

# REPORT

## Geotechnical Exploration

### Proposed Aboveground Water Tank Atlanta VA Hospital Clairmont Road DeKalb County, Georgia

**Project Number  
2010.0241.02**

**November 23, 2010**



*We're here for you*

**UNITED CONSULTING**





November 23, 2010

Mr. Jeffery Bergmann, P.E.  
**AKEA, Inc.**  
25105 Newberry Road  
Newberry, Florida 32669

*Via Email: [jbergmann@akeainc.com](mailto:jbergmann@akeainc.com)*

PROJECT: Report of Geotechnical Exploration  
Proposed Aboveground Water Tank  
**Atlanta VA Hospital**  
Clairmont Road  
DeKalb County, Georgia  
Project Number 2010.0241.02

Dear Jeff:

United Consulting has completed a geotechnical exploration on the above referenced site. We appreciate the opportunity to assist you with this project. Please contact us if you have any questions regarding this report or if we can be of further assistance.

Sincerely,

**UNITED CONSULTING**



Aaron C. Epstein, P.E.  
Senior Geotechnical Engineer



Chris Roberds, P.G.  
Senior Executive Vice President

KK/ACE/CLR/tl

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## EXECUTIVE SUMMARY

United Consulting has completed a Geotechnical Exploration on the proposed Aboveground Tank at **Atlanta VA Hospital** located at 1670 Clairmont Road in Atlanta, DeKalb County, Georgia. The results from this investigation are briefly summarized below. The text of the report should be reviewed for a discussion of these items.

1. A complete geotechnical engineering service is performed through the Observational Method as an indivisible two-phase process. The first phase provides advice about project specific risks and represents our firm's opinion of subsurface conditions with recommendations. Field observation during construction comprises the second phase of our service and provides us the opportunity to assess the reliability of the subsurface data and the appropriateness of our recommendations. Actual conditions may differ from those encountered in the exploration phase.
2. PWR was encountered in borings C-1 and C-2 at a depth of 23 and 28 feet, respectively. Auger refusal due to rock occurred in boring C-1 at a depth about 40 feet. Based on the understanding that cuts and fills for grading the proposed tank area will range up to about 14 feet or so, difficult excavation conditions associated with PWR or rock excavation are not anticipated.
3. If the proposed aboveground tank is supported on conventional shallow foundation system, differential settlement on the order of 2 inches or so across the tank footprint should be anticipated. If this settlement is not acceptable, the proposed tank should be supported on a deep foundation system such as Auger cast piles or a shallow foundation underlain by a Geopiers/Vibropiers ground improvement system. The recommendations for various foundation options are included in the text.
4. No groundwater was encountered in the areas explored. We do not envision major problems related to shallow groundwater.
5. Soil parameters for concrete retaining walls or tank walls have been presented in the body of the report.

## SITE AND PROJECT DESCRIPTION

The project site is located within the existing Veterans Affairs (VA) Hospital on Clairmont Road, DeKalb County, Georgia. More specifically, the project site was located to the western of the intersection of Clairmont Road and Southern Lane in Atlanta, DeKalb County, Georgia. A Site Plan, provided by the client via e-mail dated October 21, 2010, was used as a guide to locate the boundaries of the project site. The general location of the project site is illustrated on Figure 1. At the time of our site visit, the project site was accessed via the entrance drive to the VA Hospital off of Clairmont Road. The new location (designated as Tank Location #1) generally was located on northwestern corner of VA Hospital and consisted of an approximately 30 feet high grass-covered slope.

Topographically, the area generally slopes down to the south from the high north of the site. Based on the provided site plan, the highest elevation is located on the north side of the site with an elevation of about 940 feet and the lowest elevation is located on the south of the site with an elevation of 908 feet. Total relief across the site is approximately 32 feet.

We understand that the proposed construction will consist of an approximately 60-foot diameter aboveground water storage tank. The tank will be a prestressed, reinforced concrete structure that will be set up to 35.5 feet above the existing grade. The proposed finished floor elevation (FFE) is expected to be approximately 920 feet or so. As such, cuts in the northern portion of the tank footprint and fills in the southern portion of the tank footprint ranging up to 14 feet or so are expected to mass grade the site.

If the site grading or structural information varies significantly from the above anticipated values, United Consulting must be contacted to determine if revisions to our recommendations should be re-evaluated and/or revised.

## PURPOSE

The purpose of this exploration was to determine the general type and condition of the subsurface materials at the project site, and to provide recommendations and general information regarding depth to groundwater and rock, excavation, and foundation design.

## SCOPE

The scope of our Geotechnical Exploration has included the following items:

1. A visual reconnaissance of the Site from a geotechnical standpoint;
2. Drilling two (2) Standard Penetration Test (SPT) borings, to determine the nature and condition of the subsurface soils;

3. Evaluation of soil samples obtained during our field exploration program for further identification and classification;
4. Analyzing subsurface conditions with respect to the proposed construction; and
5. Preparing this report to document the results of our field-testing program, engineering analysis, and to provide general recommendations and comments pertinent to the proposed development.

## **SUBSURFACE CONDITIONS**

Typical residual soils of the Piedmont Physiographic Province were encountered below the topsoil layer in the borings. The residual soils encountered generally consisted of loose to dense silty sand with trace amount of rock fragments, mica and clay. The N-values in the residual soils encountered ranged from 7 to 49 bpf.

Partially weathered rock (PWR) was encountered in the borings at depths ranging from 23 feet to 28 feet. PWR is a term for the residuum that can be penetrated by soil drilling techniques and has standard penetration resistance values (N-values) in excess of 100 bpf. Auger refusal due to rock occurred in boring C-1 at a depth about 40 feet.

Groundwater was not encountered at the time of drilling. However groundwater levels should be anticipated to fluctuate with the change of seasons, during periods of very low or high precipitation, or due to changes in the floodplain or watershed upstream from the area.

A more precise description of the conditions encountered in the borings is provided on the boring logs included in The Appendix.

## **DISCUSSIONS AND RECOMMENDATIONS**

The following recommendations are based on our understanding of the proposed construction, the data obtained in our soil test borings, a site reconnaissance, and our experience with subsurface conditions similar to those encountered at the project site.

We also recommend that United Consulting be consulted during construction to conduct Geotechnical Evaluations as described elsewhere in this report. The purpose is to verify the similarity of the actual subsurface conditions versus conditions anticipated by the designers.

### **Site Preparation**

Initial site preparation should include complete removal of topsoil, organics, vegetation, debris, or other deleterious materials that may exist in or within 5 feet beyond the limits of the proposed construction area. Following the initial site preparation operations and lowering the site grade

where needed, the exposed subgrade within the planned tank areas should be proofrolled. Proofrolling should be accomplished with a fully loaded, tandem-axle dump truck or its equivalent with two complete coverages in each of two perpendicular directions in the proposed construction area. The proofrolling should be performed under the observation of the Geotechnical Engineer. Areas that exhibit "pumping" during proofrolling should be treated by a method recommended by the Geotechnical Engineer. This method may consist of undercutting and backfilling with a suitable compacted fill material, moisture conditioning, stabilization, or some other method that is deemed suitable.

### **Excavation Conditions**

PWR was encountered in borings C-1 and C-2 at a depth about 23 and 28 feet. Based on the understanding that cuts and fills for grading the proposed tank area will range up to about 14 feet or so, difficult excavation conditions associated with PWR or rock excavation are not generally anticipated.

### **Caving Considerations**

Due to the presence of low-cohesive soils, caving of steep excavations should be expected. All excavations should be performed in accordance with OSHA standards.

### **Slopes**

In areas of the site where fill placement is required along an existing slope, we recommend benching of the slopes (see detail in Appendix) to prevent sliding of the fill mass along the existing surface. The maximum height of each bench cut should not exceed 3 feet. We recommend that permanent slopes be no steeper than 2(H):1(V). A minimum tank foundation setback of 10 feet from the top of all slopes is recommended. Slopes should be protected from erosion during construction and provided with appropriate permanent vegetative cover after construction. Slopes should be protected from concentrated runoff flow by means of berms and drainage ditches to direct runoff around slopes or through concrete channels. Appropriate vegetative cover should consist of fast growing grasses that will rapidly create a dense root mat over the entire slope. Landscaping consisting of shrubs and pine straw will not provide adequate slope protection.

### **Foundation Design Recommendations**

Below the anticipated tank FFE, the northern portion of the planned tank area is underlain by dense residual soils and underlying PWR whereas the southern portion of the site will be underlain by up to roughly 12 feet of fill and 10 feet or so of loose to firm residual soils. Because of the variability of the subsurface conditions across the tank footprint, the potential exists for rather abrupt differential tank settlement (on the order of 2 inches or so) to occur where the foundation bearing material transitions from dense residual soil/PWR to engineered fill. In order to reduce the potential for abrupt differential settlement across the tank area, we recommend that the tank be supported on a deep foundation system such as auger-cast pile or a shallow mat foundation underlain by a properly designed and installed ground improvement



system (compacted aggregate piers).

The following sections include our recommendations regarding auger-cast piles and Geopiers/Vibropiers.

### ***Option 1: Auger-Cast Piles***

Auger-cast piles can be considered for support of the proposed aboveground tank with total and differential settlements less than ½-inch or so. Because continuous PWR was encountered well above the rock elevation, auger cast for this project will derive a substantial amount of their capacity from side shear of the pile surface against the surrounding PWR or soil. However, we recommend that pile be installed to practical refusal in order to optimize the capacity of the piles. Practical auger refusal shall be defined as 12 inches or less penetration per minute for a minimum of one-minute period with a hollow stem auger installed using a drive box (excluding augers and hoses) having a minimum dead weight of at least 5 kips and a torque of at least 25,000 foot-pounds.

Boring C-1 encountered auger refusal at a depth of 40 feet (elevation 870) and Boring C-2 was terminated in PWR at a depth of 50 feet (elevation 884). Based on the boring data and our experience with similar projects, we envision piles should encounter refusal at depths ranging from approximately 40 to 55 feet below the tank bottom (elevations ranging 865 to 880). We note that subsurface conditions and installation equipment may vary considerably and the actual refusal depths may differ from the provided estimates.

The tables below estimate the allowable pile design capacities based on the pile diameters and safety factors.

**TABLE 1 – AUGER CAST PILE LOAD CAPACITY**

<b>Pile Diameter (inches)</b>	<b>Allowable Pile Capacity F.S.=2.5 (tons)</b>	<b>Allowable Pile Capacity F.S=3.0* (tons)</b>
14	80	65
16	110	90
*A load test will not be required if the piles are installed to practical refusal and the design pile capacity includes a safety factor of 3.0 or higher.		

For uplift calculations, the contribution of the side shear in the upper 5 feet of subsoils below the pile cap or in existing or newly placed fill should be ignored. Average allowable side shear values of 600 psf and 2,000 psf will be available for uplift resistance in portions of the auger cast piles extending through the residual soils and PWR encountered, respectively.



## **Auger-Cast Pile Construction and Evaluation**

The auger cast piles in this application should have minimum diameters of 14 and 16 -inch depending on the capacity selected and should be constructed of minimum of 4,000 pounds per square inch (psi) grout to effectively resist the compressive stresses induced on the piles. The structural engineer should specify actual grout strength required to effectively resist the stresses induced on the piles by compressive and/or uplift loads. This compressive strength should be verified by grout testing program during construction. If the load test allows the piles to be loaded to higher capacity, this strength may require to be increased.

Auger cast piles can be monitored only by indirect means. Therefore, we recommend a very "tight" set of specifications be made in order to achieve the properly formed piles for the recommended design capacities. Pile spacing should not be less than 3.0 times the diameter of the pile shaft center-to-center to prevent a reduction of the individual pile capacity through group effects. We recommend that the contractor use fixed leads and install piles using a drive box (excluding augers and hoses) having a minimum dead weight of at least 5 kips and a torque of at least 25,000 foot-pounds.

We recommend that we verify the contractor's proposed equipment for installation of the auger cast piles. The auger teeth should be examined prior to installing each pile, and excessively worn auger teeth should be replaced.

## **Pile Load Tests**

A pile load test is recommended if the selected design pile capacity has a safety factor less than 3.0. Because of the approximate nature of pile capacity estimates, the variability of the subsurface conditions, variables being used by the contractor, we recommend that a minimum of one pile load test be conducted. The actual locations for the load test shall be based on probe piles to be drilled prior to production at locations selected by the geotechnical engineer. We recommend a minimum of four (3) probe piles be drilled across the footprint of proposed aboveground tank to determine where the load tests should be conducted. A Geotechnical Engineer must be present during the probe pile installation and load test program to monitor installation and testing procedures. Additional load test may be necessary depending on the results of the load test as well as if any special requirement such as lateral load, tension piles, etc., is incorporated in the design.

The pile load test should be performed in accordance with ASTM Method D-1143, "Standard Method of Testing Piles Under Compressive Load" and should be monitored by the Geotechnical Engineer. Where piles are installed to practical refusal, as is the case for this project, the "Quick Load Test" method may be used. The contractor should provide the load cell capable of handling at least three times the design capacity. This would allow for maximum utilization of the pile's capacity and could result in substantial foundation cost savings. We also recommend installation of a telltale on all test piles to measure movement of the pile tip during the load test.

## Lateral Load Capacity of Deep Foundations

Resistance to lateral movement of the deep foundations will be provided by passive earth pressures. We recommend the following Table 3 be used for the ultimate passive fluid pressures and/or the lateral modulus of subgrade reaction for lateral load analysis. We recommend that the passive pressure be neglected in the upper 2 feet. The assumed average soil densities and internal friction angles ( $\phi$ ) are also included in Table 3.

**TABLE 2 – DESIGN PARAMETERS FOR LATERAL LOAD**

Type	Internal Friction Angle ( $\phi$ )	Soil Density (pcf)	Ultimate Equivalent Passive Fluid Pressures (psf/ft)	Lateral Modulus of Subgrade Reaction (pcf)
New Engineered Fill	28	110	305	8
Residual Soils	30	115	345	15
PWR	36	135	520	150

We note that considerable horizontal deflection is required to mobilize the passive earth condition, which is dependent on the soil type, consistency and the length of the pile. Therefore, the designer may wish to consider a safety factor of at least 2 to the stated ultimate passive earth pressures in design.

### Pile Construction Criteria

1. A full time Geotechnical Engineer is required to verify volume of grout placed with respect to volume of hole and to verify that installation procedures are correct and are compatible with installation procedures used for installation of test piles.
2. Do not install new piles within 6 feet of piles that have been filled with concrete less than 8 hours.
3. Volume of concrete placed in the pile hole should be at least 15 percent more than the theoretical volume of the augured hole.
4. At any depth, if grouting operation is interrupted, the auger should be advanced a minimum of 5 feet into existing grout when grouting operation is resumed to reduce probability of voids forming in grout column.
5. The Geotechnical Engineer should determine when adequate depths have been achieved and pile lengths meet the refusal criteria.
6. At the bottom of the pile, a minimum of 8 feet equivalent height of grout head shall be pumped prior to withdrawal of the auger. Withdrawal of auger should be slow enough to

maintain the cross section of the drilled hole and maintain the grout head well above the grout discharge location at the auger tip.

7. The grout pump should be a positive displacement pump capable of developing pressures of at least 350 pounds per square inch (psi). The pump should contain a grout pressure gauge and a stroke counter, both in good working order and in clear view of the operator. During grouting, the pressure of 150 to 250 psi displayed in the gage is commonly acceptable.
8. Grout mix design should be approved by the structural engineer. A flow rate of 15 to 28 seconds using a standard flow cone with  $\frac{3}{4}$ -inch orifice shall be achieved to maintain the fluidity of the grout mix. The test procedure generally corresponds to ASTM C-939, except that a  $\frac{3}{4}$ -inch orifice shall be used. Compressive strength of grout should be at least 4,000 psi at 28 days.

### ***Option 2: Intermediate Foundations – Geopiers/Vibro-piers***

United Consulting believes that the use of a properly designed and installed compacted stone-column system (Geopiers or Vibro-piers) would be a lower risk and lower settlement alternative to conventional shallow foundations bearing in soil. The bearing capacity and settlement of the Geopier/Vibropier system are a function of the on-site soils, the strength (modulus) of the compacted aggregate within the piers, the length of the piers, and the percentage of the foundation bearing area that is directly supported by the piers. As such, the actual design bearing pressure, pier locations and spacing, and predicted settlement values should be provided by the licensed Geopier/Vibropier design professional.

Geopiers and Vibropiers are proprietary ground improvement systems consisting of columns of compacted stone. Placement of Geopiers or Vibropiers involves using an auger (typically 30-inch diameter) to bore a hole to a desired depth. A layer of open-graded stone is then placed at the bottom of the hole and a tamper or vibrator is then inserted into the hole to work the stone into the soil at the base of the hole. This serves to pre-stress and densify the soil at the bottom of the cavity, creating a bulb of compacted soil and stone. Once this process is completed, an aggregate layer (typically #57 stone or crusher run) measuring approximately 12 inches thick is placed in the hole and densely compacted or vibrated with a specialized densification system. Additional layers of aggregate are inserted into the hole and compacted until the hole is filled. This compactive method consolidates the stone and the soils below and around it, so a group of Geopiers or Vibropiers creates a mass of competent soils. The competent soil mass distributes loads and reduces settlement. After the Geopiers or Vibropiers are installed, conventional shallow foundations are typically designed to bear directly on the aggregate pier reinforced soil mass.

For this project, we anticipate that a conventional shallow foundation system bearing on a properly designed and installed Geopier system may be designed for an allowable bearing pressure in the range of 4,000 to 6,000 psf with total settlement of the tank base of 1-inch or less and differential settlements of about 0.5 inches. The bearing capacity and settlement of the aggregate pier systems are a function of the on-site soils, the strength (modulus) of the

compacted aggregate within the aggregate piers, the length of the aggregate piers, and the percentage of the foundation bearing area that is directly supported by the aggregate piers. As such, the actual design bearing pressure, settlement estimates, pier locations, and lengths should be provided by the licensed Geopier or Vibropier Design Professional.

If Geopiers or Vibropiers are selected, United Consulting requests the opportunity to review the design for this project to determine if the design is consistent with our recommendations for this site. At least one load test should be performed at the site, in the presence of the geotechnical engineer to evaluate the performance of the design for this project.

We note that aggregate piers can be susceptible to damage; and excavations adjacent to existing piers might result in loss of foundation support. As such, efforts should be made to perform utility installations or other excavation work prior to pier installation. No excavations should be permitted within a 1 (H): 1(V) line extending down from the top of a completed aggregate pier, or within 5 feet of a completed pier (whichever distance is greater).

We recommend footing widths of at least 20 inches for strip footings. A minimum embedment depth of 12 inches is recommended to protect against frost action.

We recommend that a Geotechnical Engineer or his representative examine footing excavations. Each footing excavation must be evaluated by a representative of our firm prior to concrete placement to evaluate whether the exposed soils are comparable to the design values and, thus, suitable for support of the foundations. The conditions observed should be compared to test boring data and design requirements. The tops of the Geopiers/Vibropiers often become disturbed by construction activities including footing excavations. Footing evaluations should include evaluating the tops of the piers and the contractor should be prepared to re-compact the tops of the piers prior to concrete placement.

### **Earthwork**

The on-site soils, if free of organic materials, should generally be suitable for reuse as engineered fill with proper moisture control. We recommend that the contractor be equipped for both drying and wetting soils. Typical restrictions on suitable fill are no organics, plasticity index less than 30, and maximum particle size of four inches, with not more than 30 percent greater than  $\frac{3}{4}$ -inch. These restrictions should also be applied to the imported borrow soils, if needed.

Positive drainage should be maintained at all times to prevent saturation of exposed soils in case of sudden rains. Rolling the surface of disturbed soils will also improve runoff and reduce the fill soil moisture and construction delays.

## **Groundwater Considerations**

Groundwater was not encountered at the time of drilling. Therefore, we do not anticipate groundwater will significantly impact construction activities at the project site. However, due to presence of silty soils, the site may be susceptible to formation of perched water. The contractor should be prepared to remove perched water as needed.

## **Concrete Retaining Walls or Tank Walls**

### **Lateral Earth Pressures**

The following lateral earth pressure values pertain to concrete tank walls or cast-in-place retaining walls and are not intended for MSE wall construction. The lateral earth pressures imposed on a retaining wall or aboveground structures are a function of the soil parameters, surcharge behind the wall and the wall restraint. The most common conditions assumed for relatively flexible earth retaining structures are active and at-rest conditions. In any soil mass, the at-rest (or in-situ) earth pressure conditions generally change when excavations are made. If the wall is rigidly restrained against movement, the wall is subjected to at-rest earth pressure condition. If some movement of the wall away from the soil occurs, then the coefficient of earth pressure starts diminishing until the wall movement has fully mobilized the shear strength of the retained soil to a minimum value referred to as an active lateral earth pressure state. On the contrary, if the wall movement is towards the soil, the coefficient of earth pressure increases to a maximum value referred to as passive lateral earth pressure state.

Based on the boring data, and our experience with similar soils and subsurface conditions, we recommend the following earth pressure coefficients and equivalent fluid pressures (EFP) for the design of retaining walls.

**TABLE 1 - SUMMARY OF ULTIMATE EQUIVALENT FLUID PRESSURES  
(Retaining Walls or Tank Walls)**

<b>Pressure Conditions</b>	<b>Coefficients of Earth Pressure</b>	<b>Ultimate Equivalent Fluid Pressure (Soil Backfill)</b>
Active( $K_a$ )	0.33	40 psf/ft
At-rest( $K_o$ )	0.50	60 psf/ft
Passive( $K_p$ )	3.0	360 psf/ft

The parameters listed above were calculated by the Rankine method based on an assumed soil unit weight of 120 pcf, an angle of internal friction of 30 degrees, and zero effective cohesion for engineered fill soil backfill, no friction at the wall-soil interface, and no surcharge effects.

To calculate resistance to sliding, a friction factor of 0.36 (already includes a safety factor of 1.5) may be used as the coefficient of friction between wall foundations and the underlying residual soils. If Geopiers or Vibropiers ground improvement system are installed directly below the tank slab foundation, an increased coefficient of sliding friction of up to 0.5 is typically available. However this value would have to be verified by the licensed Geopier/Vibropier design professional.

The equivalent fluid pressures listed are based on the presumptions of a level backfill, no surcharge effects, and that a functioning drainage system will be provided behind the walls to prevent buildup of hydrostatic pressure. Because significant wall movement (typically 0.01 to 0.02 times the wall height) is required to develop passive earth pressure, it is recommended that a safety factor of 2 be used in design for passive earth pressure conditions.

### **Fill Placement**

Moisture-density determinations should be performed for each soil type used, to provide data necessary for quality assurance testing. The natural moisture content at the time of compaction should be within moisture content limits, which will allow the required compaction to be obtained. The contractor should be prepared to increase or decrease soil water content as necessary. The fill should be placed in thin lifts and compacted. We recommend that fill be compacted to at least 98% of Standard Proctor (ASTM D 698) maximum dry density within the top two feet below pavement subgrade or floor slabs and at least 95% of the Standard Proctor maximum dry density elsewhere. A representative of our firm should monitor fill placement on a full time basis. In place density tests performed by that individual will evaluate the degree of compaction being attained.

## **LIMITATIONS**

This report is for the exclusive use of **AKEA, Inc.** and the designers of the project described herein, and may only be applied to this specific project. Our conclusions and recommendations have been prepared using generally accepted standards of Geotechnical Engineering practice in the State of North Carolina. No other warranty is expressed or implied. Our firm is not responsible for conclusions, opinions or recommendations of others.

The right to rely upon this report and the data within may not be assigned without UNITED CONSULTING'S written permission.

The scope of this evaluation was limited to an evaluation of the load-carrying capabilities and stability of the subsoils. Oil, hazardous waste, radioactivity, irritants, pollutants, molds, or other dangerous substance and conditions were not the subject of this study. Their presence and/or absence are not implied or suggested by this report, and should not be inferred.

Our conclusions and recommendations are based upon design information furnished us, data obtained from the previously described exploration and testing program and our past experience.

They do not reflect variations in subsurface conditions that may exist intermediate of our borings and in unexplored areas of the site. Should such variations become apparent during construction, it will be necessary to re-evaluate our conclusions and recommendations based upon “on-site” observations of the conditions.

If the design or location of the project is changed, the recommendations contained herein must be considered invalid, unless the changes are reviewed by our firm, and our recommendations are either verified or modified in writing. When design is complete, we should be given the opportunity to review the grading plan, and applicable portions of the specifications to see if they are consistent with the intent of our recommendations.

**UNITED CONSULTING**



## **APPENDIX**

Exploration Procedures  
General Notes/Narrative of Drilling Operations  
Figure 1 - Boring Location Plan  
Boring Logs (2)

## **EXPLORATION PROCEDURES**

Standard Penetration Test (SPT) borings were drilled at this project site at the approximate locations shown on the attached Boring Location Plan (Figure 1). Soil samples obtained using the split spoon sampler were examined by the Geotechnical Engineer and classified according to the visual-manual procedure described in ASTM D 2488-00. Soil test borings were performed in general accordance with ASTM D 1586. A narrative of field operations is included in the Appendix.

The boring locations were determined in the field by our engineering representative who measured distances and estimated angles with a measurement tape and a hand-held compass from existing site features. Therefore, the location of the borings as indicated on the boring location plan should be considered approximate. The elevations provided on the boring logs were obtained from the provided topographic site plan by interpolation and should be considered very approximate.

## GENERAL NOTES

The soil classifications noted on the Boring Logs are visual classifications unless otherwise noted. Minor constituents of a soil sample are termed as follows:

Trace	0 - 10%
Some	11 - 35%
Suffix "y" or "ey"	36 - 49%

### LEGEND



Split Spoon Sample obtained during Standard Penetration Testing



Relatively Undisturbed Shelby Tube Sample



Groundwater Level at Time of Boring Completion



Groundwater Level at 24 hours (or as noted) after Termination of Boring

w                      Natural Moisture Content

LL                      Liquid Limit

PL                      Plastic Limit                      Atterberg Limits

PI                      Plasticity Index

PF                      Percent Fines (Percent Passing #200 Sieve)

$\gamma_d$                       Dry Unit Weight (Pounds per Cubic Foot or PCF)

$\gamma_m$                       Moist or In-Situ Unit Weight (PCF)

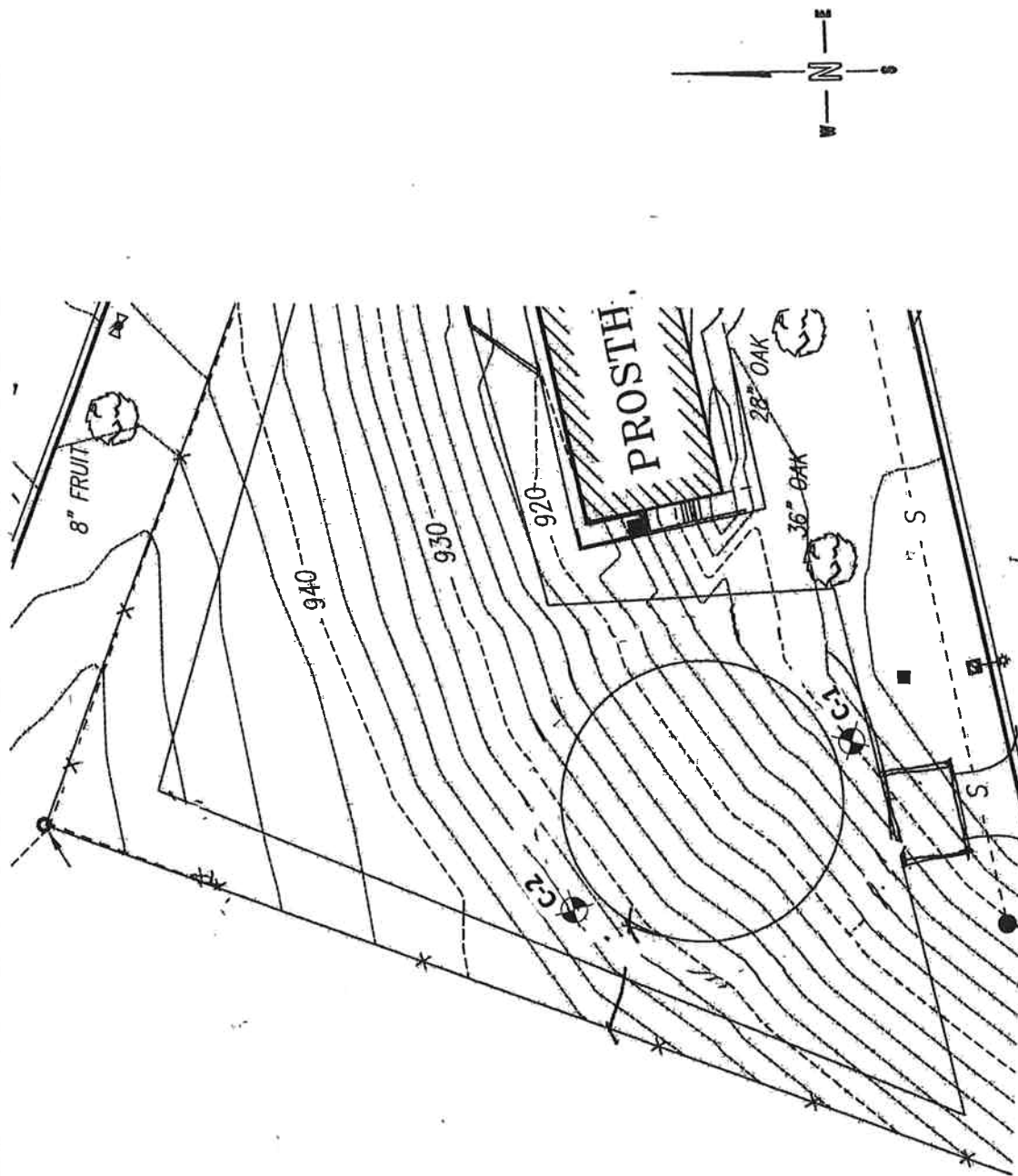
$\gamma_{sat}$                       Saturated Unit Weight (PCF)

## **BORING LOG DATA AND NARRATIVE OF DRILLING OPERATIONS**

The test borings were made by mechanically advancing helical hollow stem augers into the ground. Samples were covered at regular intervals in each of the borings following established procedures for performing the Standard Penetration Test in accordance with ASTM Specification D-1586. Soil samples were obtained with a standard 1.4" I.D. x 2.0" O.D. split barrel sampler. The sampler is first seated 6" to penetrate any loose cuttings and then driven an additional foot with the blows of a 140 pound hammer freely falling a distance of 30." The number of blows required to drive the sampler each six inches is recorded on the Boring Logs. The total number of blows required to drive the sampler the final foot is designated the "standard penetration resistance." This driving resistance, known as the "N" value, is a measure of the relative density of granular soils and is an indication of the consistency of cohesive deposits.

The following table describes soil consistencies and relative densities based on standard-penetration resistance values (N) determined by the Standard Penetration Test.

	"N"	Consistency
Clay and Silt	0-2	Very Soft
	3-4	Soft
	5-8	Firm
	9-15	Stiff
	16-30	Very Stiff
	Over 31	Hard
	"N"	Relative Density
Sand	0-4	Very Loose
	5-10	Loose
	10-19	Firm
	20-29	Medium Dense
	30-49	Dense
	50+	Very Dense




SCALE: N/A	DATE: 11/18/10	PROJECT NO: 2010.0241.02	TITLE: BORING LOCATION PLAN  PROPOSED ABOVEGROUND TANK, ATLANTA VA HOSPITAL DECATUR, DEKALB COUNTY, GEORGIA	
PREPARED: KK	CHECKED:			
CLIENT:	AKEA, INC.	UNITED CONSULTING 625 Holcomb Bridge Road, Norcross, GA 30071 Tel. 770-209-0029 FAX 770-582-5900 www.unitedconsulting.com		

FIG. 1

## BORING LOG

CONTRACTED WITH: AKEA, INC

BORING NO.: C-1










PROJECT NAME: ABOVEGROUND TANK

DATE: 11-08-10

JOB NO.: 2010.0241.02 DRILLER: GEORGE

RIG: CME 45

LOGGED BY: MH

ELEV.	DESCRIPTION	DEPTH in FEET	SAMPLES					NOTES
			NO.	TYPE	BLOWS/6"	RECOV.	W%	
910	8" TOPSOIL/GRASS	0						No groundwater encountered at the time of boring
	Sand-silty, trace clay, mica and root hair; firm; tan (Residual)  -loose		1		2-6-8	14"		
		2		4-4-3	12"			
905		5						
		-some mica; medium dense; light tan						
900	10	3		12-13-12	14"			
	-dense; white							
895	15	4		16-14-16	12"			
	-trace rock fragments; orange brown							
890	20	5		22-19-20	18"			
	Partially Weathered Rock sampled as sand-some silt and mica; very dense; gray							
885	25	6		16-22-50/4	10"			
880	30	7		10-44-50/3	10"			
875	35	8		46-31-50/2	12"			
870	40	9		50/2	10"			
	AUGER REFUSAL AT 40 FEET							



**UNITED CONSULTING**  
625 HOLCOMB BRIDGE ROAD  
NORCROSS, GEORGIA 30071  
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Sheet 1 of 2

## BORING LOG

CONTRACTED WITH: AKEA, INC

BORING NO.: C-2

PROJECT NAME: ABOVEGROUND TANK

DATE: 11-08-10

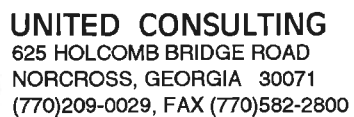
JOB NO.: 2010.0241.02 DRILLER: GEORGE

RIG: CME 45

LOGGED BY: MH

ELEV.	DESCRIPTION	DEPTH in FEET	SAMPLES				NOTES
			NO.	TYPE	BLOWS/6"	RECOV.	
935	4" TOPSOIL/GRASS	0					
	Sand-silty, trace clay, mica and root hair; loose; orange (Residual)		1		4-4-6	18"	
	-medium dense; tan		2		12-12-13	14"	
930		5					
	-gray and white		3		8-9-11	14"	
925		10					
	-orange		4		7-11-10	12"	
920		15					
	-dense		5		16-15-16	18"	
915		20					
	-some silt		6		21-28-21	14"	
910		25					
	Partially Weathered Rock sampled as sand-some silt and mica; very dense; tan		7		50/5	4"	
905		30					
			8		25-50/5	10"	
900		35					
			9		50/4	4"	
895		40					





BORING NO.: C-2

DATE: 11-08-10

LOGGED BY: MH

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