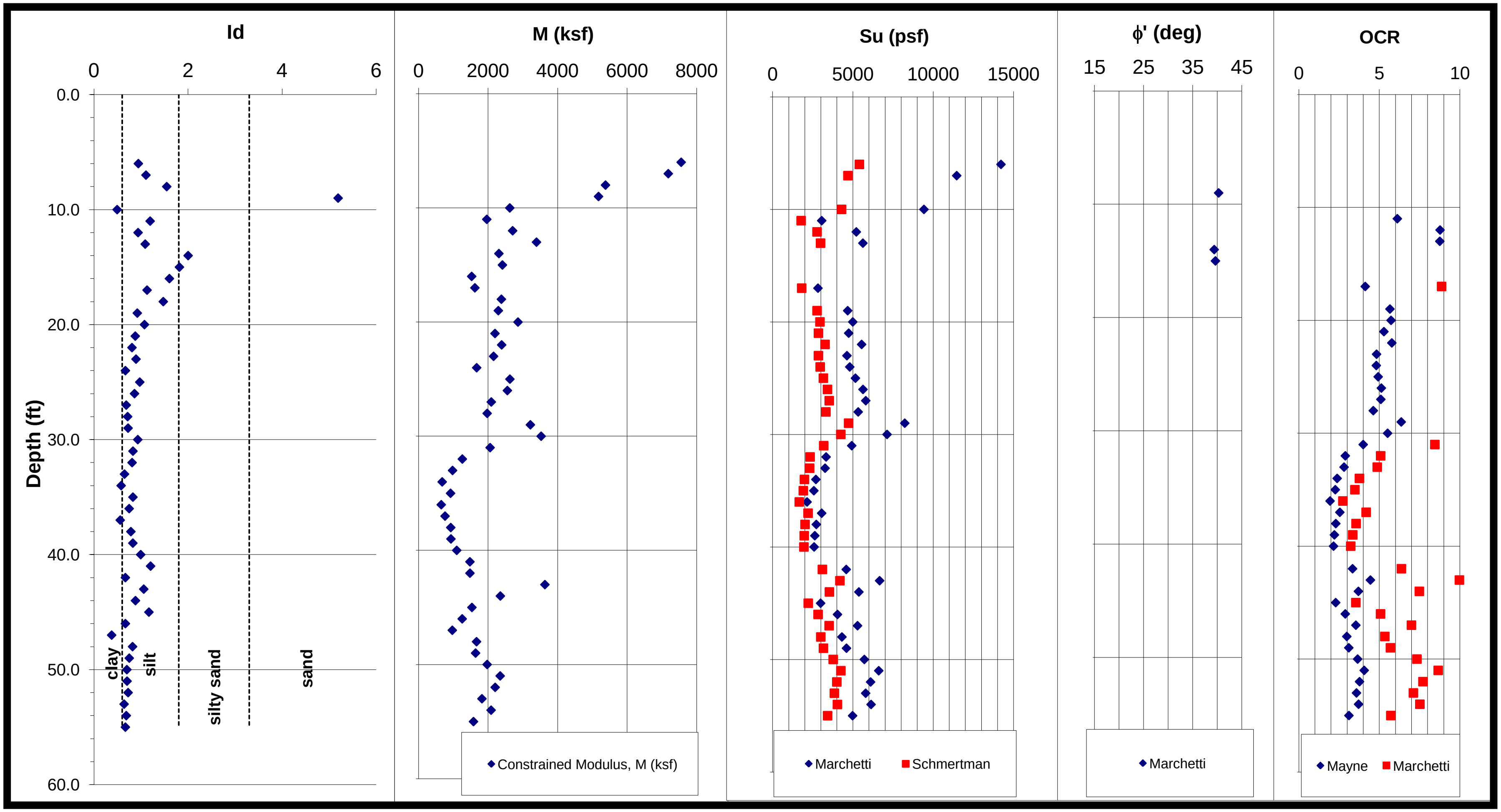
File: 13-54061_CP03.COR
Unit Wt: SBT Chart Soil Zones

SBT: Lunne, Robertson and Powell, 1997
Coords: N: 37.29578 E: -77.25437
Page No: 1 of 1

Dilatometer Test Data

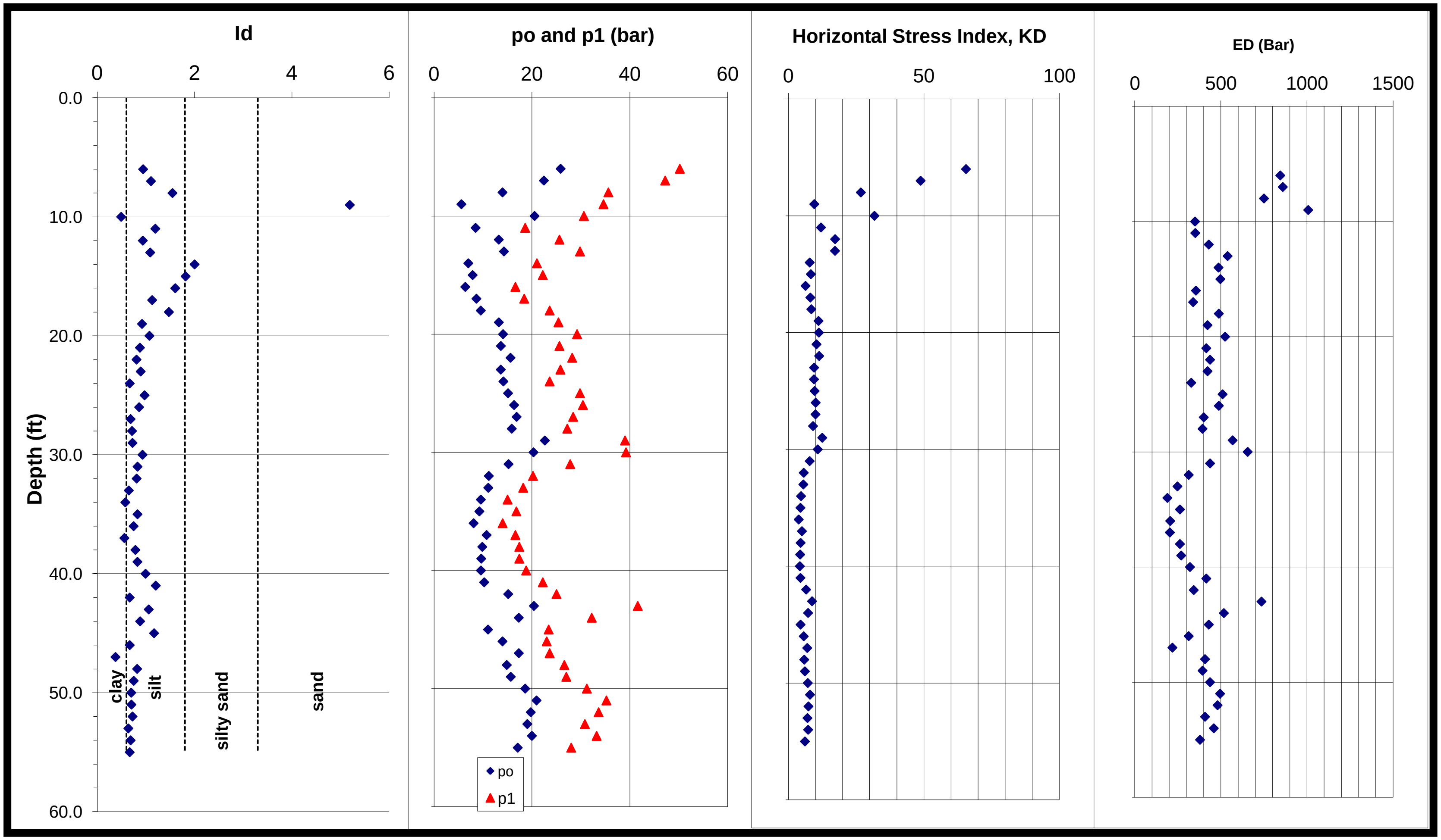
DILATOMETER TEST RESULTS

Test ID: DMT-1
Site: Hopewell Regional Treatment Facility
Location: Hopewell, Virginia
Project No.: 13-54061



DILATOMETER TEST RESULTS

Test ID: DMT-1
Site: Hopewell Regional Treatment Facility
Location: Hopewell, Virginia
Project No.: 13-54061



Job No: 13-54061
Job Name: Hopewell Regional Treatment Fac
Job Location: Hopewell, Virginia
Date: 9/6/13
Sounding No: DMT-1
Ground Water Depth (ft): 31

	Membrane 1	Membrane 2	Membrane 3
ΔA =	0.225	0	0
ΔB =	0.975	0	0
Zm=	0	bar	

Latitude: 37.29521
Longitude: -77.25463

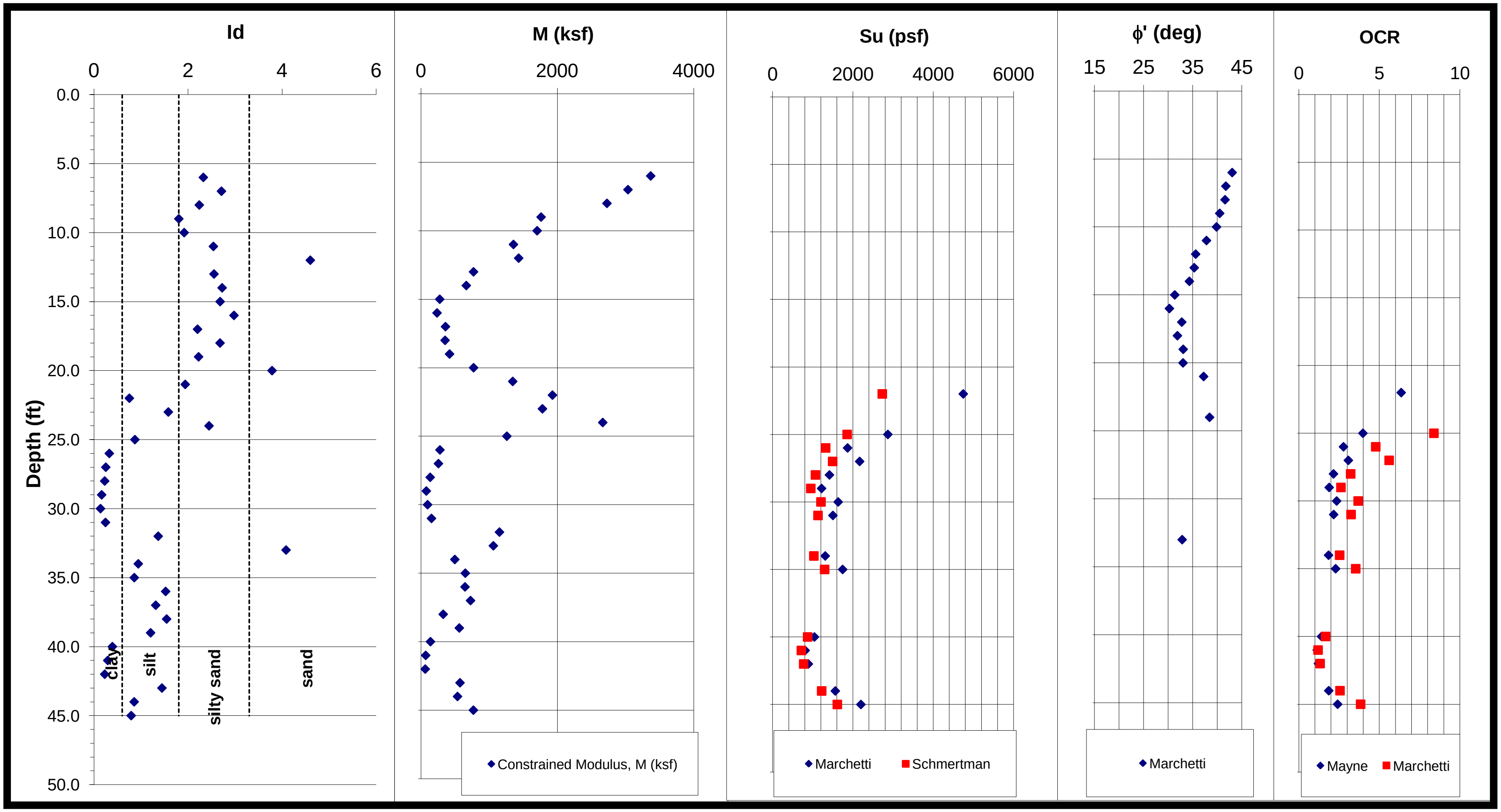
[1] Depth Below Existing Ground Surface
[2] Mayne, 1995
[3] Marchetti,1980
[4] Marchetti, 1997
[5] Campanella and Robertson, 1991
[6] Marchetti,1980
[7] Schmertman, 1981

CONETEC

Depth[1] (ft)	A (bar)	B (bar)	C (bar)	po (bar)	p1 (bar)	p2 (bar)	u_o (psf)	γ_T (pcf)	σ_{vo} (psf)	σ_{vo}' (psf)	Id	K_D	E_D (bar)	K_o	OCR[2]	OCR[3]	ϕ'[4] (deg)	R_M	E_D (ksf)	s_u[6] (psf)	s_u[7] (psf)	M (ksf)
6.0	26.8	51.2		25.9	50.225		0	137	824	824	0.94	65.5	845	5.3	33.4	231.3		4.28	1765	14218	5402	7556
7.0	23.4	48.2		22.4	47.225		0	137	961	961	1.10	48.8	860	4.5	24.8	146.0		4.00	1796	11460	4688	7185
8.0	14.8	36.6	0	14.0	35.625		0	132	1093	1093	1.55	26.7	751					3.43	1568			5379
9.0	6.75	35.6		5.6	34.625		0	128	1221	1221	5.19	9.6	1007				40.3	2.46	2104			5178
10.0	20.8	31.6		20.5	30.625		0	130	1351	1351	0.49	31.8	350	3.6	16.2	74.7		3.59	731	9422	4291	2625
11.0	8.75	19.6		8.5	18.625		0	124	1476	1476	1.19	12.0	352	2.1	6.1	16.4		2.67	734	3055	1774	1964
12.0	13.6	26.6	0	13.2	25.625		0	128	1604	1604	0.94	17.2	430	2.6	8.8	28.8		3.02	898	5209	2764	2707
13.0	14.8	30.8		14.3	29.825		0	130	1734	1734	1.09	17.2	539	2.5	8.8	28.7		3.01	1126	5620	2983	3394
14.0	7.45	22		7.0	21.025		0	125	1860	1860	2.00	7.9	486				39.4	2.28	1016			2313
15.0	8.35	23.2		7.9	22.225		0	126	1986	1986	1.82	8.3	497				39.6	2.32	1039			2414
16.0	6.65	17.6	0	6.4	16.625		0	123	2108	2108	1.60	6.3	355					2.06	742			1529
17.0	8.9	19.4		8.7	18.425		0	124	2233	2233	1.13	8.1	339	1.6	4.1	8.9		2.29	708	2823	1809	1622
18.0	10	24.6		9.6	23.625		0	127	2360	2360	1.47	8.5	488					2.34	1020			2384
19.0	13.6	26.4		13.2	25.425		0	128	2488	2488	0.92	11.1	423	2.0	5.7	14.5		2.60	883	4672	2766	2295
20.0	14.6	30.2	0	14.1	29.225		0	130	2618	2618	1.07	11.3	525	2.0	5.7	14.8		2.61	1096	4991	2946	2862
21.0	14	26.6		13.7	25.625		0	128	2747	2747	0.88	10.4	415	1.9	5.3	13.1		2.54	867	4735	2852	2200
22.0	16	29.2		15.6	28.225		0	130	2876	2876	0.81	11.3	437	2.0	5.8	15.0		2.62	913	5540	3263	2392
23.0	14	26.8		13.6	25.825		0	129	3005	3005	0.89	9.5	423	1.8	4.8	11.3		2.45	883	4626	2850	2160
24.0	14.4	24.6		14.2	23.625		0	127	3132	3132	0.67	9.5	328	1.8	4.8	11.3		2.44	685	4802	2961	1673
25.0	15.6	30.8	1.35	15.1	29.825	1.58	0	130	3262	3262	0.97	9.7	510	1.8	4.9	11.7		2.47	1065	5154	3159	2629
26.0	16.8	31.4		16.4	30.425		0	131	3393	3393	0.86	10.1	488	1.8	5.1	12.4		2.51	1020	5628	3416	2556
27.0	17.2	29.4		16.9	28.425		0	130	3522	3522	0.68	10.0	401	1.8	5.1	12.3		2.50	837	5798	3524	2093
28.0	16.2	28.2	5.75	15.9	27.225	5.98	0	129	3651	3651	0.71	9.1	393	1.7	4.6	10.6		2.40	822	5328	3318	1974
29.0	23.2	40		22.6	39.025		0	134	3785	3785	0.72	12.5	568	2.1	6.4	17.4		2.71	1187	8225	4729	3218
30.0	21	40.2		20.3	39.225		0	134	3919	3919	0.93	10.8	656	1.9	5.5	13.9		2.58	1370	7123	4245	3528
31.0	15.6	28.8		15.2	27.825		0	129	4048	4048	0.83	7.9	437	1.6	4.0	8.4		2.26	913	4924	3180	2060
32.0	11.4	21.2	0	11.2	20.225		62	125	4174	4111	0.81	5.7	313	1.3	2.9	5.1		1.93	654	3329	2332	1261
33.0	11.2	19.2		11.1	18.225		125	124	4298	4173	0.65	5.5	248	1.2	2.8	4.9		1.89	517	3265	2303	979
34.0	9.6	16		9.6	15.025		187	121	4419	4232	0.58	4.7	189	1.1	2.4	3.8		1.72	396	2692	1979	681
35.0	9.4	17.8		9.3	16.825		250	123	4542	4292	0.83	4.5	262	1.1	2.3	3.5		1.68	548	2566	1910	922
36.0	8.15	15	0	8.1	14.025		312	121	4662	4350	0.75	3.8	206	1.0	1.9	2.7		1.52	430	2144	1659	654
37.0	10.8	17.6		10.7	16.625		374	122	4785	4410	0.56	5.0	204	1.2	2.5	4.2		1.79	426	3053	2207	763
38.0	10	18.4		9.9	17.425		437	123	4908	4471	0.78	4.5	262	1.1	2.3	3.6		1.69	548	2718	2017	928
39.0	9.8	18.4		9.7	17.425		499	123	5031	4532	0.83	4.3	270	1.0	2.2	3.3		1.66	563	2625	1967	933
40.0	9.8	19.8	4.2	9.6	18.825	4.43	562	124	5156	4594	0.99	4.2	321	1.0	2.2	3.2		1.64	670	2582	1946	1099
41.0	10.6	23.2		10.3	22.225		624	126	5282	4658	1.20	4.5	415					1.70	867			1479
42.0	15.4	26		15.2	25.025		686	128	5410	4723	0.67	6.6	342	1.4	3.3	6.4		2.07	715	4583	3097	1480
43.0	21.2	42.6		20.4	41.625		749	135	5545	4796	1.06	8.7	736	1.7	4.4	10.0		2.37	1537	6661	4189	3636
44.0	17.8	33.2	0	17.3	32.225		811	131	5676	4865	0.88	7.3	517	1.5	3.7	7.5		2.18	1081	5369	3535	2354
45.0	11.4	24.4		11.0	23.425		874	127	5803	4929	1.17	4.5	430	1.1	2.3	3.5		1.71	898	2987	2217	1536
46.0	14.2	24		14.0	23.025		936	127	5929	4993	0.67	5.7	313	1.3	2.9	5.1		1.92	654	4038	2829	1257
47.0	17.4	24.6		17.3	23.625		998	126	6055	5057	0.37	7.0	219	1.5	3.5	7.0		2.13	457	5286	3519	972
48.0	15.2	27.6	1.25	14.9	26.625	1.48	1061	129	6184	5123	0.82	5.9	408	1.3	3.0	5.3		1.96	852	4314	2999	1669
49.0	16	28		15.7	27.025		1123	129	6312	5189	0.75	6.1	393	1.3	3.1	5.7		2.00	822	4598	3164	1642
50.0	19	32.2		18.6	31.225		1186	131	6443	5257	0.70	7.2	437	1.5	3.7	7.3		2.16	913	5709	3771	1974
51.0	21.4	36.2		20.9	35.225		1248	132	6575	5327	0.70	8.0	496	1.6	4.1	8.7		2.27	1035	6606	4250	2349
52.0	20.2	34.6	0.75	19.8	33.625	0.98	1310	132	6706	5396	0.72	7.4	481	1.5	3.8	7.7		2.19	1004	6099	3997	2205
53.0	19.4	31.8		19.1	30.825		1373	130	6837	5464	0.64	7.0	408	1.5	3.6	7.1		2.14	852	5792	3845	1824
54.0	20.4	34.2		20.0	33.225		1435	131	6968	5533	0.69	7.3	459	1.5	3.7	7.5		2.18	959	6129	4033	2088
55.0	17.4	29	0	17.1	28.025		1498	129	7097	5599	0.67	6.1	379	1.3	3.1	5.7		2.00	791	4978	3423	1581

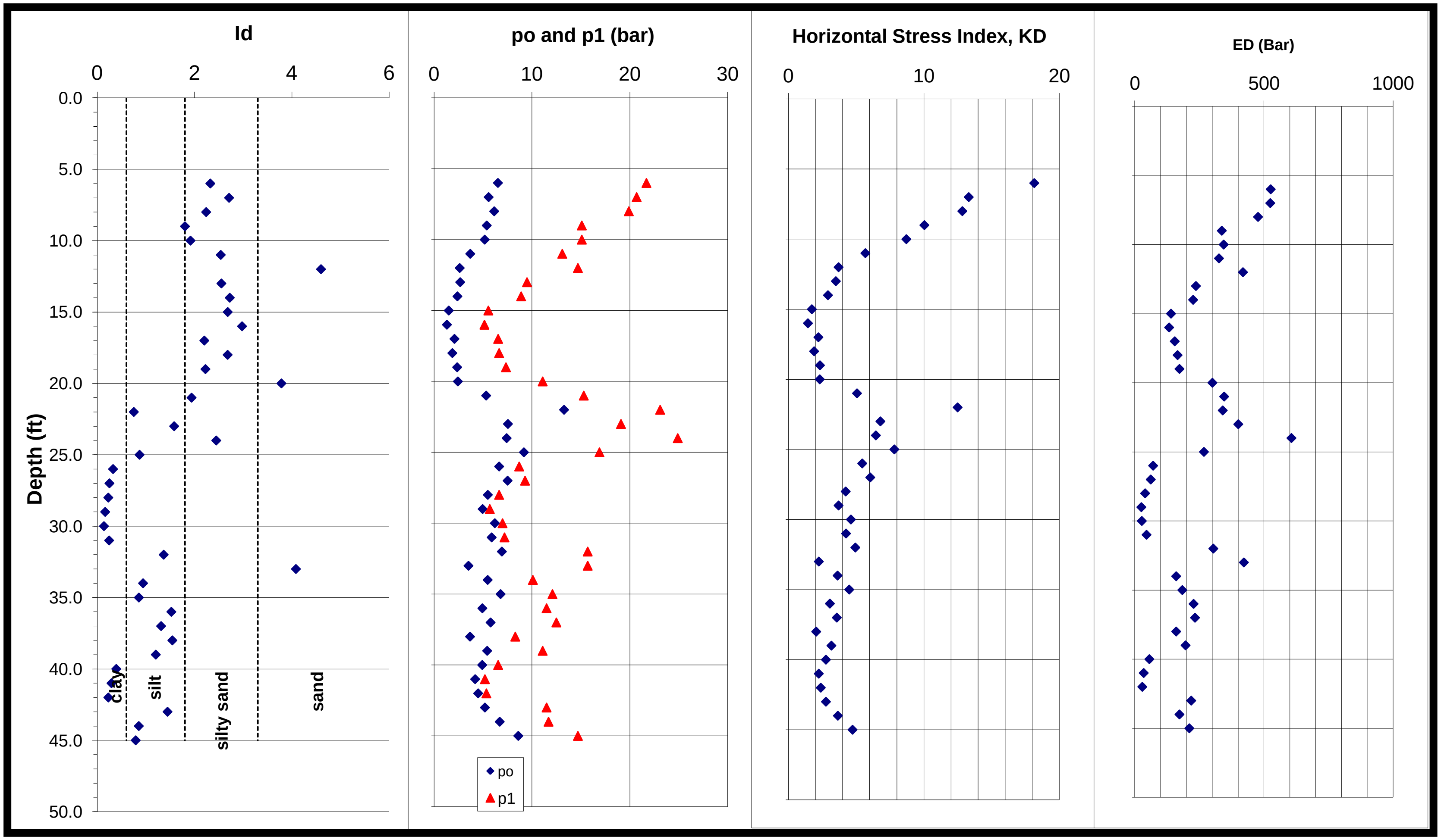
DILATOMETER TEST RESULTS

Test ID: DMT-2
Site: Hopewell Regional Treatment Facility
Location: Hopewell, Virginia
Project No.: 13-54061



DILATOMETER TEST RESULTS

Test ID: DMT-2
Site: Hopewell Regional Treatment Facility
Location: Hopewell, Virginia
Project No.: 13-54061



Job No: 13-54061
Job Name: Hopewell Regional Treatment Fac
Job Location: Hopewell, Virginia
Date: 9/6/13
Sounding No: DMT-2
Ground Water Depth (ft): 15

	Membrane 1	Membrane 2	Membrane 3
ΔA =	0.15	0	0
ΔB =	1.1	0	0
Zm=	0	bar	

Latitude: 37.29519
Longitude: -77.25380

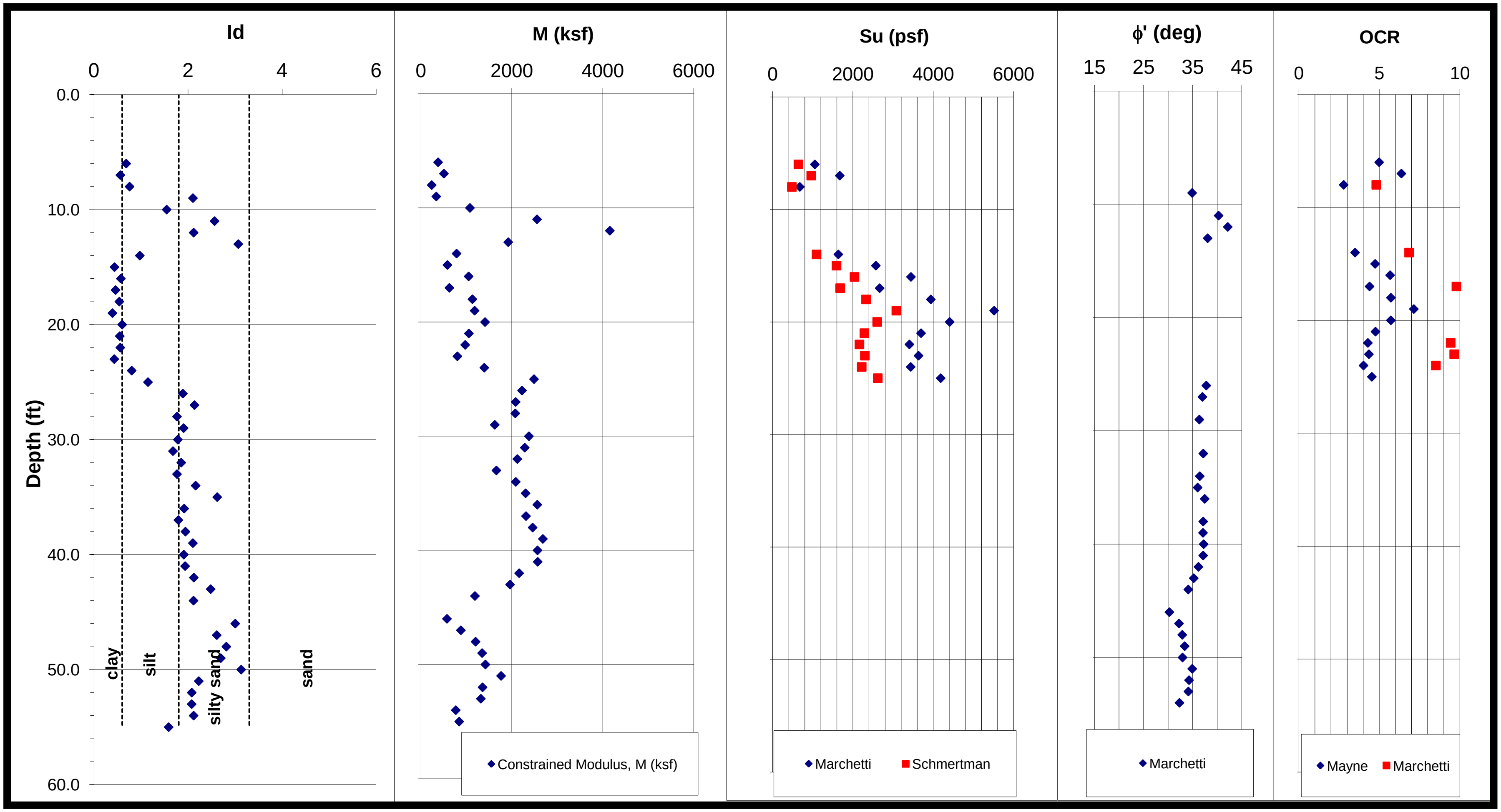
[1] Depth Below Existing Ground Surface
[2] Mayne, 1995
[3] Marchetti,1980
[4] Marchetti, 1997
[5] Campanella and Robertson, 1991
[6] Marchetti,1980
[7] Schmertman, 1981

CONETEC

Depth[1] (ft)	A (bar)	B (bar)	C (bar)	po (bar)	p1 (bar)	p2 (bar)	u_o (psf)	γ_T (pcf)	σ_{vo} (psf)	σ_{vo}' (psf)	Id	K_D	E_D (bar)	K_o	OCR[2]	OCR[3]	ϕ'[4] (deg)	R_M	E_D (ksf)	s_u[6] (psf)	s_u[7] (psf)	M (ksf)
6.0	7.1	22.8		6.5	21.7		0	125	751	751	2.32	18.1	526				43.1	3.06	1100			3369
7.0	6.15	21.8		5.6	20.7		0	124	876	876	2.71	13.3	525				41.8	2.77	1096			3036
8.0	6.65	21	0	6.1	19.9		0	124	1000	1000	2.24	12.8	477				41.6	2.74	997			2728
9.0	5.7	16.2		5.4	15.1		0	121	1121	1121	1.80	10.0	337				40.5	2.50	704			1762
10.0	5.5	16.2		5.2	15.1		0	121	1242	1242	1.92	8.7	344				39.9	2.37	719			1704
11.0	4	14.2		3.7	13.1		0	119	1361	1361	2.54	5.7	326				37.8	1.99	681			1357
12.0	3.05	15.8	0	2.6	14.7		0	118	1479	1479	4.60	3.7	419				35.6	1.64	875			1433
13.0	2.85	10.6		2.7	9.5		0	115	1594	1594	2.55	3.5	237				35.3	1.56	495			771
14.0	2.55	10		2.4	8.9		0	114	1709	1709	2.72	2.9	226				34.3	1.41	472			665
15.0	1.55	6.65		1.5	5.55		0	109	1817	1817	2.68	1.7	140				31.4	0.94	293			276
16.0	1.35	6.25	0	1.3	5.15		62	108	1925	1863	2.98	1.4	133				30.3	0.85	278			236
17.0	2.15	7.65		2.1	6.55		125	111	2036	1912	2.20	2.2	155				32.8	1.11	323			360
18.0	1.95	7.75		1.9	6.65		187	111	2147	1960	2.68	1.9	166				31.9	1.02	346			354
19.0	2.45	8.45		2.4	7.35		250	112	2259	2010	2.22	2.3	173				33.1	1.16	361			420
20.0	2.7	12.2	0	2.4	11.1		312	116	2375	2063	3.79	2.3	301				33.0	1.23	628			772
21.0	5.65	16.4		5.3	15.3		374	121	2496	2122	1.94	5.1	346				37.2	1.86	723			1346
22.0	13.6	24.2		13.3	23.1		437	127	2623	2186	0.75	12.5	341	2.1	6.4	17.4		2.71	711	4748	2730	1928
23.0	7.95	20.2		7.6	19.1		499	124	2747	2248	1.58	6.8	401					2.13	837			1781
24.0	8.1	26	0.25	7.4	24.9	0.40	562	127	2874	2312	2.45	6.5	607				38.4	2.10	1267			2666
25.0	9.4	18		9.2	16.9		624	123	2997	2373	0.87	7.8	268	1.6	4.0	8.4		2.25	559	2870	1855	1260
26.0	6.6	9.8		6.7	8.7		686	113	3110	2423	0.32	5.4	71	1.2	2.8	4.8		1.88	148	1867	1321	279
27.0	7.45	10.4		7.5	9.3		749	113	3223	2474	0.25	6.0	62	1.3	3.1	5.6		1.98	129	2167	1495	257
28.0	5.4	7.75	1.95	5.5	6.65	2.10	811	109	3332	2520	0.23	4.2	40	1.0	2.2	3.2		1.62	84	1415	1067	135
29.0	4.85	6.8		5.0	5.7		874	106	3437	2564	0.16	3.7	26	0.9	1.9	2.6		1.48	53	1219	950	79
30.0	6.1	8.1		6.2	7		936	107	3545	2609	0.14	4.6	27	1.1	2.3	3.7		1.71	57	1632	1204	97
31.0	5.8	8.3		5.9	7.2		998	110	3654	2656	0.24	4.3	46	1.0	2.2	3.2		1.62	95	1501	1130	154
32.0	7.2	16.8	0	6.9	15.7		1061	122	3776	2715	1.36	4.9	304					1.81	635			1152
33.0	3.95	16.8		3.5	15.7		1123	119	3895	2772	4.08	2.2	423				32.9	1.20	883			1062
34.0	5.55	11.2		5.5	10.1		1186	116	4012	2826	0.94	3.6	160	0.9	1.8	2.5		1.48	335	1310	1026	497
35.0	6.9	13.2		6.8	12.1		1248	118	4130	2882	0.86	4.5	184	1.1	2.3	3.5		1.69	384	1744	1295	651
36.0	5.1	12.6	0	4.9	11.5		1310	118	4248	2937	1.52	3.1	228					1.36	476			646
37.0	5.95	13.6		5.8	12.5		1373	119	4366	2994	1.31	3.6	233					1.49	487			727
38.0	3.75	9.4		3.7	8.3		1435	113	4480	3045	1.54	2.1	160					0.97	335			326
39.0	5.55	12.2		5.4	11.1		1498	117	4597	3100	1.20	3.2	197					1.37	411			563
40.0	4.85	7.65	2.15	4.9	6.55	2.30	1560	110	4707	3147	0.39	2.8	56	0.7	1.4	1.7		1.18	118	1041	872	140
41.0	4.1	6.3		4.2	5.2		1622	106	4813	3190	0.29	2.2	35	0.6	1.1	1.2		0.97	72	810	715	70
42.0	4.4	6.45		4.5	5.35		1685	105	4918	3233	0.23	2.4	29	0.6	1.2	1.3		1.03	61	890	773	63
43.0	5.35	12.6		5.2	11.5		1747	117	5035	3288	1.44	2.8	219					1.26	457			573
44.0	6.8	12.8		6.7	11.7		1810	118	5153	3344	0.85	3.7	173	0.9	1.9	2.6		1.48	361	1561	1221	536
45.0	8.75	15.8	1.9	8.6	14.7	2.05	1872	121	5274	3402	0.79	4.7	211	1.1	2.4	3.8		1.74	441	2198	1611	769

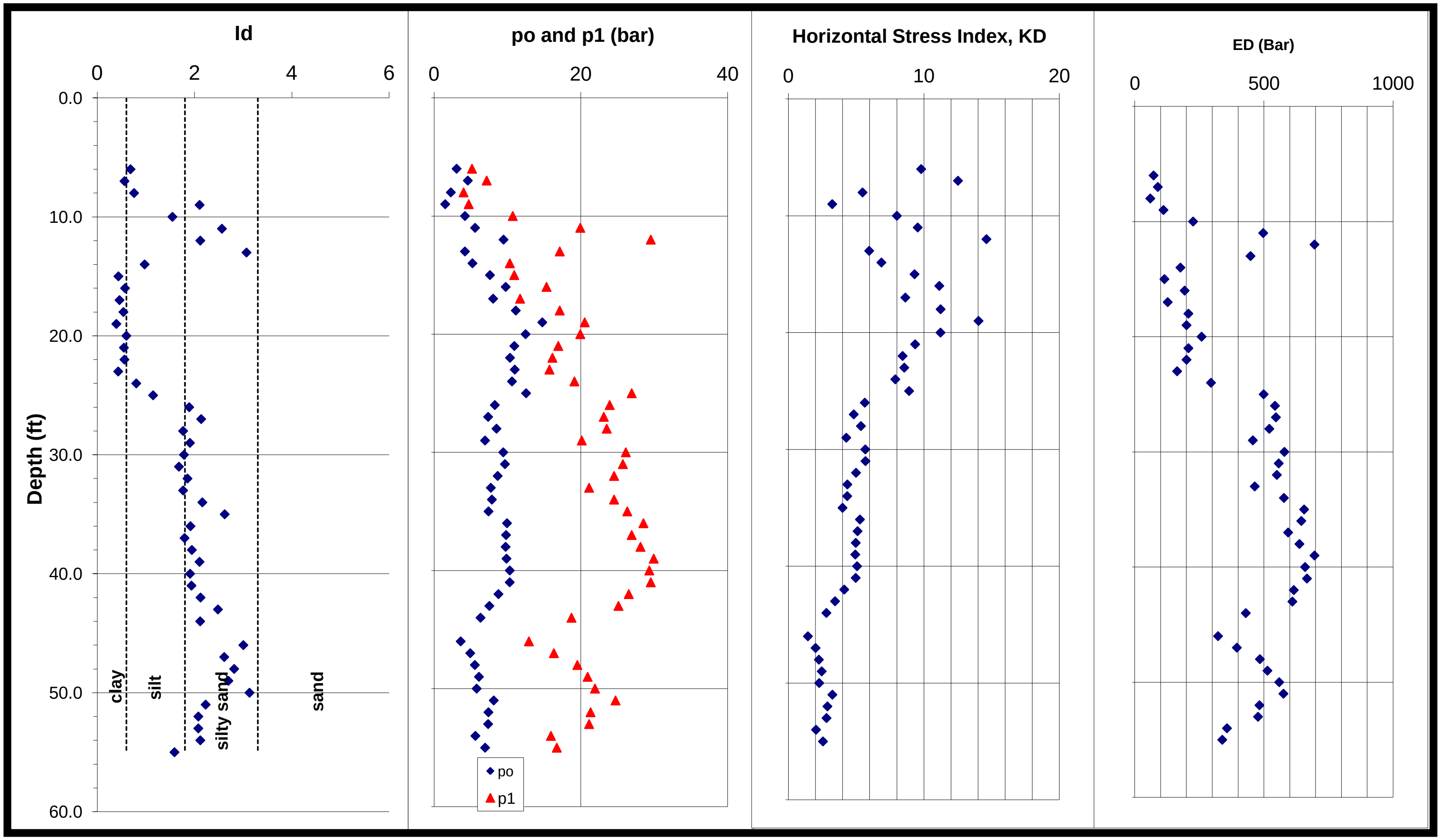
DILATOMETER TEST RESULTS

Test ID: DMT-3
Site: Hopewell Regional Treatment Facility
Location: Hopewell, Virginia
Project No.: 13-54061



DILATOMETER TEST RESULTS

Test ID: DMT-3
Site: Hopewell Regional Treatment Facility
Location: Hopewell, Virginia
Project No.: 13-54061



Job No: 13-54061
Job Name: Hopewell Regional Treatment Fac
Job Location: Hopewell, Virginia
Date: 9/6/13
Sounding No: DMT-3
Ground Water Depth (ft): 28

	Membrane 1	Membrane 2	Membrane 3
ΔA =	0.225	0	0
ΔB =	0.875	0	0
Zm=	0	bar	

Latitude: 37.29599
Longitude: -77.25439

[1] Depth Below Existing Ground Surface
[2] Mayne, 1995
[3] Marchetti,1980
[4] Marchetti, 1997
[5] Campanella and Robertson, 1991
[6] Marchetti,1980
[7] Schmertman, 1981

Depth[1] (ft)	A (bar)	B (bar)	C (bar)	po (bar)	p1 (bar)	p2 (bar)	u_o (psf)	γ_T (pcf)	σ_{vo} (psf)	σ_{vo}' (psf)	Id	K_D	E_D (bar)	K_o	OCR[2]	OCR[3]	ϕ'[4] (deg)	R_M	E_D (ksf)	s_u[6] (psf)	s_u[7] (psf)	M (ksf)
6.0	2.95	6.05		3.1	5.175		0	109	655	655	0.68	9.8	73	1.8	5.0	11.9		2.48	152	1051	642	377
7.0	4.5	8.05		4.6	7.175		0	113	768	768	0.56	12.5	89	2.1	6.4	17.5		2.71	186	1672	961	506
8.0	2.15	4.9		2.3	4.025		0	107	875	875	0.76	5.5	60	1.2	2.8	4.8		1.89	126	677	479	237
9.0	1.45	5.6		1.5	4.725		0	108	982	982	2.10	3.2	111				34.9	1.45	232			338
10.0	4.3	11.6	0	4.2	10.725		0	117	1100	1100	1.54	8.0	226					2.29	472			1078
11.0	6.05	20.8		5.6	19.925		0	124	1224	1224	2.56	9.5	497				40.3	2.46	1039			2553
12.0	10.2	30.4		9.5	29.525		0	129	1353	1353	2.12	14.6	696				42.2	2.86	1453			4156
13.0	4.6	18		4.2	17.125		0	122	1475	1475	3.07	6.0	448				38.1	2.05	936			1920
14.0	5.25	11.2		5.2	10.325		0	117	1592	1592	0.97	6.9	177	1.4	3.5	6.8		2.12	369	1636	1093	784
15.0	7.55	11.8	0.9	7.6	10.925	1.13	0	117	1709	1709	0.43	9.3	115	1.8	4.7	11.0		2.43	240	2571	1591	582
16.0	9.8	16.2		9.8	15.325		0	122	1830	1830	0.57	11.1	193	2.0	5.7	14.6		2.60	403	3444	2038	1049
17.0	8	12.6		8.1	11.725		0	118	1948	1948	0.46	8.6	128	1.7	4.4	9.8		2.35	266	2665	1681	626
18.0	11.2	18		11.1	17.125		0	123	2071	2071	0.54	11.2	208	2.0	5.7	14.8		2.61	434	3940	2327	1132
19.0	14.8	21.4		14.8	20.525		0	124	2195	2195	0.39	14.0	200	2.3	7.1	20.9		2.82	419	5515	3081	1181
20.0	12.6	20.8	1.95	12.5	19.925	2.18	0	125	2320	2320	0.60	11.2	259	2.0	5.7	14.7		2.61	540	4410	2604	1410
21.0	11	17.8		10.9	16.925		0	123	2443	2443	0.55	9.4	208	1.8	4.8	11.1		2.43	434	3696	2285	1055
22.0	10.4	17		10.4	16.125		0	122	2565	2565	0.56	8.4	200	1.7	4.3	9.4		2.32	419	3407	2162	973
23.0	11	16.6		11.0	15.725		0	121	2686	2686	0.43	8.6	164	1.7	4.4	9.6		2.34	342	3634	2297	801
24.0	10.8	20		10.6	19.125		0	125	2811	2811	0.80	7.9	295	1.6	4.0	8.5		2.26	616	3439	2218	1393
25.0	13	27.8	0	12.5	26.925		0	129	2940	2940	1.15	8.9	499	1.7	4.5	10.3		2.39	1043	4185	2619	2487
26.0	8.8	24.8		8.3	23.925		0	127	3067	3067	1.89	5.6	543				37.8	1.96	1134			2223
27.0	7.9	24		7.4	23.125		0	126	3193	3193	2.14	4.8	547				37.0	1.83	1141			2084
28.0	9	24.4		8.5	23.525		0	127	3320	3320	1.76	5.4	521					1.91	1088			2075
29.0	7.35	21		6.9	20.125		62	125	3444	3382	1.90	4.3	457				36.4	1.70	955			1624
30.0	10	27	0	9.4	26.125		125	128	3572	3448	1.78	5.7	579					1.96	1210			2375
31.0	10.2	26.6		9.7	25.725		187	128	3700	3513	1.68	5.7	557					1.96	1164			2284
32.0	9.2	25.4		8.7	24.525		250	127	3828	3578	1.85	5.0	550				37.2	1.84	1149			2119
33.0	8.15	22		7.7	21.125		312	125	3953	3641	1.76	4.4	465					1.71	970			1660
34.0	8.45	25.4		7.9	24.525		374	127	4080	3705	2.16	4.3	577				36.5	1.73	1206			2086
35.0	8.1	27.2	0	7.4	26.325		437	127	4207	3770	2.62	4.0	656				36.0	1.68	1370			2302
36.0	10.6	29.4		9.9	28.525		499	129	4336	3837	1.92	5.3	645				37.5	1.90	1347			2559
37.0	10.4	27.8		9.8	26.925		562	128	4465	3903	1.79	5.1	594					1.86	1240			2311
38.0	10.4	29		9.8	28.125		624	129	4593	3969	1.94	5.0	638				37.2	1.85	1332			2457
39.0	10.6	30.8		9.9	29.925		686	129	4723	4036	2.10	4.9	696				37.1	1.85	1453			2683
40.0	11	30.2		10.3	29.325		749	129	4852	4103	1.91	5.1	659				37.2	1.86	1377			2564
41.0	11	30.4	0	10.3	29.525		811	129	4981	4170	1.94	5.0	667				37.1	1.84	1393			2568
42.0	9.4	27.4		8.8	26.525		874	128	5109	4236	2.12	4.1	616				36.2	1.68	1286			2160
43.0	8.15	26		7.5	25.125		936	127	5236	4300	2.48	3.4	610				35.2	1.54	1275			1960
44.0	6.7	19.6		6.3	18.725		998	123	5359	4361	2.12	2.8	430				34.1	1.32	898			1188
45.0			**Invalid Test**				1061	130	5489	4428												
46.0	3.85	13.8	0	3.6	12.925		1123	118	5607	4484	3.00	1.4	322				30.3	0.85	673			572
47.0	5.25	17.2		4.9	16.325		1186	121	5728	4542	2.61	2.0	395				32.2	1.06	826			879
48.0	6	20.4		5.6	19.525		1248	123	5851	4603	2.81	2.3	485				32.9	1.19	1012			1201
49.0	6.6	21.8		6.1	20.925		1310	124	5975	4664	2.70	2.5	514				33.4	1.25	1073			1345
50.0	6.35	22.8	0	5.8	21.925		1373	124	6099	4726	3.13	2.3	559				32.9	1.21	1168			1418
51.0	8.7	25.6		8.1	24.725		1435	127	6226	4790	2.23	3.2	576				34.9	1.47	1202			1763
52.0	7.85	22.2		7.4	21.325		1498	125	6350	4853	2.08	2.9	483				34.3	1.34	1008			1356
53.0	7.8	22		7.4	21.125		1560	125	6475	4915	2.08	2.8	477				34.1	1.32	997			1318
54.0	5.9	16.8		5.6	15.925		1622	121	6596	4974	2.12	2.0	357				32.3	1.03	746			767
55.0	7.2	17.6	0	7.0	16.725		1685	122	6718	5033	1.59	2.6	339					1.19	708			841

Pore Pressure Dissipation Test Data

Hopewell Regional Treatment Facility

September 5th and 6th, 2013
13-54061

Table 2: Pore Pressure Dissipation Test Summary Table

Sounding	Depth (ft)	Duration (sec)	c_h (cm^2/min)**	Comments
CPT-1	15.1	1460	0.59	

*Using Ir = 100

$$c_h = \frac{T^* a^2 \sqrt{I_r}}{t^*}$$

where:
a= cone radius
Ir= rigidity index
T*= modified time factor for given consolidation
t*= time of given consolidation

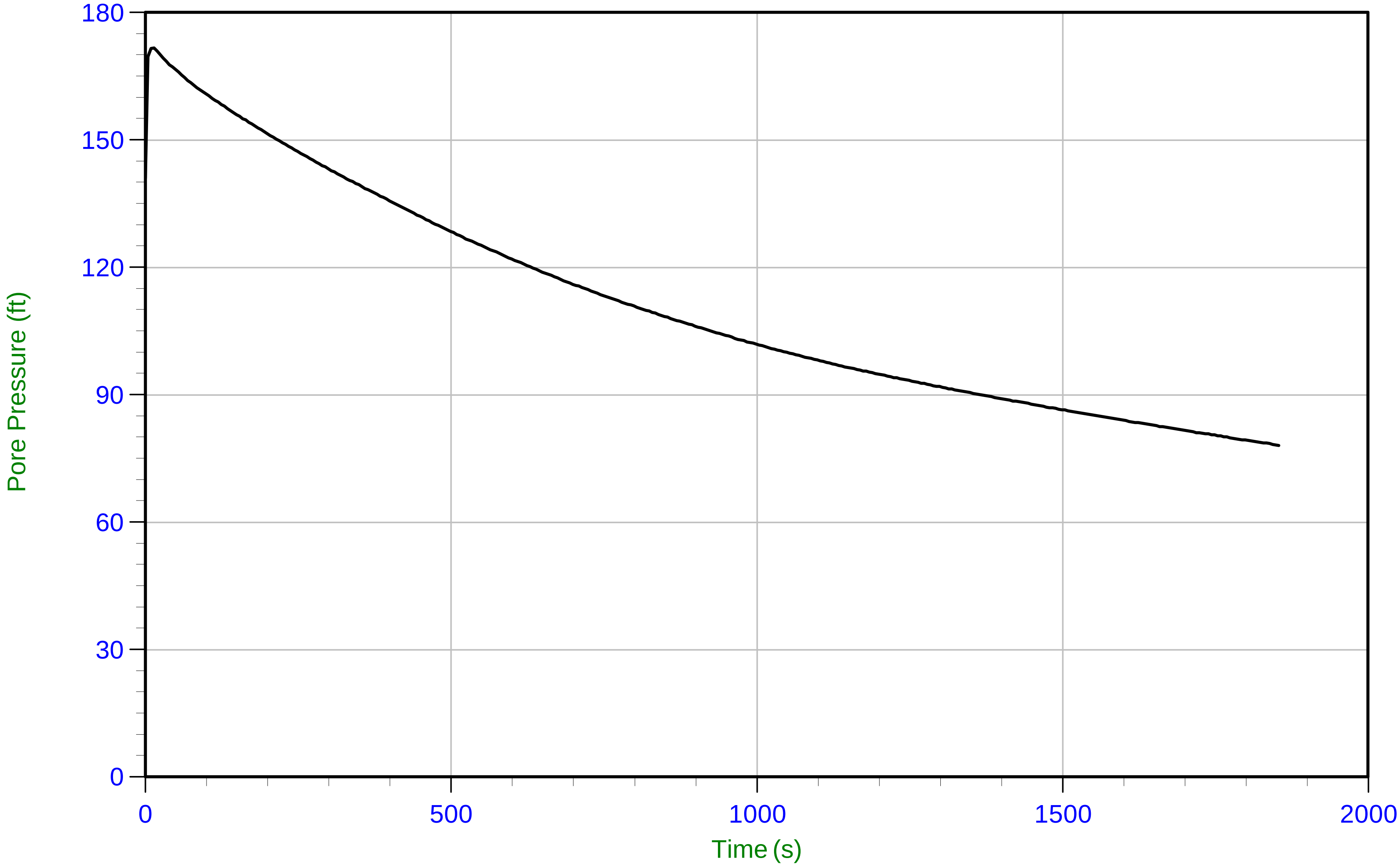
Degree of Consolidation	Modified Time Factors, T*
20%	0.038
30%	0.078
40%	0.142
50%	0.245
60%	0.439
70%	0.804
80%	1.60

CONETEC

GET

Job No: 13-54061
Date: 09/05/2013 13:26
Site: Hopewell Regional Treatment Facility

Sounding: CPT-2
Cone: 367:T1500F15U500
Cone Area: 15 sq cm



Trace Summary:

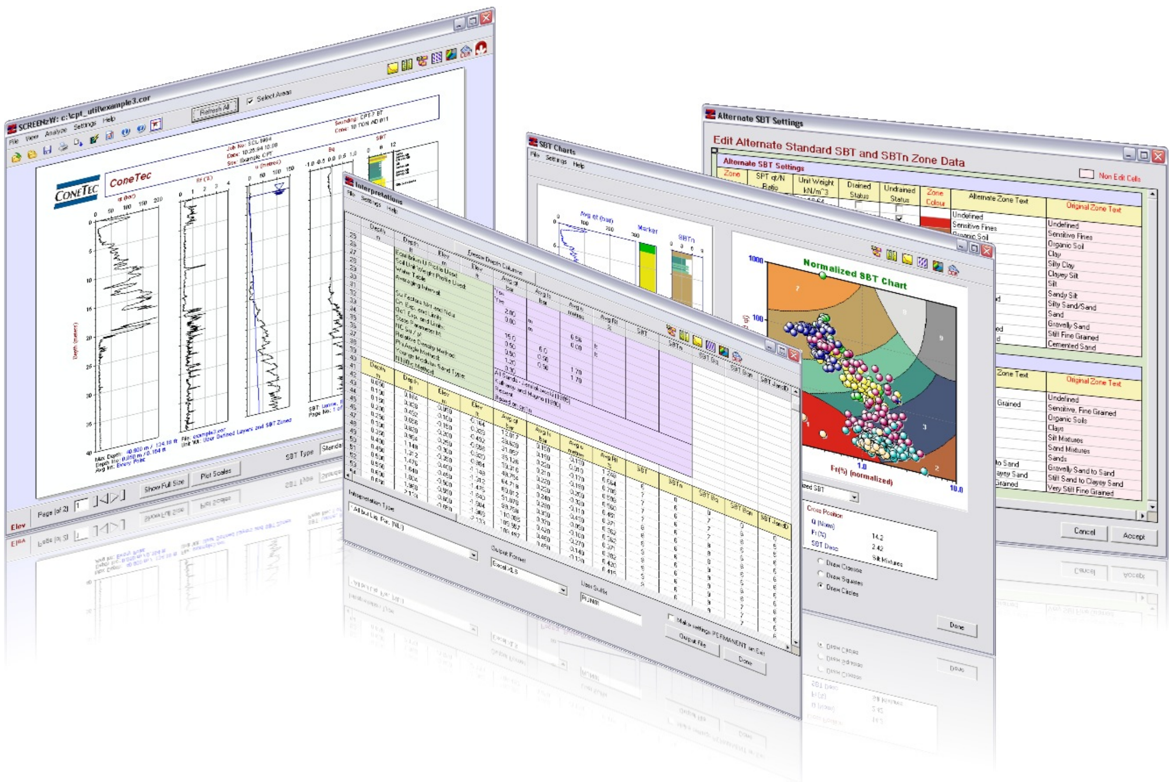
Filename: 13-54061_CP02.PPD
Depth: 10.100 m / 33.136 ft
Duration: 1855.0 s

U Min: 78.1 ft
U Max: 171.6 ft

Interpretation Methods

CONETEC INTERPRETATION METHODS

A Detailed Description of the Methods Used in ConeTec's CPT Interpretation and Plotting Software



Revision SZW-Rev 05B

Revised April 25, 2013

Prepared by Jim Greig

ConeTec Interpretations as of April 25, 2013

ConeTec's interpretation routine provides a tabular output of geotechnical parameters based on current published CPT correlations and is subject to change to reflect the current state of practice. The interpreted values are not considered valid for all soil types. The interpretations are presented only as a guide for geotechnical use and should be carefully scrutinized for consideration in any geotechnical design. Reference to current literature is strongly recommended. ConeTec does not warranty the correctness or the applicability of any of the geotechnical parameters interpreted by the program and does not assume liability for any use of the results in any design or review. Representative hand calculations should be made for any parameter that is critical for design purposes. The end user of the interpreted output should also be fully aware of the techniques and the limitations of any method used in this program. The purpose of this document is to inform the user as to which methods were used and what the appropriate papers and/or publications are for further reference.

The CPT interpretations are based on values of tip, sleeve friction and pore pressure averaged over a user specified interval (e.g. 0.20m). Note that q_t is the tip resistance corrected for pore pressure effects and q_c is the recorded tip resistance. Since all ConeTec cones have equal end area friction sleeves, pore pressure corrections to sleeve friction, f_S, are not required.

The tip correction is: $q_t = q_c + (1\text{-}a) \cdot u_2$

where: q_t is the corrected tip resistance
q_c is the recorded tip resistance
u_2 is the recorded dynamic pore pressure behind the tip (u_2 position)
a is the Net Area Ratio for the cone (typically 0.80 for ConeTec cones)

The total stress calculations are based on soil unit weights that have been assigned to the Soil Behavior Type zones, from a user defined unit weight profile or by using a single value throughout the profile.

Effective vertical overburden stresses are calculated based on a hydrostatic distribution of equilibrium pore pressures below the water table or from a user defined equilibrium pore pressure profile (this can be obtained from CPT dissipation tests). For over water projects the effects of the column of water have been taken into account as has the appropriate unit weight of water. How this is done depends on where the instruments were zeroed (i.e. on deck or at mud line).

Details regarding the interpretation methods for all of the interpreted parameters are provided in Table 1. The appropriate references cited in Table 1 are listed in Table 2. Where methods are based on charts or techniques that are too complex to describe in this summary the user should refer to the cited material.

The Soil Behavior Type classification charts (normalized and non-normalized) shown in Figures 1 and 2 are based on the charts developed by Dr. Robertson and Dr. Campanella at the University of British Columbia. These charts appear in many publications, most notably: Robertson, Campanella, Gillespie and Greig (1986); Robertson (1990) and Lunne, Robertson and Powell (1997). The Bq classification charts shown in Figures 3a and 3b are based on those described in Robertson (1990) and Lunne, Robertson and Powell (1997). The Jefferies and Davies SBT chart shown in Figure 3c is based on that discussed in Jefferies and Davies, 1993.

Where the results of a calculation/interpretation are declared *'invalid'* the value will be represented by the text strings "-9999" or "-9999.0". In some cases the value 0 will be used. Invalid results will occur because of (and not limited to) one or a combination of:

1. Invalid or undefined CPT data (e.g. drilled out section or data gap).

2. Where the interpretation method is inappropriate, for example, drained parameters in an undrained material (and vice versa).

3. Where interpretation input values are beyond the range of the referenced charts or specified limitations of the interpretation method.

4. Where pre-requisite or intermediate interpretation calculations are invalid.

The parameters selected for output from the program are often specific to a particular project. As such, not all of the interpreted parameters listed in Table 1 may be included in the output files delivered with this report.

The output files are provided in Microsoft Excel XLS format. The ConeTec software has several options for output depending on the number or types of interpreted parameters desired. Each output file will be named using the original COR file basename followed by a three or four letter indicator of the interpretation set selected (e.g. BSC, TBL, NLI or IFI) and possibly followed by an operator selected suffix identifying the characteristics of the particular interpretation run.

Table 1
CPT Interpretation Methods

Interpreted Parameter	Description	Equation	Ref
Depth	Mid Layer Depth *(where interpretations are done at each point then Mid Layer Depth = Recorded Depth)*	*Depth (Layer Top) + Depth (Layer Bottom) / 2.0*	
Elevation	Elevation of Mid Layer based on sounding collar elevation supplied by client	Elevation = Collar Elevation - Depth	
Avgqc	Averaged recorded tip value (q_c)	$Avgqc = \frac{1}{n}\sum_{i=1}^{n} q_c$ *n=1 when interpretations are done at each point*	
Avgqt	Averaged corrected tip (q_t) where: $q_t = q_c + (1-a) \bullet u$	$Avgqt = \frac{1}{n}\sum_{i=1}^{n} q_t$ *n=1 when interpretations are done at each point*	
Avgfs	Averaged sleeve friction (f_s)	$Avgfs = \frac{1}{n}\sum_{i=1}^{n} fs$ *n=1 when interpretations are done at each point*	
AvgRf	Averaged friction ratio (Rf) where friction ratio is defined as: $Rf = 100\% \bullet \frac{fs}{qt}$	$AvgRf = 100\% \cdot \frac{Avgfs}{Avgqt}$ *n=1 when interpretations are done at each point*	
Avgu	Averaged dynamic pore pressure *(u)*	$Avgu = \frac{1}{n}\sum_{i=1}^{n} u_i$ *n=1 when interpretations are done at each point*	
AvgRes	Averaged Resistivity (this data is not always available since it is a specialized test requiring an additional module)	$Avgu = \frac{1}{n}\sum_{i=1}^{n} RESISTIVITY_i$ *n=1 when interpretations are done at each point*	
AvgUVIF	Averaged UVIF ultra-violet induced fluorescence (this data is not always available since it is a specialized test requiring an additional module)	$Avgu = \frac{1}{n}\sum_{i=1}^{n} UVIF_i$ *n=1 when interpretations are done at each point*	
AvgTemp	Averaged Temperature (this data is not always available since it is a specialized test)	$Avgu = \frac{1}{n}\sum_{i=1}^{n} TEMPERATURE_i$ *n=1 when interpretations are done at each point*	

Interpreted Parameter	Description	Equation	Ref
AvgGamma	Averaged Gamma Counts (this data is not always available since it is a specialized test requiring an additional module)	$Avgu = \frac{1}{n}\sum_{i=1}^{n} GAMMA_i$ *n=1 when interpretations are done at each point*	
SBT	Soil Behavior Type as defined by Robertson and Campanella	See Figure 1	2, 5
U.Wt.	Unit Weight of soil determined from one of the following user selectable options: 1) uniform value 2) value assigned to each SBT zone 3) user supplied unit weight profile	See references	5
T. Stress σ_v	Total vertical overburden stress at Mid Layer Depth. *A layer is defined as the averaging interval specified by the user. For data interpreted at each point the Mid Layer Depth is the same as the recorded depth.*	$TStress = \sum_{i=1}^{n} \gamma_i h_i$ *where* γ_i is layer unit weight h_i is layer thickness	
E. Stress σ_v'	Effective vertical overburden stress at Mid Layer Depth	*Estress = Tstress -* u_{eq}	
Ueq	Equilibrium pore pressure determined from one of the following user selectable options: 1) hydrostatic from water table depth 2) user supplied profile	For hydrostatic option: $u_{eq} = \gamma_w \cdot (D - D_{wt})$ where u_{eq} is equilibrium pore pressure γ_w is unit weight of water D is the current depth D_{wt} is the depth to the water table	
Cn	SPT N_{60} overburden correction factor	$Cn=(\sigma_v')^{-0.5}$ where σ_v' *is in tsf* $0.5 < C_n < 2.0$	
N_{60}	SPT N value at 60% energy calculated from qt/N ratios assigned to each SBT zone. This method has abrupt N value changes at zone boundaries.	*See Figure 1*	4, 5
$(N_1)_{60}$	SPT N_{60} value corrected for overburden pressure	$(N_1)_{60} = Cn \cdot N_{60}$	4
$N_{60}I_c$	SPT N_{60} values based on the Ic parameter	$(qt/pa)/ N_{60} = 8.5 (1 – Ic/4.6)$	5
$(N_1)_{60}Ic$	SPT N_{60} value corrected for overburden pressure (using N_{60} $I_{c)}$. User has 2 options.	1) $(N_1)_{60}Ic = Cn \cdot (N_{60}\, Ic)$ 2) $q_{c1n}/ (N_1)_{60}Ic = 8.5 (1 – Ic/4.6)$	4 5
$(N_1)_{60cs}Ic$	Clean sand equivalent SPT $(N_1)_{60}Ic$. User has 3 options.	1) $(N_1)_{60cs}Ic = \alpha + \beta((N_1)_{60}Ic)$ 2) $(N_1)_{60cs}Ic = K_{SPT} * ((N_1)_{60}Ic)$ 3) $q_{c1ncs})/ (N_1)_{60cs}Ic = 8.5 (1 – Ic/4.6)$ FC ≤ 5%: α = 0, β=1.0 FC ≥ 35% α = 5.0, β=1.2 5% < FC < 35% $\alpha = \exp[1.76 – (190/FC^2)]$ $\beta = [0.99 + (FC^{1.5}/1000)]$	10 10 5
Su	Undrained shear strength based on q_t Su factor N_{kt} is user selectable	$Su = \frac{qt - \sigma_v}{N_{kt}}$	1, 5
Su	Undrained shear strength based on pore pressure Su factor $N_{\Delta u}$ is user selectable	$Su = \frac{u_2 - u_{eq}}{N_{\Delta u}}$	1, 5
k	Coefficient of permeability (assigned to each SBT zone)		5

Interpreted Parameter	Description	Equation	Ref
Bq	Pore pressure parameter	$Bq = \frac{\Delta u}{qt - \sigma_v}$ *where:* $\Delta u = u - u_{eq}$ *and* u = *dynamic pore pressure* u_{eq} = *equilibrium pore pressure*	1, 5
Q_t	Normalized q_t for Soil Behavior Type classification as defined by Robertson, 1990	$Qt = \frac{qt - \sigma_v}{\sigma'_v}$	2, 5
F_r	Normalized Friction Ratio for Soil Behavior Type classification as defined by Robertson, 1990	$Fr = 100\% \cdot \frac{fs}{qt - \sigma_v}$	2, 5
Net qt	Net tip resistance	$qt - \sigma_v$	
qe	Effective tip resistance	$qt - u_2$	
qeNorm	Normalized effective tip resistance	$\frac{qt - u_2}{\sigma'_v}$	
SBTn	Normalized Soil Behavior Type as defined by Robertson and Campanella	See Figure 2	2, 5
SBT-BQ	Non-normalized Soil Behavior type based on the Bq parameter	See Figure 3	2, 5
SBT-BQn	Normalized Soil Behavior based on the Bq parameter	See Figure 3	2, 5
SBT-JandD	Soil Behaviour Type as defined by Jeffries and Davies	See Figure 3	7
SBT-BQn	Normalized Soil Behavior base on the Bq parameter	See Figure 3	2, 5
I_c	Soil index for estimating grain characteristics	$Ic = [(3.47 - \log_{10}Q)^2 + (\log_{10} Fr + 1.22)^2]^{0.5}$ *Where:* $Q = \left(\frac{qt - \sigma_v}{P_{a2}}\right)\left(\frac{P_a}{\sigma'_v}\right)^n$ *And* *Fr is in percent* P_a = *atmospheric pressure* P_{a2} = *atmospheric pressure* *n varies from 0.5 to 1.0 and is selected in an iterative manner based on the resulting* I_c	3, 8
FC	Apparent fines content (%)	$FC=1.75(Ic^{3.25}) - 3.7$ $FC=100$ *for* $Ic > 3.5$ $FC=0$ *for* $Ic < 1.26$ $FC = 5\%$ *if* $1.64 < Ic < 2.6$ *AND* $F_r<0.5$	3
Ic Zone	This parameter is the Soil Behavior Type zone based on the Ic parameter (valid for zones 2 through 7 on SBTn chart)	Ic < 1.31 Zone = 7 1.31 < Ic < 2.05 Zone = 6 2.05 < Ic < 2.60 Zone = 5 2.60 < Ic < 2.95 Zone = 4 2.95 < Ic < 3.60 Zone = 3 Ic > 3.60 Zone = 2	3
PHI ϕ	Friction Angle determined from one of the following user selectable options: a) Campanella and Robertson b) Durgunoglu and Mitchel c) Janbu d) Kulhawy and Mayne	See reference	5 5 5 11

Interpreted Parameter	Description	Equation	Ref
Dr	Relative Density determined from one of the following user selectable options: a) Ticino Sand b) Hokksund Sand c) Schmertmann 1976 d) Jamiolkowski - All Sands	See reference	5
OCR	Over Consolidation Ratio	a) Based on Schmertmann's method involving a plot of S_u/σ_v' /($S_u/\sigma_v')_{NC}$ and OCR where the Su/p' ratio for NC clay is user selectable	9
State Parameter	The state parameter is used to describe whether a soil is contractive (SP is positive) or dilative (SP is negative) at large strains based on the work by Been and Jefferies	See reference	8, 6, 5
Es/qt	Intermediate parameter for calculating Young's Modulus, E, in sands. It is the Y axis of the reference chart.	Based on Figure 5.59 in the reference	5
Young's Modulus E	Young's Modulus based on the work done in Italy. There are three types of sands considered in this technique. The user selects the appropriate type for the site from: a) OC Sands b) Aged NC Sands c) Recent NC Sands Each sand type has a family of curves that depend on mean normal stress. The program calculates mean normal stress and linearly interpolates between the two extremes provided in the Es/qt chart.	Mean normal stress is evaluated from: $\sigma_m' = \frac{1}{3}\left(\sigma_v' + \sigma_h' + \sigma_h'\right)^3$ where σ_v'= vertical effective stress σ_h'= horizontal effective stress and $\sigma_h = K_o \cdot \sigma_v'$ with Ko assumed to be 0.5	5
q_{c1}	q_t normalized for overburden stress used for seismic analysis	$q_{c1} = q_t \cdot (Pa/\sigma_v')^{0.5}$ where: Pa = atm. Pressure q_t is in MPa	3
q_{c1n}	q_{c1} in dimensionless form used for seismic analysis	$q_{c1n} = (q_{c1} / Pa)(Pa/\sigma_v')^n$ where: Pa = atm. Pressure and n ranges from 0.5 to 1.0 based on Ic.	3
K_{SPT}	Equivalent clean sand factor for $(N_1)60$	$K_{SPT} = 1 + ((0.75/30) \cdot (FC - 5))$	10
K_{CPT}	Equivalent clean sand correction for q_{c1N}	$K_{cpt} = 1.0$ for $I_c \leq 1.64$ $K_{cpt} = f(I_c)$ for $I_c > 1.64$ (see reference)	10
q_{c1ncs}	Clean sand equivalent q_{c1n}	$q_{c1ncs} = q_{c1n} \cdot K_{cpt}$	3
CRR	Cyclic Resistance Ratio (for Magnitude 7.5)	$q_{c1ncs} < 50$: $CRR_{7.5} = 0.833\,[(q_{c1ncs}/1000] + 0.05$ $50 \leq q_{c1ncs} < 160$: $CRR_{7.5} = 93\,[(q_{c1ncs}/1000]^3 + 0.08$	10

Interpreted Parameter	Description	Equation	Ref
CSR	Cyclic Stress Ratio	$CSR = (\tau_{av}/\sigma_v') = 0.65\ (a_{max} / g)\ (\sigma_v / \sigma_v')\ r_d$ $r_d = 1.0 - 0.00765\ z \quad z \leq 9.15m$ $r_d = 1.174 - 0.0267\ z \quad 9.15 < z \leq 23m$ $r_d = 0.744 - 0.008\ z \quad 23 < z \leq 30m$ $r_d = 0.50 \quad z > 30m$	10
MSF	Magnitude Scaling Factor	See Reference	10
FofS	Factor of Safety against Liquefaction	$FS = (CRR_{7.5} / CSR)\ MSF$	10
Liquefaction Status	Statement indicating possible liquefaction	Takes into account FofS and limitations based on I_c and q_{c1ncs}.	10
Cont/Dilat Tip	Contractive / Dilative qc1 Boundary based on $(N_1)_{60}$	$(\sigma_v')_{boundary} = 9.58 \times 10^{-4}\ [(N_1)_{60}]^{4.79}$ qc1 is calculated from specified qt(MPa)/N ratio	13
Cq	Normalizing Factor	$Cq = 1.8 / (0.8 + ((\sigma_v'/Pa))$	12
qc1 (Cq)	Normalized tip resistance based on Cq	$q_{c1} = C_q * q_t$ (some papers use q_c)	12
Su(Liq)/s'v	Liquefied Shear Strength Ratio	$\frac{Su(Liq)}{\sigma_v'} = 0.03 + 0.0143(q_{c1})$	13

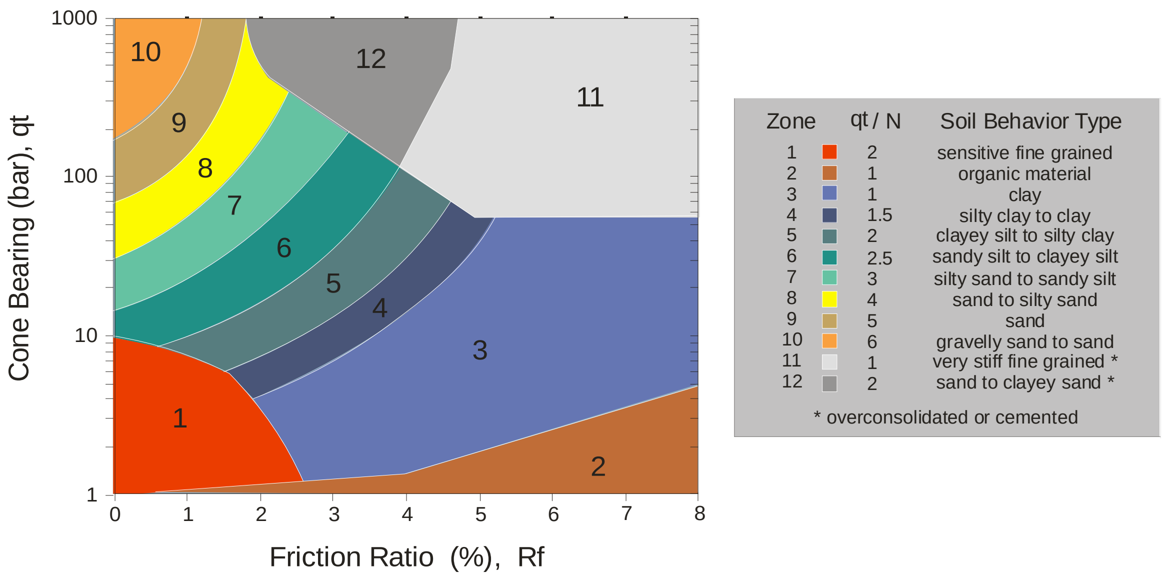


Figure 1 Non-Normalized Behavior Type Classification Chart

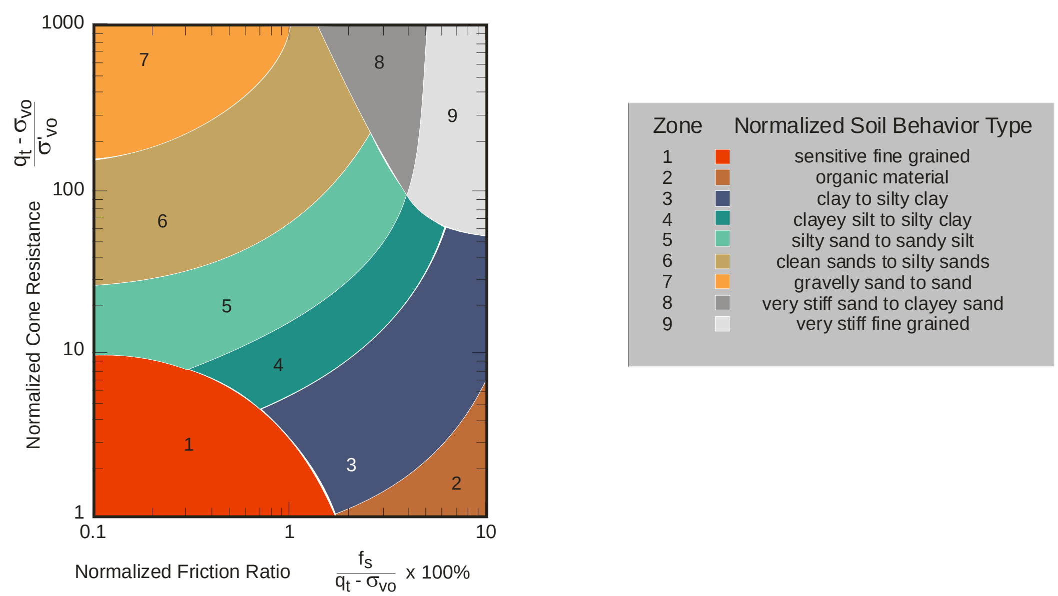


Figure 2 Normalized Behavior Type Classification Chart

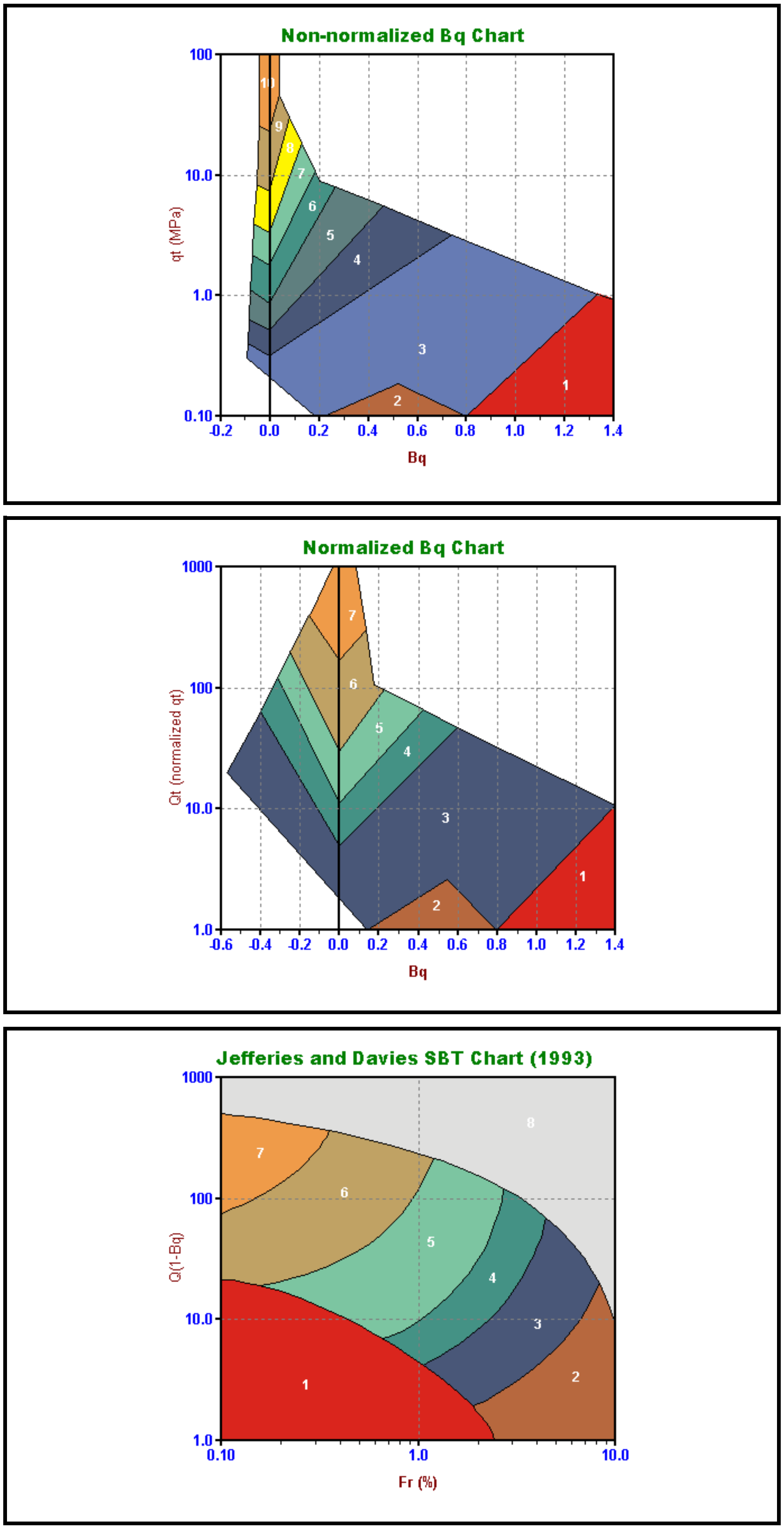


Figure 3 – Alternate Soil Behaviour Type Charts

Table 2 References

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Dilatometer (DMT) Data Reduction Correlations

Parameter	Correlation	Reference
Corrected First Reading	$p_0 = 1.05(A - Z_M + \Delta A) - 0.05(B - Z_M - \Delta B)$	
Corrected Second Reading	$p_1 = B - Z_M - \Delta B$	
Corrected Third Reading	$p_2 = C - Z_M + \Delta A$	
Material Index	$I_D = \dfrac{p_1 - p_o}{p_o - u_o}$	Marchetti, 1997
Horizontal Stress Index	$K_D = \dfrac{p_o - u_o}{\sigma'_{vo}}$	Marchetti, 1997
Dilatometer Modulus	$E_D = 34.7(p_1 - p_o)$	Marchetti, 1997
Coeff. Earth Pressure In Situ	$K_o = \left(\dfrac{K_D}{1.5}\right)^{0.47} - 0.6$	Marchetti, 1997
Overconsolidation Ratio	$OCR = (0.5K_D)^{1.56}$	Marchetti, 1997
Undrained Shear Strength	$c_u = 0.22\sigma'_{vo}(0.5K_D)^{1.25}$	Marchetti, 1997
Friction Angle	$\phi = 28 + 14.6\log K_D - 2.1\log^2 K_D$	Marchetti, 1997
Vertical Drained Constrained Modulus	$M_{DMT} = R_M E_D$ If $I_D \le 0.6$ $R_M = 0.14 + 2.36\log K_D$ If $I_D \ge 3$ $R_M = 0.5 + 2\log K_D$ If $0.6 < I_D < 3$ $R_M = R_{M,o} + (2.5 - R_{M,o})\log K_D$ where $R_{M,o} = 0.14 + 0.15(I_D - 0.6)$ If $K_D > 10$ $R_M = 0.32 + 2.18\log K_D$ If $R_M < 0.85$ set $R_M = 0.85$	Marchetti, 1997
Preconsol. Stress	$\sigma_p' = 0.509\,(p_o - u_o)$	Mayne, 1995
Total Unit Weight	$\gamma_T = 1.12\gamma_w \left(\dfrac{E_D}{p_a}\right)^{0.1} I_D^{-0.05}$	Mayne, et. al., 2002
Undrained Shear Strength	$s_u = \dfrac{p_o - u_o}{10}$	Schmertmann, 1991

Definitions

ΔA Reading: Quantification of resistance imparted by membrane to travel from the membrane's natural position to the A-position.

ΔB Reading: Quantification of resistance imparted by membrane to travel from the membrane's natural position to the B-position

A Position: Membrane just above feeler on sensing disk. Approximately flush with blade.

B Position: Membrane extended 1.1 mm into surrounding soil.

A Reading: Inflation pressure (reported in bar) required to expand membrane to A-position

B Reading: Inflation pressure (reported in bar) required to expand membrane to B-position.

C Reading: Deflation pressure (reported in bar) recorded when membrane is slowly deflated and returns to A- position.

Z_M: Zero gage reading. Reading of pressure gage when system is vented to atmosphere.

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