1112 Manatee Ave. West
Bradenton, FL 34205
purchasing@mymanatee.org

# Solicitation Addendum

| | |
|---|---|
| Addendum No.: | 3 |
| Solicitation No.: | **19-TA003164AJ** |
| Project No.: | **6091380, 6091480, 6091580** |
| Solicitation Title: | **NORTH WATER RECLAMATION FACILITY HEADWORKS, CLARIFIERS AND CHLORINE CONTACT CHAMBER REHABILITATION IMPROVEMENTS** |
| Addendum Date: | **December 23, 2019** |
| Procurement Contact: | **Abby Jenkins** |

**19-TA003164AJ is amended as set forth herein. Responses to questions posed by prospective bidders are provided below. This addendum is hereby incorporated in and made a part of IFBC 19-TA003164AJ.**

**Change to:**

**DATE, TIME AND PLACE DUE:**
The Due Date and Time for submission of Bids in response to this IFBC **is January 14, 2020 at 3:00 P.M. ET.** Bids must be delivered to the following location: Manatee County Administration Building, 1112 Manatee Ave. W., Suite 803, Bradenton, FL 34205 prior to the Due Date and Time.

**Change to:**

**SECTION A, INFORMATION FOR BIDDERS, A.O1, BID DUE DATE**
The Due Date and Time for submission of Bids in response to this Invitation for Bid (IFBC) **is January 14, 2020 at 3:00 P.M. ET.** Bids must be delivered to the following location: Manatee County Administration Building, 1112 Manatee Ave. W., Suite 803, Bradenton, FL 34205 and time stamped by a Procurement representative prior to the Due Date and Time.

**Change to:**

**SECTION A, INSTRUCTIONS TO PROPOSERS, PARAGRAPH A.51, SOLICITATION SCHEDULE**

| Scheduled Item | Scheduled Date |
|---|---|
| Bid Response Due Date and Time | January 14, 2020 @ 3:00 PM |
| Due Diligence Review Completed | January 2020 |
| Projected Award | February 2020 |

**Add:**
**BID ATTACHMENT 2, TECHNICAL SPECIFICATIONS, SECTION 02720A, BYPASS PUMPING**

Add Section 02720A, bypass pumping to Bid Attachment 2, Technical Specifications that is issued with this Addendum 3.

**Change to:**
**BID ATTACHMENT 2, TECHNICAL SPECIFICATION, SECTION 06910, 1.02, WARRANTY A.**

Change section 1.02.A in Specification 06910 as follows:

<u>Manufacturer shall warrant its products in accordance with Specification Section 01030</u>**.**

**Change to:**
**BID ATTACHMENT 2, TECHNICAL SPECIFICATION, SECTION 06930, 1.05(6), WARRANTY.**

Change section 1.05(6) in Specification 06930 as follows:

<u>Manufacturer shall warrant its products in accordance with Specification Section 01030</u>**.**

**Change to:**
**BID ATTACHMENT 2, TECHNICAL SPECIFICATION, SECTION 09900, 1.03(A), GUARANTEE.**

Change section 1.03(A) in Specification 09900 as follows:

<u>Manufacturer shall warrant its product in accordance with Specification Section 01030.</u> Failure of any coating during the guarantee period shall be repaired by the Contractor who shall absorb all costs related to the repair of the coating. Failure shall be defined as peeling, blistering, delamination or loss of adhesion of any of the coatings.

**Change to:**
**BID ATTACHMENT 2, TECHNICAL SPECIFICATION, SECTION 11300, 1.01(G), MANUFACTURER WARRANTY.**

Change section 1.01(G) in Specification 11300 as follows:

<u>Manufacturer shall warrant its products in accordance with Specification Section 01030</u>**.**

**Replace:**
**BID ATTACHMENT 3, NWRF IMPROVEMENTS- CONSTRUCTION PLANS, PLAN SHEET NUMBERS HW-M-3 AND HW-M-4.**

Replace Bid Attachment 3 Plan Sheet numbers HW-M-3 and HW-M-4 with the Revised Plan Sheet numbers HW-M-3 and HW-M-4 that are issued with this Addendum 3.

**Add:**
**BID ATTACHMENT 4, GEOTECHNICAL REPORT**

Bid Attachment 4 Geotechnical Report, issued with this Addendum 3, is hereby incorporated into the IFBC.

**Add:**

**THE FOLLOWING ITEMS ARE ISSUED WITH THIS ADDENMDUM 3 FOR INFORMATIONAL PURPOSES ONLY.**

1. North Sub-regional Wastewater Treatment Facilities Record Drawing Sheets T65 through T67 dated 12/19/1989.

**Questions and Answers:**

**Q1. 111000.2.01.D.1.c: Hydro's standard flow meter is acrylic body with 316SS stainless steel float and guide rod, cut sheet attached. Will our standard FM be acceptable?**

R1. Yes, the standard flow meter will be acceptable.

**Q2. 11100.2.05.A.2.g: The logic is specified here as programmable relay which disagrees with the requirement for PLC logic in section 2.05.A.5. Please confirm which logic is required?**

R2. Logic shall be via a Programmable Logic Controller (PLC).

**Q3. 11100.2.05.A.3 requires controls for the grit pump to be in the grit system panel. Drawings HW-E-2 and -3 shows notes about having a separate pump panel. Please confirm all grit pump controls will be mounted in the grit system control panel.**

R3. The logic to operate the Grit Pumps automatically shall reside in the Grit Removal Control Panel. The Local Control Panels also allow for manual control of the motors locally.

**Q4. 11100 doesn't list an Operator Interface model. Will the standard Allen-Bradley 4" display for set point changes be acceptable?**

R4. Yes, a standard Allen-Bradley 4" display will be acceptable.

**Q5. Drawing HW-M-6: The HeadCell weir elevation is called out at 53.54' on this sheet but the hydraulic profile on sheet G-8 calls out the weir elevation at 52.09', which is correct?**

R5. The correct headcell weir elevation is 53.54'.

**Q6. Based on the HeadCell tank invert elevation shown at 30.48', the weir elevation should be at a minimum elevation of 50.88'. Hydro estimates the head over the 16' long weir at the peak flow of 22.5 mgd would be ~9". Adding this to the HeadCell unit headloss of 12" at peak flow suggests the upstream water elevation would be 21" (1.75') above the HeadCell weir elevation. The elevation of the top of the HeadCell inlet duct will need to be no less than the upstream water elevation at peak flow. This was Hydro's first opportunity to review the elevation layout of the HeadCell, we don't recommend placing the weir elevation so high if not needed.**

R6. This project is intended to match existing Hydro Headcell conditions. The existing Headcell inlet duct elevation is unknown, therefore Hydro International shall follow the previous installation measurements.

**Q7. If bypass pumping is required at the Anoxic/Aeration Basin, please provide the required pumping rate.**

R7. If bypass pumping is required, the average flow rate conditions are 4.5 MGD (3,125 GPM) with peak flow conditions of 18 MGD (12,500 GPM).

**Q8. Please provide the anticipated flow rate required for bypassing pumping at the Clarifier Flow Splitter Box**.

R8. Average flow rate conditions are 4.5 MGD (3,125 GPM) with peak flow conditions of 18 MGD (12,500 GPM).

**Q9. Please provide the required bypass pumping rate for the CC Chamber bypass shown on Drawing G6.**
R9. See response to question 8 above.

**Q10. Does the contractor have to provide a man watch during bypass pumping operations, or is a telemetry system adequate since the NWRF has staff 24 hours per day?**
R10. A man watch shall be required during bypass pumping operations. Please refer to attached Specification 02720A for details.

**Q11. Are there any redundancy requirements for each of the bypass pumping operations?**
R11. Please refer to attached Specification 02720A for details.

**Q12. How long will it take for the Owner to drain each of the Chlorine Contact Chambers?**
R12. The Contractor shall be responsible for draining the chlorine contact chambers through coordination with onsite operation staff.

**Q13. How long will it take the owner to drain the Anoxic Basin?**
R13. The anoxic basin does not have to be drained for the replacement of the gates.

**Q14. How long will it take for the Owner to drain each Clarifier?**
R14. The Contractor shall be responsible for draining the clarifiers through coordination with onsite operation staff. Each clarifier has a dedicated drain line to the onsite sewer system.

**Q15. Please provide a cross section of the Clarifier Splitter Box.**
R15. See the attached Record Drawing Sheet number T67 for the clarifier splitter box record drawings.

**Q16. Will the grout need be removed and replaced in the existing Clarifiers 1 and 2?**
R16. The grout on Clarifiers #1 and #2 floors will remain in place.

**Q17. Are there any Geotech Reports for this site that confirm the water level at the Clarifiers and the Chlorine Contact Chambers since we are repairing the existing Pressure Relief Valves in Clarifiers 1 and 2? This information is needed to determine the extent of any dewatering that might be required.**
R17. No, Geotech exploration was performed as part of this project, see Attachment 4 Geotech report performed for the construction of the Equalization Tanks.

**Q18. Please clarify the limits of the interior of the Clarifier structures that are to be coated per Drawing CL-M-2 note "6. Clean Abrasive Blast and coat all tank interior concrete surfaces, including launder troughs and recessed sump, per specifications section 09900 "Concrete Surfaces – Immersion". Does the entire Clarifier wall from the floor to the top need to be coated?**
R18. The note on CL-M-2 identifies that all of the clarifier's interior concrete surfaces shall be cleaned and coated. This includes all interior concrete surfaces, excluding the clarifier floor.

**Q19. Can the Odor Control System be taken off-line during the replacement of the FRP Dampeners?**
R19. Yes, the Odor Control System can be taken off-line during the replacement of the FRP dampeners.

**Q20. Please identify the Owner's System Integrator that will be performing work on this project under a separate contract per specification 13300 1.02-C.**
R20. The County will select the system integrator prior to construction. No firm is selected at this point in time.

**Q21. Please confirm whether or not a hazardous material survey has been done, and if it is anticipated that any hazardous material will be encountered in this scope of work.**

R21. A hazardous material survey has not been done and it is not anticipated that any hazardous material will be encountered on this project.

**Q22. Please confirm that all warranties are 3 years per Specification 01030 Section 1.13 Warranties and not per the individual specification sections. For example, Section 06910 FRP Weirs and Scum Baffles 1.02 Warranty calls for a 1-year warranty, Section 06930 FRP Dampeners Section 1.05-1 have a 2-year warranty, and Section 09900 Section 1.03 Guarantee calls for a 1-year Warranty.**

R22. All warranties shall be a 3-year warranty per Specification 01030.

**Q23. Can the bid form be revised to breakout and quantify the joint repair on a per lineal foot basis?**

R23. No, bid form is final.

**Q24. Are there any E fixtures in the drawings?**

R24. E fixtures shall be installed in lieu of fixture type 'F' shown in CCC area. No Fixture type 'F' (Lithonia QTE) will be required.

**Q25. Are fixtures A, B and C supposed to be provided with square poles per fixture schedule (Drawings E-3) or do we follow the drawing notes and existing poles to remain (Drawing HW-E-4 Note 2, 3, and 4)?**

R25. Fixture schedule (Drawing E-3) requires that a square pole universal mounting adapter be provided (not a square pole). New poles shall not be required. The Contractor shall be responsible for providing all materials necessary to install the new fixtures on the existing poles.

**Q26. Is there any intent to add an ultrasonic unit to the 3rd clarifier?**

R26. No, only two clarifiers will be included in this project.

**Q27. Does each transducer head for the algae unit get 20 meters of cable for a total of 40 meters per clarifier or is the 20 meters of cable sufficient for both transducer heads for a total of 20 meters per clarifier?**

R27. Each Clarifier requires two transducer heads and one control box. Each transducer head required shall have an individual cable connection to the associated control box. Cable distances vary and shall be determined by the Contractor.

**Q28. 11100.2.02.A.5 – To size our panel correctly, please confirm the TDH of 31 feet for the grit pumps. See the TDH flyer attached to assist with the calculation. The engineer will just need to include the estimated minor/major friction losses to finalize.**

R28. The TDH has been revised to 30 feet for the grit pumps.

**Q29. Is it the intent for the bypass pumps at the North Effluent Pump Station to pump from the North CCC into the South Effluent Pump station or are they intended to manifold into the distribution piping due to the limited number of pumps available in the South Pump Station? If it is to be manifolded into distribution, please provide the required pressure in addition to the flow rate for the bypass pumps.**

R29. The Contractor shall manifold the bypass pump into the reclaimed water distribution piping by connecting to the existing 12" butterfly valve (referring to Sheet G-6, the valve is located in the third header when counting from the south.) To connect to the valve, the Contractor shall disconnect the 12" piping and appurtenances; and reassemble the header after the bypass is done. The anticipated pressure is 85 psi with average flow rate conditions of 4.5 MGD (3,125 GPM) and peak flow conditions of 18 MGD (12,500 GPM).

**Q30. Drawing AX-D-1 shows the proposed gate modifications to be made at the Anoxic /Aeration Basin. Please provide the allowable duration that one of the Anoxic/Aeration Basins can be out of service.**

R30. The allowable duration that one of the anoxic/aeration basin can be out of service is 14 days.

**Q31. Is the Owner responsible for cleaning any residuals or material anticipated to be in the Clarifiers, Clarifiers Splitter Box or Aeration/Anoxic Basin Splitter Box? If not, please provide an anticipated quantity and confirm whether they can be processed through the Plant or if they need to be disposed of offsite?**

R31. It is not anticipated that grit will be encountered in the clarifiers, clarifier splitter box, or the aeration/anoxic splitter box. However, if the Contractor encounters any inorganic material, such as rags or debris, the Contractor shall be responsible for disposal. This material can be disposed of in the onsite dumpster located at the Headworks.

**Q32. Please provide the information that is necessary to comply with the ADA Section 504 requirements mentioned in the Pre-Bid Meeting.**

R32. Section 508 of the Rehabilitation Act and best practices (W3C WCAG 2) is a set of regulations that provide accessibility for digitally published material within government agencies and members of the public with disabilities.

NOTE: Items that are ~~struck through~~ are deleted. Items that are underlined have been added or changed. All other terms and conditions remain as stated in the IFBC.

**End of Addendum**

**INSTRUCTIONS**:
Receipt of this addendum must be acknowledged as instructed in the solicitation document. Failure to acknowledge receipt of this Addendum may result in the response being deemed non-responsive.

AUTHORIZED FOR RELEASE Chris Daley, CPPO, CPPB

Digitally signed by Chris Daley, CPPO, CPPB
DN: cn=Chris Daley, CPPO, CPPB, o=manatee County Government, ou=Procurement Manager, email=chris.daley@mymanatee.org, c=US
Date: 2019.12.23 13:56:10 -05'00'

# SECTION 02720A

# BYPASS PUMPING

## PART 1 GENERAL

### 1.01 SCOPE

The Contractor shall furnish all labor, materials, equipment and incidentals required to maintain existing and anticipated flows of the treatment facility throughout the construction period.

### 1.02 PUBLIC IMPACTS

The contractor shall not create a public nuisance due to excessive noise or dust, nor impact the public with flooding of adjacent lands, discharge of raw sewage, or release of other potential hazards, nor shall he encroach on or limit access to adjacent lands. No extra charge may be made for increased costs to the Contractor due to any of the above.

### 1.03 SUBMITTALS

A. The Contractor shall, within 30 days of the date of the Notice to Proceed, submit to the Project Manager a detailed Pumping Plan for each site that by-pass pumping will be needed. The Pumping Plan shall address all measures and systems to prevent any overflows. The Plan shall include as a minimum:

1. Working drawings and sketches showing work location, pump location, piping layout & routing. Show all proposed encroachment and access impacts on adjacent properties or facilities.
2. Pump, control, alarm and pipe specifications or catalog cuts. Detailed sketch of controls and alarm system.
3. Power requirements and details on methods to provide by-pass power or fueling.
4. Calculation and determination of response times to prevent an overflow after a high water alarm. If anticipated peak flows are 750 G.P.M. or greater, an operator is required on site at all times pump is in service. If the anticipated peak flows are less than 750 G.P.M. an operator may not be required to be on site at all times; show operator on-site schedule.
5. Procedures to be taken in case of power, pump, or piping failures; including contact names and numbers for emergency notifications.
6. Frequency and specific responsibility for monitoring pump operation, fuel levels, pump maintenance and entire length of piping.

## PART 2 PRODUCTS

### 2.01 EQUIPMENT

A. Pumps:

1. By-pass pumping system shall consist of at least a primary pump and a backup pump.

2. Pumps shall be low noise or sound attenuated. The noise level at any operating condition, in any direction, shall not exceed 70dBA at a distance of twenty three (23) feet (7 meters) from the pump and/or power source.

B. Controls:

The by-pass pump system shall be equipped with automatic controls and an alarm system. The automatic controls will automatically start the backup pump in the event of a high water condition or failure of the primary pump. The alarm system will immediately notify the Contractor of a pump failure or high water condition.

C. Pipe:

Pipe shall be of adequate size and capacity to prevent any overflow. Pipe type and materials will depend on the particulars of the site conditions, and shall be detailed in the Pumping Plan. Contractor will provide all connections.

## PART 3 EXECUTION

### 3.01 SITE CONDITIONS

Site conditions will vary by site. Contractor is responsible to determine and address requirements such excavation, connections & fittings, impacts on access to adjacent properties, routing and support of by-pass piping, etc., in the Pumping Plan.

### 3.02 ON-SITE MONITORING

A. All by-pass operations where the anticipated flow rates are 750 G.P.M or greater shall require an employee on-site at all times (full-time on-site monitoring attended by personnel experienced with the pumps and controls, with demonstrated ability to monitor, turn on & off, and switch between pumps while the by-pass pump system is in service.

B. By-pass operations where the anticipated flow rates are less than 750 G.P.M may not require an employee on-site at all times while the by-pass pump system is in operation. The Contractor shall have personnel experienced with the pumps and controls on site within the calculated response time to prevent an overflow after a high water alarm.

C. During by-pass operations, the Contractor shall have posted on site a copy of the approved Plan and the name and 24 hour contact number of the primary response person, the job site superintendent, and the construction company owner.

### 3.03 OPERATIONS

A. The Contractor is responsible for securing and providing power, fuel, site security, traffic control and all other supplies, and materials required for the by-pass pumping.

B. Contractor shall demonstrate automatic pump switching and alarm system to the satisfaction of: the County inspector, or Project Manager. Satisfactory demonstration shall be documented by the inspector's or PM's dated signature on the posted copy of the approved Pumping Plan.

## 3.04 DAMAGE RESTORATION & REMEDIATION

A. The Contractor shall be responsible for any pre-pump notifications, all restoration of pre-pump conditions and any damage caused by by-pass operations.

B. Should there be an overflow caused by or as a direct result of the by-pass pumping, the Contractor is responsible for all immediate & long term response, notifications, clean up, mitigation, etc. Copies of all written response plans, notifications, documentation, mitigation plans, etc., shall be submitted to the County Project Manager.

END OF SECTION

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**ATTACHMENT 4**
**GEOTECHNICAL REPORT**

# SUBSURFACE SOIL EXPLORATION AND GEOTECHNICAL ENGINEERING EVALUATION PROPOSED EQUALIZATION TANKS AT MANATEE COUNTY NORTH WRF, 8500 69TH STREET EAST, ELLENTON, MANATEE COUNTY, FLORIDA

Ardaman & Associates, Inc.


**OFFICES**

**FLORIDA**
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**Miami,** 2608 W. 84th Street, Hialeah, Florida, 33016, Phone (305) 825-2683
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Ardaman & Associates, Inc.

Geotechnical, Environmental and
Materials Consultants

(revised May 20, 2016)
February 25, 2016
File No. 15-7337

TO: Kimley-Horn & Associates, Inc.
655 North Franklin Street, Suite 150
Tampa, FL 33602

Attention: W. Wade Wood, III, P.E.

SUBJECT: Subsurface Soil Exploration and Geotechnical Engineering Evaluation
Proposed Equalization Tanks at Manatee County North WRF,
8500 69th Street East, Ellenton, Manatee County, Florida

Dear Wade:

As requested and authorized, Ardaman & Associates has completed a subsurface soil exploration program at the site referenced above. Our services were provided in general accordance with those outlined in our proposals dated July 31, 2015, and February 9, 2016 (Kimley-Horn IPO Nos. 38 and 40). The purpose of this program was to evaluate the subsurface soil conditions and provide recommendations for site preparation and foundation design for three 1.0 million gallon (MG) equalization tanks and an associated electrical/storage building.

This report has been revised to include the results of laboratory consolidation tests and the revised settlement analyses for the equalization tanks. The analysis in this report supersedes the recommendations in our previous report.

This revised report documents our findings and presents our engineering recommendations. It has been prepared for the exclusive use of Kimley-Horn & Associates for specific application to the subject project, in accordance with generally-accepted geotechnical engineering practices.

## PROJECT INFORMATION

### Equalization Tanks

We understand that three 1.0 MG equalization tanks are proposed, and that each tank is approximately 87 feet in diameter and 25 feet tall. The tanks are to be supported upon a reinforced concrete mat foundation at the existing ground surface elevation, so that the entire 25 feet tank height will be above existing grade (i.e. the tanks will not be partially buried below grade).

Based upon information provided by Kimley-Horn, we understand that the maximum soil bearing pressure from the tanks on the foundations will be no greater than 1,825 pounds per square foot (lb/sq ft).

We have also assumed that only minimal (less than 1 foot) of fill will be placed surrounding the tanks and that no fill we be necessary beneath the mat foundation. If actual foundation loads or fill height exceed our assumptions, then the recommendations in this report may not be valid.

**Electrical/Storage Building**

We understand that this will be a one-story building that is approximately 30 feet by 50 feet in "footprint" plan dimensions, with a wall height of 12 feet. The proposed building will include load bearing walls and interior columns with a slab-on-grade ground floor.

The maximum foundation loads for the proposed structure were not available at the time of this report. However, based on our experience with similar projects, the maximum loads are expected to be as follows:

| | |
|---|---|
| Wall Load: | 1 to 3 kips per linear foot (kip/ft) |
| Column Load: | 40 kips |
| Floor Load: | 200 pounds per square foot (lb/sq ft) |

We have also assumed that less than 2 feet of fill will be required to achieve the ground (finished) floor elevation. If actual building loads or fill height exceed our assumptions, then the recommendations in this report may not be valid.

## SITE LOCATION AND CONDITIONS

The subject site is located northeast of Ellenton, in Manatee County, Florida. More specifically, the site is located at 8500 69$^{th}$ Street East, near the intersection of 69$^{th}$ Street East with Erie Road. The proposed equalization tank areas are shown on the attached Figure 1.

The site is an active water reclamation facility (WRF) with several existing buildings and tank structures. At the time of our field explorations, the area of the proposed equalization tanks and electrical/storage building site was mostly clear with a short grass cover and some driveways, but also with an existing structure on the southern portion of the area. The proposed electrical/storage building is to be located at our boring location No. 5, which is shown on Figure 1.

The USGS topographic survey map for the site vicinity (Parrish, Florida Quadrangle, dated 1973, (photo-revised 1987) was reviewed for ground surface features at the proposed project location. Based on this review, the natural ground surface elevation is in the range of +25 to +30 feet National Geodetic Vertical Datum of 1929 (NGVD). The natural ground surface appears to be relatively flat, but with a gentle slope generally from the south downward to the north or east.

## REVIEW OF SOIL SURVEY MAPS

Based on the U.S. Department of Agriculture, Soil Conservation Service (now the Natural Resources Conservation Service) "Soil Survey of Manatee County, Florida," the site is located in an area mapped primarily as the "EauGallie fine sand" soil series, transitioning to an area of "Floridana fine sand" to the north.

The "EauGallie fine sand" consists of a nearly level, poorly drained soil in broad flatwood areas. A typical soil profile consists of fine sand from the ground surface to a depth of 3.5 feet, underlain by sandy clay loam to 4.2 feet, then fine sand to sandy loam to 5.4 feet. According to the Soil Survey, during most years the water table is at a depth of less than 10 inches below the natural ground surface for about 2 to 4 months and within a depth of 40 inches for more than 6 months of the year.

The "Floridana fine sand" consists of nearly level, very poorly drained soil in low flats that have been drained by ditches or channels in many places. Slopes are smooth to concave and are less than 2 percent. A typical soil profile consists of a surface layer of black to very dark gray fine sand about 15 inches thick, underlain by gray fine sand to a depth of 32 inches, then by dark gray sandy clay loam to a depth of 44 inches and gray sandy loam to a depth of 65 inches. The substratum is a light gray fine sand to a depth of 80 inches or more. According to the Soil Survey, during most years and if the soil is not drained, the water table is at a depth of less than 10 inches below the natural ground surface for about 6 months out of the year.

## FIELD EXPLORATION PROGRAM

Our field exploration program included conducting five (5) Standard Penetration Test (SPT) borings at the locations shown on the attached Figure 1. These borings were performed on January 5 to 8, 2016. The borings in the tank areas (Boring Nos. 1 to 4) were performed to determine the nature and condition of the subsurface soils to a depth of 90 feet below the existing ground surface, and the boring in the proposed building area (boring No. 5) was performed to a depth of approximately 40 feet. The SPT soil borings were initially drilled to a depth of 4.5 feet with a hand auger at each boring location, in an effort to avoid damaging possible underground utilities. The equipment and procedures used in the borings are described in Appendix I of this report.

Relatively undisturbed, thin-walled (Shelby) tube soil samples were obtained on March 31, 2016. These were obtained to supplement the data obtained from the previous SPT borings, in order to better define soil properties related to settlement of the tank structures. These were obtained by performing a rotary-wash boring to a depth of 25 feet, then sampling the clay and clayey soils beneath this depth. Additional information on undisturbed sampling is included in Appendix I.

Test borings were located in the field utilizing an aerial photograph of the site and visual reckoning to available landmarks. The locations should be considered accurate only to the degree implied by the method used. Should more accurate locations be required, a registered land surveyor should be retained.

## LABORATORY TESTING

The field soil boring logs and recovered soil samples were transported to our Sarasota office following the completion of the field exploration activities. Each representative sample was examined by a geotechnical engineer in our laboratory for visual classification and assignment of laboratory tests, if deemed necessary to aid in classification or to better define engineering properties.

The laboratory tests performed on the SPT “split spoon” samples included determining the fines (silt and clay) content and water (natural moisture) content of selected samples. The test results are presented on the graphic soil profiles on Figures 2 and 3, at the depth from which the respective sample was recovered.

The laboratory tests performed on the Shelby tube samples included one clay consolidation test, plus determining the fines (silt and clay) content, water (natural moisture) content and unconfined compressive strength of the clayey specimens by pocket penetrometer. These test results are included in Appendix II of this report.

The tests were performed in accordance with the applicable ASTM standards, which are listed in the Appendix. The soil descriptions shown on the soil profiles are based on the laboratory test results and a visual classification procedure in general accordance with the Unified Soil Classification System (ASTM D-2487 or D-2488).

## SUBSURFACE SOIL CONDITIONS

The general subsurface soil conditions encountered during the field exploration program are depicted on the graphic soil profiles (boring logs) on Figures 2 and 3 of this report. Soil stratification is based on examination of recovered soil samples and interpretation of field boring logs. The stratification lines represent the approximate boundaries between the soil types, while the actual transitions may be gradual.

A generalization of the subsurface soil conditions encountered in the borings is described below:

| DEPTH (feet) | | |
|---|---|---|
| **From** | **To** | **SOIL DESCRIPTION** |
| 0 | 6 | Medium dense fine sand |
| 6 | 20 | Medium dense clayey fine sand and silty fine sand, sometimes very dense in the lower part |
| 20 | 35 | Stratified layers of medium dense clayey fine sand, stiff to very stiff sandy clay to clay with sand, and medium dense to dense silty fine sand |
| 35 | 70 | Medium dense to very dense clayey fine and silty fine sand, with some layers of hard cemented sandy silt and hard sandy clay |
| 70 | 90 | Hard cemented sandy silt, calcareous silt and limestone, with some very dense silty to clayey fine sand |

## GROUNDWATER LEVEL

At the time of our field exploration program, the measured water level in the boreholes ranged from approximately 7 to 7½ feet below the existing ground surface. These water level readings may differ from the actual groundwater table due to variations in the permeability of soil layers. The degree of accuracy of the reported water levels is also related to the time allowed for the borehole water level to reach equilibrium. In addition, the groundwater level will fluctuate over time, due to variations is seasonal rainfall and other factors.

Water levels could not be reliably measured at boring Nos. 1, 2 and 5, since the silty/clayey soils prevented stabilization of the water level prior to introducing drilling fluid (bentonite) needed to be introduced in order to stabilize the borehole walls. This does not mean that groundwater does not occur within the depth of these borings. Considering the site conditions and generally flat topography, groundwater at boring Nos. 1, 2 and 5 likely occurs at a depth similar to that encountered at Boring Nos. 3 and 4, but would need to be verified by piezometer installation if desired.

## ENGINEERING EVALUATION AND RECOMMENDATIONS

The proposed equalization tanks and the electrical/storage building are structures with very different loading scenarios, so will be discussed separately, as follows.

### Equalization Tanks

The soils encountered in the surficial 17 to 22 feet are primarily granular in nature, consisting mostly of fine sand, clayey fine sand and silty fine sand. These soils are in a generally medium dense state, although sometimes were encountered in a very dense state at a depth of approximately 20 feet.

These sands are underlain by an approximately 15 feet thick, very stratified sequence that primarily consists of interbedded layers of medium dense clayey fine sand, stiff to very stiff sandy clay and medium dense to dense silty fine sand. These extend to a depth of approximately 30 to 35 feet. Refusal while pushing the Shelby tubes and close examination of the recovered samples indicated that layers of cemented silt (soft rock) also occur in this sequence. Three of the four

Shelby tubes (all except S-1) encountered refusal and two (S-2 and S-3) could only be pushed 3 to 6 inches.

Underlying this stratified sequence are primarily medium dense to very dense clayey fine sand and silty fine sand to a depth of approximately 70 feet, which are underlain primarily by hard cemented/calcareous silt, limestone and very dense silty to clayey fine sands.

The most cost effective foundation for supporting a tank structure is often a shallow mat foundation. This typically consists of a reinforced concrete slab upon which the tanks rest. Tank structures of this size impose relatively large loads over a large area, resulting in significant stress increases at greater depth than would occur beneath small foundation areas. These stress increases result in settlement within the subsurface soils, such that settlement is often the limiting factor in deciding if a mat foundation is practical.

Settlement Analysis for Mat Foundation

Settlement analyses was performed using the "Settle3D" software (by Rocscience, Inc.), which can model the subsurface conditions and stress distributions as a three-dimensional model. Published correlations relying on the SPT "N-values" and laboratory test results were used to estimate elastic moduli of the soils and the results of the consolidation test was used to determine consolidation characteristics of the "sandy clay" to "clay with sand". The Westergaard stress distribution method was used for calculating the stress changes caused by the structural loading, with a maximum allowable soil pressure of 1,825 pounds per square foot.

With the soils prepared as recommended in the site preparation section, total settlement of the proposed tanks is estimated to be in the range of approximately 1 to 1½ inches at the center of the tanks, and approximately ½ to 1 inch at the tank perimeters. The maximum differential settlement at each tank is anticipated to be in the range of approximately 1/2 to 1 inch, with greater settlement near the center and less settlement at the tank perimeter.

Approximately 70 percent the settlement will occur as elastic, or short-term, settlement that will occur shortly after or during initial loading (filling) of the tanks. This immediate settlement is due to compression of the predominately sandy (fine sand, silty fine sand and similar) soils that occur primarily within the upper 20 feet of the soil profile, and below a depth of approximately 35 feet.

The remainder of the settlement is expected to occur as plastic, long-term settlement, due to slow consolidation settlement within the stiff sandy clay strata.

If the maximum settlements described above are not acceptable, methods to decrease post-construction settlement can be considered.

Methods to Decrease Post-Construction Settlements

These methods would include either modifying the existing soils by improvement or using a deep foundation system, such as piles.

The soils in the proposed tank areas could be preloaded (modified) by surcharging, in order to pre-consolidate soils prior to construction of the tanks. A surcharging program consists of mounding soil over the proposed tank area to simulate the foundation loads. As the surcharge load is applied, settlement occurs and is monitored. Once settlements have subsided to acceptable levels, the surcharge load is removed and shallow foundations can then be constructed. Assuming a surcharge load of moderately compacted soil is used, a surcharge height of at least 17 feet would likely be required over the entire foundation area.

Another surcharge alternative would be to construct the tanks and, prior to making final pipe connections to the tanks, simultaneously fill the three tanks with water to surcharge the soils. In this alternative, the tanks may still need to withstand the total settlements presented above, but most of the total settlement could be induced before making final pipe connections.

It should be noted that an additional, long-term settlement of ¼ to ½ inch may occur after either of the above surcharging programs, depending on the length of the surcharge program.

During a surcharging program, settlement needs to be monitored by use if either settlement plates embedded in the soil beneath the surcharge fill, or by monitoring points on the mat foundation if the tank filling procedure is used. Surcharging should continue until the settlement (elevation) readings indicate that initial settlements have stabilized. We estimate that this should occur within one to two weeks after the full surcharge has been placed.

Deep soil improvement by the vibro-replacement (stone column) methodology was also considered, however, the soil conditions at the site are not well suited to significant improvement by this method, since most of the sand soils are already medium dense to dense (so would not likely be significantly densified further), and this methodology is relatively ineffective on clays. This method is, therefore, not recommended.

The most cost effective type of deep foundation would most likely be either driven prestressed concrete piles or augered cast-in-place piles. Due to potential difficulties in driving piles through the sometimes dense sands and the stiff sandy clay strata, augered cast-in-place piles would be preferable. Additional recommendations for auger-cast piles are presented below.

**Augered Cast-In-Place Piles for Equalization Tanks**

For comparison, a review of our records indicates that the nearby influent (headworks) structure was at least partially supported upon 18-inch diameter, augered cast-in-place piles installed to a depth of 40 feet and having a design allowable compressive capacity of 35 tons.

For the proposed equalization tanks, we have performed axial (compressive and tensile) pile capacity analyses using the FB-Deep (v. 2.04) computer program. Lateral capacity analyses were performed using the LPile (v. 2013) computer program. Soil parameters used in the analyses were based upon the soil profiles encountered and the laboratory test results. Based upon our analyses, we recommend one of the following options be utilized:

| Pile Size & Type | Pile Toe Embedment (feet below existing ground surface) | Allowable Compressive Capacity (tons) | Allowable Tensile Capacity (tons) | Allowable Lateral Capacity (tons) |
|---|---|---|---|---|
| 16" dia. ACIP | 40 | 38 | 22 | 2½ |
| 18" dia. ACIP | 37 | 40 | 22 | 3 |
| 18" dia. ACIP | 45 | 55 | 31 | 3 |
| 24" dia. ACIP | 50 | 90 | 49 | 5½ |

ACIP = augered cast-in-place concrete

The above axial capacities assume a minimum pile spacing (center to center) of at least three (3x) pile diameters. For laterally loaded piles, however, the pile spacing should be at least eight (8x) pile diameters, in order to have negligible group effects. If the pile spacings are closer than the above, we should review our findings to see if a group reduction factor for each pile capacity is necessary.

In addition, if the selected pile option has a design allowable compressive capacity greater than 40 tons, we recommend that a pile load test be performed on at least one pile. The load test should be performed to twice (2x) the pile design capacity, to confirm that the pile capacities are achieved. The load test should be a static load test performed according to current Florida Building Code requirements and ASTM procedures.

The estimated allowable lateral load capacity was calculated based upon the following assumptions: (1) the top of the pile, or bottom of pile cap, is at a depth of 2 to 3 feet below the existing ground surface, (2) the lateral load is applied at the top of the pile, (3) a pinned pile top condition, a.k.a. free head condition, (3) storm scour is not a factor, (4) the allowable lateral load is one-half of the load that would result in a lateral deflection of 1.0 inch at the top of the pile, and (5) the structural reinforcing steel is approximately one percent of the pile cross section area. A pinned pile top condition means that the top of the pile is free to rotate under an applied lateral load or moment. Lateral deflections would be less than the above values if a fixed pile top condition were used, which is the condition where the structural connection of the pile to the pile cap and structure are such that the pile top does not rotate (remains vertical) under an applied lateral load or moment. If conditions vary from our assumptions, we should review this to determine if a revised lateral capacity is necessary.

Should the design professionals require a pile top elevation or pile toe embedment depth different from the above, we must be given the opportunity to review their requirements, since they may impact our recommendations. Should the design professionals require a pile cap bottom (top of pile) more than 3 feet below the existing ground surface, the pile capacities may be reduced since there will be less side area to develop frictional resistance. In this case, we must be given an opportunity to review the situation and estimate new pile capacities based on the reduced pile lengths.

<u>Augered Cast-In-Place Concrete Piles</u>

The successful auger cast pile installation will depend upon the expertise of the contractor and the techniques used. While the installation of piles can be monitored to determine that the piles are installed in general accordance with specifications, it is not possible to make an absolute determination of actual pile capacity based upon installation activities as with driven piles.

A representative of Ardaman & Associates, Inc. should be present during pile installation to provide the necessary engineering documentation. Documentation would include information relative to pile penetration, condition of hole prior to concrete placement, the amount of concrete injected and the type of reinforcement used. Concrete quality control is also essential and should include field slump tests and compressive strength determinations.

We have included a sample auger injected concrete pile specification as Appendix II of this report. This specification is made as a guide to the design professionals and we recommend that part of it be incorporated into the project specifications.

In order to penetrate the overlying clays, it will be necessary for the auger pile contractor to provide equipment with sufficient torque and dead weight to penetrate the soils to the required minimum depth requirement.

**Electrical/Storage Building**

Based on the results of our exploration and our engineering analyses, the soils encountered at the subject site are capable of supporting the proposed electrical/storage building on conventionally designed shallow foundation systems, if the soils are properly prepared.

We estimate that a total settlement of less than one inch will occur, with an estimated differential settlement of less than one-half inch. Most of the settlement should occur concurrent with application of the structural loads.

This soil evaluation assumes that the soils are prepared in accordance with the soil preparation recommendations of this report, that foundation loads are no greater than those indicated previously and that our foundation design recommendations are followed. The recommended site preparation program involves densification of the subgrade foundation surfaces to compress loose surficial soils, as well as subgrade soils disturbed by other site preparation procedures, thereby creating a more uniform and less yielding soil mass. The created conditions will promote a more uniform settlement of the structure, thereby reducing the incidence and magnitude of differential settlement.

## Foundation Design

Foundations for the proposed structure may be designed for an allowable soil contact pressure of 2,500 pounds per square foot (psf). We recommend that all wall foundations be no less than 18 inches wide and column foundations be no less than 24 inches wide. All foundations should be designed for an equal dead load distribution in accordance with building code requirements.

All footings should be embedded so that the bottom of the foundation is a minimum of 18 inches below adjacent compacted ground surface grades on all sides. This minimum embedment is desired to provide adequate confinement of the bearing soils, and to achieve the recommended bearing pressure. In addition, all footings should be constructed in a "dry" fashion. We recommend that the building grades be selected so that normal seasonal high groundwater levels remain at least one foot below footings.

If the building design includes perimeter stem walls that retain soil fill, these should be designed assuming that they act as a cantilevered retaining wall without a fixed top condition (i.e. the design should disregard the floor slab connection at the top). This will allow the stem wall to better resist lateral earth pressures during construction without significant lateral or rotational movement.

## Floor Slab Recommendations

The floor slab may be safely supported as a slab-on-grade provided that the site preparation recommendations are followed. We recommend that all ground floor slabs be "floating", meaning that they are 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. If an integral footing-slab construction is planned for this building, we recommend additional reinforcing steel in this area to tie the footings and slab together, and to reduce the potential for cracking caused by differential movement.

We also recommend that in areas where floor finishes will be used, the floor slab bearing soils should be covered by a lapped polyethylene sheeting in order to reduce the potential for floor dampness, which can affect the performance of glued tile, carpet and other flooring. This membrane should consist of a minimum 6-mil single layer of non-corroding, non-deteriorating sheeting material placed to minimize seams and to cover all of the soil below the building floor. This membrane should be cut in cross shape for pipes or other penetrations and the membrane should extend to within one-half inch of all pipes or other penetrations. All seams of the membrane should be lapped at least 12 inches. Punctures or tears in the membrane should be repaired with the same or compatible material.

The performance of concrete floor slabs is also affected by the concrete mix that is used. A relatively high water-cement ratio can cause aesthetic disruptions, such as slab curling and shrinkage cracking. Also, an additional waiting period may be required prior to installing moisture sensitive floor covering because of moisture loss from the concrete floor slab. For these reasons, we recommend a concrete mix design be selected with a water-cement ratio not exceeding 0.45. In addition, we recommend water curing for the first 3 days to minimize floor cracking and curling.

**Soil Preparation Recommendations for Electrical/Storage Building and Equalization Tanks**

The existing surficial soils should be prepared, prior to placement of structural fill and foundation construction on the soils, in accordance with the following site preparation recommendations. The recommended procedures should be covered in the project specifications, and completed prior to construction of the foundation system.

1. The structural areas, plus a margin of at least 5 feet outside building perimeter lines, should be cleared (stripped) of all surface vegetation and organic debris. After stripping, this area should be grubbed or root-raked to completely remove roots with a diameter greater than ½ inch, stumps, or smaller roots in a concentrated state. The actual depths of stripping and grubbing must be determined by visual observation and judgment during the earthwork operation. All existing slabs, abandoned utilities and underground structures should either be removed or filled with cement grout to reduce the possibility of soil erosion into the voids.

2. Following the clearing operations, the exposed subgrade should be evaluated and roof-rolled to confirm that all unsuitable materials have been removed. The proof-rolling should consist of compaction with equipment capable of providing the densities required below. Careful observations should be made during proof-rolling to help identify any areas of soft yielding soils that may require over-excavation and

replacement. Care should be used when operating vibratory compactors near the existing structures (within 75 feet) to avoid transmission of vibrations that could cause settlement damage or disturb occupants. Areas close to existing structures should be compacted using static (non-vibratory) compaction methods.

3. After proof-rolling and remediation of any yielding areas noted, the structure areas (plus the 5 feet margin) should be compacted with at least 6 passes with a vibratory roller; a loaded, rubber-tired, front-end loader; or other equipment capable of achieving the compaction requirements. Each pass should overlap the preceding pass by at least 30 percent (%) and some of the passes should be made in a perpendicular direction. Sufficient passes should be made over the structure areas, plus the 5 feet margin, to produce a density of at least 95% of Modified Proctor (ASTM D-1557) maximum density to a depth of 1.0 foot below the compacted surface.

4. After compaction and testing to verify that the desired compaction has been achieved at this elevation, fill consisting of clean fine sands containing no more than 12% passing the No. 200 sieve, and having a Unified Soil Classification (ASTM D-2487) of "SP" or "SP-SM," can be placed in level lifts not exceeding 12 inches loose thickness and compacted with the equipment described above. Each lift should be compacted to at least 95% of Modified Proctor maximum density prior to the placement of subsequent lifts and density tests to confirm compaction should be performed in each fill lift before the next lift is placed. We note that soils with more than 12% passing the No. 200 sieve can be used as fill in some applications, but will be more difficult to moisture condition and compact due to their inherent nature to retain moisture.

5. After excavation for the foundations, the foundation contact soils should be compacted to a minimum of 95% of Modified Proctor maximum density, using suitable mechanical equipment to achieve the specified level of density to the required depth. Foundation bottom grade should be tested to confirm that a minimum density of 95 percent of the Modified Proctor maximum dry density (ASTM D-1557) exists to a depth of 12 inches below footing bottom. If necessary, the bottom of the footing excavation shall be over-excavated, refilled, and recompacted with mechanical equipment to achieve the necessary minimum field density to the required depth.

6. Fill necessary to raise the grade from the top of the foundation elevation to finished floor slab subgrade elevation should also consist of clean fine sands meeting the requirements of item No. 4, above, and compacted to at least 95% of Modified Proctor maximum density. If fill is placed inside partially completed walls, extreme care should be exercised to avoid damage to these walls.

7. A geotechnical engineer or his representative from Ardaman & Associates, Inc., Sarasota office, should inspect and test the compacted excavated elevations and each layer of fill to verify compliance with the above recommendations. In addition, a representative should inspect and test the foundation contact soils immediately prior to concrete placement.

During the compaction process, soil moisture contents may need to be controlled in order to facilitate proper compaction. If additional moisture is necessary to achieve compaction objectives of imported structural fill, then water should be applied in such a way that it will not cause erosion or removal of the subgrade soils. In the event that applied water does not penetrate sufficiently deep into natural soils to act as a lubricant in the compaction process, it will be necessary to disk or otherwise break up the soils before and during application of water. A moisture content within two percentage points of the optimum indicated by the modified Proctor test (ASTM D-1557) is recommended prior to compaction of the natural ground and structural fill.

**Dewatering**

If the control of groundwater is required to achieve the necessary stripping, excavation, proof-rolling, filling, compaction, and any other earthwork, sitework, or foundation subgrade preparation operations required for the project, the actual method(s) of dewatering should be determined by the contractor. Dewatering should be performed to lower the groundwater level to depths that are adequately below excavations and compaction surfaces. Adequate groundwater level depths below excavations and compaction surfaces vary depending on soil type and construction method, and are usually two feet or more. Dewatering solely with sump pumps may not achieve the desired results.

## QUALITY ASSURANCE

We recommend establishing a comprehensive quality assurance program to verify that all site preparation and foundation construction is conducted in accordance with the appropriate plans and specifications. Since Ardaman & Associates has performed and interpreted the results of a geotechnical exploration for the site and has prepared earthwork and foundation design recommendations based upon this interpretation, Ardaman is best suited to provide quality assurance testing and inspection services to assure that the intent of our recommendations have been implemented during construction.

As a minimum, an on-site engineering technician should monitor the installation of all foundation piles, should monitor all stripping and grubbing to verify that all deleterious materials have been removed, and should observe the proof-rolling operation to verify that the appropriate number of passes are applied to the subgrade. In-situ density tests should be conducted during filling activities and below all footings and floor slabs to verify that the required densities have been achieved. In-situ density values should be compared to laboratory Proctor moisture-density results for each of the different natural and fill soils encountered.

We also recommend inspecting and testing the construction materials for the foundations and other structural components.

**In-Place Density Testing Frequency**

In this region, earthwork testing is typically performed on an on-call basis when the contractor has completed a portion of the work. The test result from a specific location is only representative of a larger area if the contractor has used consistent means and methods and the soils are practically uniform throughout. The frequency of testing can be increased and full-time construction inspection can be provided to account for variations. We recommend that the following minimum testing frequencies be utilized.

| Structure Test Location | Percent Compaction (ASTM D1557) | Depth (inches) | Recommended Minimum Test Frequency |
|---|---|---|---|
| Bottom of Footings | 95 | 12 | At column footings and every 75 l.f. of wall footing |
| Slab Subgrade | 95 | 12 | per 2,500 sq.ft. of structural area |
| Structural Fill | 95 | full depth | per 2,500 sq.ft. of structural area per lift |

If the plans and specifications for the project are more stringent than the requirements listed above, the requirements of the plans and specifications should be followed.

Representative samples of the various natural ground and fill soils should be obtained and transported to our laboratory for Proctor compaction tests. These tests will determine the maximum dry density and optimum moisture content for the materials tested and will be used in conjunction with the results of the in-place density tests to determine the degree of compaction achieved.

## GENERAL COMMENTS

The analysis and recommendations submitted in this report are based upon the data obtained from five (5) test borings performed at the locations indicated on the attached Figure 1. This report does not reflect any variations which may occur outside of or between the boring locations.

*Kimley-Horn & Associates, Inc.*
*File No. 15-7337*
*February 25, 2016 (revised May 20, 2016)*

While the borings are representative of the subsurface conditions at their respective locations and within their respective vertical reaches, local variations characteristic of the subsurface materials of the region are anticipated and may be encountered. The nature and extent of variations may not become evident until during the course of a ground improvement program, if such a program is undertaken. If variations then appear evident, it will be necessary to reevaluate the recommendations of this report, after performing on-site observations during the construction period and noting the characteristics of any variations. The boring logs and related information are based upon the driller's logs and visual examination of selected samples in the laboratory. The delineation between soil types shown on the logs is approximate, and the description represents our interpretation of the subsurface conditions at the designated boring location on the particular date drilled.

We note that additional explorations and analyses may be necessary prior to finalization of design recommendations for the equalization tanks.

The groundwater table depths shown on the boring logs represent the groundwater surfaces encountered on the dates shown. Fluctuation of the groundwater table should be anticipated throughout the year.

It has been a pleasure to be of assistance to you with this project. Please contact us when we may be of further service to you, or should you have any questions concerning this report.

Very truly yours,

ARDAMAN & ASSOCIATES, INC.
*Certificate of Authorization No. 5950*

JERRY H. KUEHN LICENSE No. 35557 STATE OF FLORIDA PROFESSIONAL ENGINEER

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JHK/GSS:ly

# APPENDIX I

## SOIL BORING, SAMPLING AND TESTING METHODS

### Standard Penetration Test

The Standard Penetration Test (SPT) is a widely accepted method of in situ testing of foundation soils (ASTM D-1586). A 2-foot long, 2-inch O.D. split-barrel sampler attached to the end of a string of drilling rods is driven 18 inches into the ground by successive blows of a 140-pound hammer freely dropping 30 inches. The number of blows needed for each 6 inches of penetration is recorded. The sum of the blows required for penetration of the second and third 6-inch increments of penetration constitutes the test result or N-value. After the test, the sampler is extracted from the ground and opened to allow visual examination and classification of the retained soil sample. The N-value has been empirically correlated with various soil properties allowing a conservative estimate of the behavior of soils under load. The following tables relate N-values to a qualitative description of soil density and, for cohesive soils, an approximate unconfined compressive strength (Qu):

| Cohesionless Soils: | N-Value | Description |
|---|---|---|
| | 0 to 4 | Very loose |
| | 4 to 10 | Loose |
| | 10 to 30 | Medium dense |
| | 30 to 50 | Dense |
| | Above 50 | Very dense |

| Cohesive Soils: | N-Value | Description | Qu (ton/ft$^2$) |
|---|---|---|---|
| | 0 to 2 | Very soft | Below 0.25 |
| | 2 to 4 | Soft | 0.25 to 0.50 |
| | 4 to 8 | Medium stiff | 0.50 to 1.0 |
| | 8 to 15 | Stiff | 1.0 to 2.0 |
| | 15 to 30 | Very stiff | 2.0 to 4.0 |
| | Above 30 | Hard | Above 4.0 |

The tests are usually performed at 5-foot intervals. However, more frequent or continuous testing is done by our firm through depths where a more accurate definition of the soils is required. The test holes are advanced to the test elevations by rotary drilling with a cutting bit, using circulating fluid to remove the cuttings and hold the fine grains in suspension. The circulating fluid, which is a bentonitic drilling mud, is also used to keep the hole open below the water table by maintaining an excess hydrostatic pressure inside the hole. In some soil deposits, particularly highly pervious ones, NX-size flush-coupled casing must be driven to just above the testing depth to keep the hole open and/or prevent the loss of circulating fluid.

Representative split-spoon samples from each sampling interval and from every different stratum are brought to our laboratory in air-tight jars for further evaluation and testing, if necessary. After thorough examination and testing of the samples, the samples are discarded unless prior arrangements have been made. After completion of a test boring, the hole is kept open until a steady state groundwater level is recorded. The hole is then sealed, if necessary, and backfilled.

A hammer with an automatic drop release (auto-hammer) is sometimes used. In this case, a correction factor is applied to the raw blow counts, since the energy efficiency of the auto-hammer is greater than that of the safety hammer. The auto-hammer blow counts are corrected to equivalent safety hammer “N” values, based upon calibration of the auto-hammer (per ASTM D4633) and standard practice.

## Hand Auger Borings

Hand auger borings are used, if soil conditions are favorable, when the soil strata are to be determined within a shallow (approximately 5 to 9 feet) depth or when access is not available to power drilling equipment. A 3-inch diameter, hand bucket auger with a cutting head is simultaneously turned and pressed into the ground. The bucket auger is retrieved to the surface at approximately 6-inch intervals and its contents emptied for inspection. The soil sample so obtained is classified and representative samples put in bags or jars and transported to the laboratory for further classification and testing.

## Undisturbed Sampling

Undisturbed sampling implies the recovery of soil samples in a state as close to their natural condition as possible. Complete preservation of *in situ* conditions cannot be realized; however, with careful handling and proper sampling techniques, disturbance during sampling can be minimized for most geotechnical engineering purposes. Examination and testing of undisturbed samples gives a more accurate estimate of in situ soil behavior than is possible with disturbed samples.

Normally, we obtain undisturbed samples by pushing a 2.875-inch I.D., thin wall seamless steel tube, 24 inches into the soil with a single stroke of a hydraulic ram. The sampler, which is a Shelby tube, is 30 inches long. After the sampler is retrieved, the ends are sealed in the field and it is transported to our laboratory for further examination and testing, as needed.

In some instances, when even less disturbed samples are required, a fixed-piston sampling device is used. The fixed-piston sampler is a 2.875-inch I.D. Shelby tube with a piston inside it. When the sampler is lowered into the bore hole, the piston is located at the lower end of the sampling tube. The piston is then placed at the bottom of the hole on top of the soil to be sampled, and is held stationary while the tube is smoothly pushed past the piston 24 inches into the soil. The sample is sheared from the parent soil by rotating the sampling device. After the sampler is brought out of the hole, the ends of the tube are sealed and the sample is brought back to our laboratory.

Four major improvements over our conventional undisturbed sampling procedures are achieved with the piston sampler; a larger sample is obtained; no soil enters the tube as the sampler is lowered to the sampling depth; excess soil does not enter the tube during the sampling operation; and a vacuum is generated between the piston and the sample as the sampler is being retrieved, thus helping to retain the sample in the tube.

## Laboratory Test Methods

Soil samples returned to our laboratory are examined by a geotechnical engineer or geotechnician to obtain more accurate descriptions of the soil strata. Laboratory testing is performed on selected samples as deemed necessary to aid in soil classification and to further define engineering properties of the soils. The test results are presented on the soil boring logs at the depths at which the respective sample was recovered, except that grain size distributions or selected other test results may be presented on separate tables, figures or plates as described in this report. The soil descriptions shown on the logs are based upon a visual-manual classification procedure in general accordance with the Unified Soil Classification System (ASTM D-2488-84) and standard practice. Following is a list of abbreviations which may be used on the boring logs or elsewhere in this report.

| | |
|---|---|
| -200 | - Fines Content (percent passing the No. 200 sieve); ASTM D1140 |
| DD | - Dry Density of Undisturbed Sample; ASTM D2937 |
| Gs | - Specific Gravity of Soil; ASTM D854 |
| k | - Hydraulic Conductivity (Coefficient of Permeability) |
| LL | - Liquid Limit; ASTM D423 |
| OC | - Organic Content; ASTM D2974 |
| pH | - pH of Soil; ASTM D2976 |
| PI | - Plasticity Index (LL-PL); ASTM D424 |
| PL | - Plastic Limit; ASTM D424 |
| Qp | - Unconfined Compressive Strength by Pocket Penetrometer; |
| Qu | - Unconfined Compressive Strength; ASTM D2166 (soil), D7012 (rock) |
| SL | - Shrinkage Limit; ASTM D427 |
| ST | - Splitting Tensile Strength; ASTM D3967 (rock) |
| USCS | - Unified Soil Classification System; ASTM D2487, D2488 |
| w | - Water (Moisture) Content; ASTM D2216 |

**Soil Classifications**

The soil descriptions presented on the soil boring logs are based upon the Unified Soil Classification System (USCS), which is the generally accepted method (ASTM D-2487 and D-2488) for classifying soils for engineering purposes. The following modifiers are the most commonly used in the descriptions.

| | Modifier | Fines, Sand or Gravel Content* |
|---|---|---|
| For Sands: | with silt or with clay | 5% to 12% fines |
| | silty or clayey | 12% to 50% fines |
| | with gravel or with shell | 15% to 50% gravel or shell |

| | Modifier | Fines, Sand or Gravel Content* |
|---|---|---|
| For Silts or Clays: | with sand | 15% to 30% sand and gravel; and % sand > % gravel |
| | sandy | 30% to 50% sand and gravel; and % sand > % gravel |
| | with gravel | 15% to 30% sand and gravel; and % sand < % gravel |
| | gravelly | 30% to 50% sand and gravel; and % sand < % gravel |

* may be determined by laboratory testing or estimated by visual/manual procedures. Fines content is the combined silt and clay content, or the percent passing the No. 200 sieve.

The USCS also uses a set of Group Symbols, which may also be listed on the soil boring logs. The following is a summary of these.

| Group Symbol | General Group Name* | Group Symbol | General Group Name* |
|---|---|---|---|
| GW | Well-graded gravel | SW | Well-graded sand |
| GP | Poorly graded gravel | SP | Poorly graded sand |
| GW-GM | Well-graded gravel with silt | SW-SM | Well-graded sand with silt |
| GW-GC | Well-graded gravel with clay | SW-SC | Well-graded sand with clay |
| GP-GM | Poorly graded gravel with silt | SP-SM | Poorly graded sand with silt |
| GP-GC | Poorly graded gravel with clay | SP-SC | Poorly graded sand with clay |
| GM | Silty gravel | SM | Silty sand |
| GC | Clayey gravel | SC | Clayey sand |
| GC-GM | Silty, clayey gravel | SC-SM | Silty, clayey sand |
| CL | Lean clay | ML | Silt |
| CL-ML | Silty clay | MH | Elastic silt |
| CH | Fat clay | OL or OH | Organic silt or organic clay |

* Group names may also include other modifiers, per standard or local practice.

Other soil classification standards may be used, depending on the project requirements. The AASHTO classification system is commonly used for highway design purposes and the USDA soil textural classifications are commonly used for septic (on-site sewage disposal) system design purposes.

# APPENDIX II

## Laboratory Test Results for Shelby Tube Samples

## Table 1
## Summary of Laboratory Tests Results for Shelby Tube Samples

| Sample No. | Recovery (inches) | Average Unit Weight (pcf) | Test Specimen Depth (feet) | Test Specimen Description (Unified Soil Class.) | -200 (%) | w (%) | Qp (tsf) |
|---|---|---|---|---|---|---|---|
| S-1 | 24 | 109.1 | 25.6 | Gray clayey fine sand (SC) | 22 | 31 | --- |
| | | | 26.0 | Olive gray sandy clay (CL-CH) | --- | 46 | 1.25 |
| | | | 26.4 | Olive gray sandy clay (CL-CH) * | --- | 60 | 1.5 |
| | | | 26.6 | Olive gray sandy clay (CL-CH) | 68 | 62 | 1.75 |
| | | | 26.9 | Olive gray sandy clay (CL-CH) | --- | 54 | 1.75 |
| | | | 27.0 | Gray clayey fine sand with phosphate (SC) | --- | 9.8 | --- |
| | | | 27.1 | Gray clayey fine sand with phosphate (SC) | --- | 29 | --- |
| | | | 27.4 | Gray clayey fine sand with phosphate (SC) | 15 | 28 | --- |
| S-2 | 6 | --- | 30 - 30.5 | Gray clayey fine sand (SC) | 18 | --- | --- |
| S-3 | 3 | --- | 33 - 33.25 | Gray clayey fine sand (SC) | 13 - 14 | --- | --- |
| S-4 | 15 | 116.8 | 35.1 | Gray clayey fine sand with phosphate (SC) | --- | 32 | --- |
| | | | 35.2 | Gray clayey fine sand with phosphate (SC) | 19 | 30 | --- |
| | | | 35.7 | Gray clayey fine sand with phosphate (SC) | --- | 31 | --- |
| | | | 36.1 | Gray sand with phosphate (SP) | --- | 30 | --- |
| | | | 36.3 | Gray sand with phosphate (SP) | 3.8 | 14 | --- |
| * Consolidation test specimen, see attached test report | | | | | | | |

-200 = Percent passing U.S. Standard No. 200 sieve

w = Water content

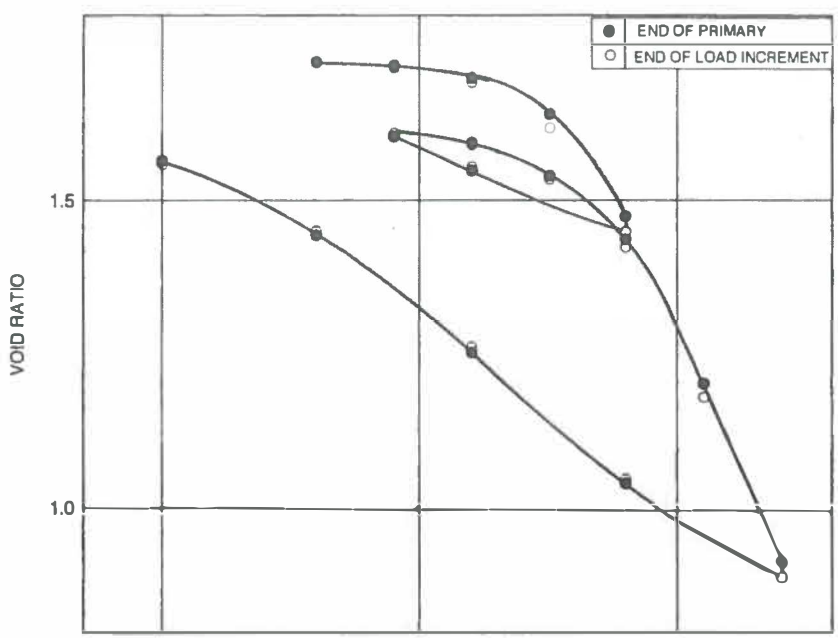
Qp = Unconfined compressive strenght by pocket penetrometer

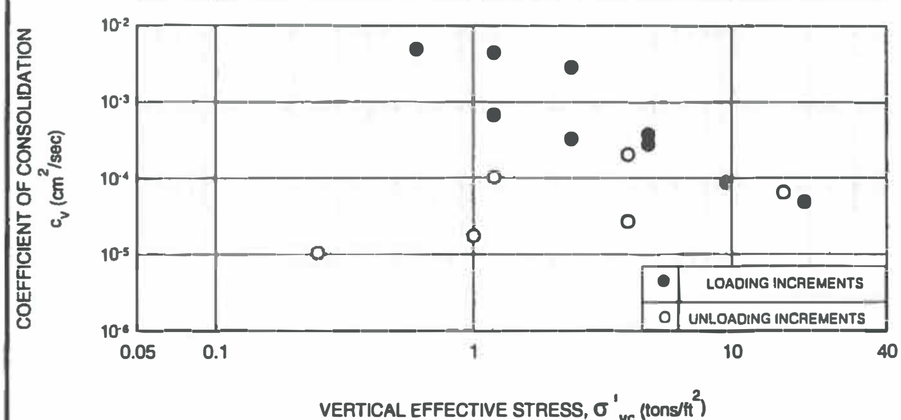
# ARDAMAN & ASSOCIATES, INC. GEOTECHNICAL TESTING LABORATORY
# ONE-DIMENSIONAL INCREMENTAL LOADING CONSOLIDATION TEST REPORT

CLIENT:
PROJECT: North County WRF
FILE NO.: 15-36-7337

DATE SAMPLE RECEIVED: 4/5/2016
DATE SAMPLE SET-UP: 4/6/2016
DATE REPORTED: 5/3/2016

INCOMING SAMPLE NO.: -----
BORING: 1 SAMPLE: S-1
DEPTH: 25.0 - 27.0 ☑ ft; ☐ m
LAB IDENTIFICATION NO.: 157337/B1S1
SAMPLE DESCRIPTION.: Gray-brown clay with trace fine sand seams (friable)




### Test Methods & Procedures

ASTM Standard D2435
☐ Method A
☑ Method B
$C_v$ Interpretation Method
☑ $c_v$ [Log Time]
☐ $c_v$ [Sq. Root Time]
Trimming Method
☑ cutting shoe
☐ other________
Initial Sample Diameter 7.30 (cm)

### Test Conditions

☐ Tested at Natural Moisture Content
☑ Specimen Tested Inundated
Inundated at $\sigma'_{vc}$ 0.05 (tsf)
Inundation Fluid:
☑ tap water
☐ other

### Specimen Conditions

| Parameter | Initial | Final |
|---|---|---|
| D (cm) | 5.00 | 5.00 |
| H (cm) | 1.905 | 1.786 |
| $w_c$ (%) | 47.0 | 44.6 |
| $\gamma_d$ (pcf) | 62.5 | 66.6 |
| e | 1.729 | 1.558 |
| S (%) | 74 | 78 |

$G_s$: 2.73 ☐ Assumed ☑ Measured

### Index Properties

| | |
|---|---|
| LL | --- |
| PL | --- |
| PI | --- |

| Particle-Size Analysis | U.S. Standard Sieve Size | Gravel | | | Coarse Sand | Medium Sand | | Fine Sand | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| ☐ ASTM D422<br>☑ ASTM D1140-Method | | 3/4" | 3/8" | No. 4 | No. 10 | No. 20 | No. 40 | No. 60 | No. 100 | No. 140 | No. 200 |
| Dry Mass(g): 37.42 | Soil Passing (%, dry mass basis) | ----- | ----- | ----- | ----- | ----- | ----- | ----- | ----- | ----- | 98.6 |

The test data and all associated project information presented hereon shall be held in confidence and disclosed to other parties only with the authorization of the Client. Physical and electronic records of each project are kept for a minimum of 7 years. Test samples are kept in storage for at least 10 working days after mailing of the test report, prior to being discarded, unless a longer storage period is requested in writing and accepted by Ardaman & Associates, Inc.

Where: H=Specimen height; D = Specimen diameter; $w_c$ = Water content (ASTM D2216); $\gamma_d$ = Dry density; e = Void ratio; S = Saturation; $G_s$ = Specific gravity; $c_v$ = Coefficient of consolidation; and $C_{\alpha e}$ = Secondary compression index.

Checked By: M Date: 05/03/16

# ARDAMAN & ASSOCIATES, INC. GEOTECHNICAL TESTING LABORATORY
# ONE-DIMENSIONAL INCREMENTAL LOADING CONSOLIDATION TEST REPORT

CLIENT: ____________________

PROJECT: North County WRF

FILE NO.: 15-36-7337

INCOMING SAMPLE NO.: -----

BORING: 1 SAMPLE: S-1

DEPTH: 25.0 - 27.0 ☑ ft; ☐ m

LAB IDENTIFICATION NO.: ____________________

SAMPLE DESCRIPTION.: Gray-brown clay with trace fine sand seams (friable)

DATE SAMPLE RECEIVED: 4/5/2016

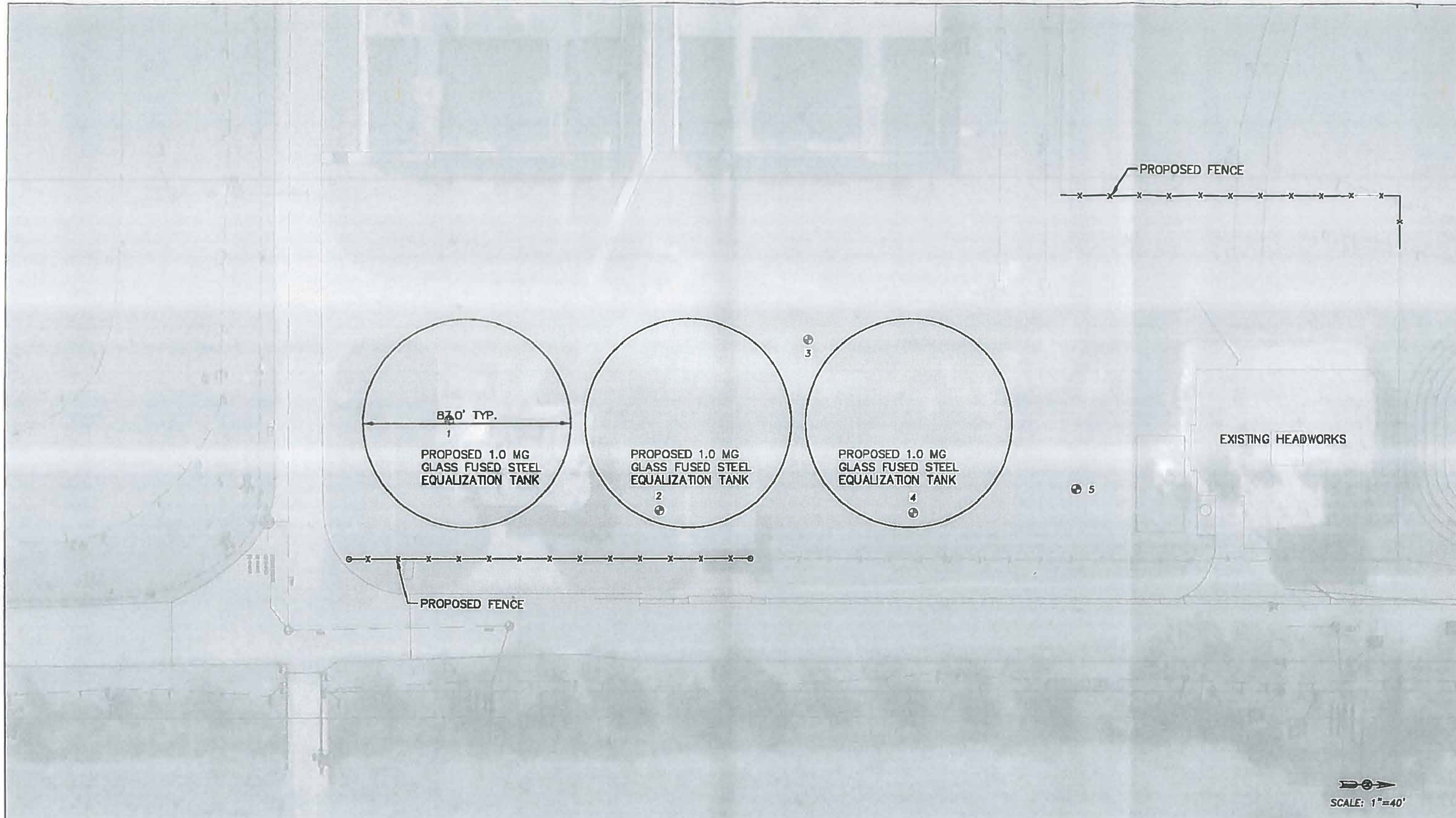
DATE SAMPLE SET-UP: 4/6/2016

DATE REPORTED: 5/3/2016

| Effective Vertical Stress $\sigma'_{vc}$ (tons/ft$^2$) | Void Ratio | | Vertical Strain (%) | | Coefficient of Consolidation (cm$^2$/sec) ☑ $c_v$ [Log Time] ☐ $c_v$ [Sq. Root Time] | Secondary Compression Index, $C_{\alpha e}$ |
|---|---|---|---|---|---|---|
| | End of Primary Consolidation | End of Increment | End of Primary Consolidation | End of Increment | | |
| 0.0 | 1.729 | 1.729 | 0.00 | 0.00 | --- | ----- |
| 0.40 | 1.723 | 1.723 | 0.20 | 0.20 | --- | ----- |
| 0.80 | 1.718 | 1.714 | 0.41 | 0.53 | 4.9E-03 | 0.0015 |
| 1.60 | 1.698 | 1.690 | 1.15 | 1.41 | 4.4E-03 | 0.0032 |
| 3.20 | 1.639 | 1.617 | 3.29 | 4.11 | 2.8E-03 | 0.0032 |
| 6.30 | 1.473 | 1.449 | 9.36 | 10.27 | 3.7E-04 | 0.0255 |
| 1.60 | 1.548 | 1.553 | 6.63 | 6.43 | 2.0E-04 | ----- |
| 0.80 | 1.603 | 1.608 | 4.61 | 4.44 | 1.0E-04 | ----- |
| 1.60 | 1.592 | 1.590 | 5.00 | 5.11 | 6.8E-04 | 0.0004 |
| 3.20 | 1.539 | 1.534 | 6.96 | 7.15 | 3.3E-04 | 0.0044 |
| 6.30 | 1.437 | 1.424 | 10.69 | 11.17 | 2.7E-04 | 0.0109 |
| 12.70 | 1.202 | 1.181 | 19.29 | 20.09 | 8.6E-05 | 0.0291 |
| 25.50 | 0.917 | 0.891 | 29.76 | 30.69 | 4.8E-05 | 0.0364 |
| 6.30 | 1.042 | 1.048 | 25.17 | 24.96 | 6.4E-05 | ----- |
| 1.60 | 1.253 | 1.262 | 17.44 | 17.12 | 2.6E-05 | ----- |
| 0.40 | 1.444 | 1.450 | 10.45 | 10.23 | 1.7E-05 | ----- |
| 0.10 | 1.563 | 1.558 | 6.06 | 6.25 | 1.0E-05 | ----- |
| | | | | | | |
| | | | | | | |



Checked By: TM Date: 05/03/16



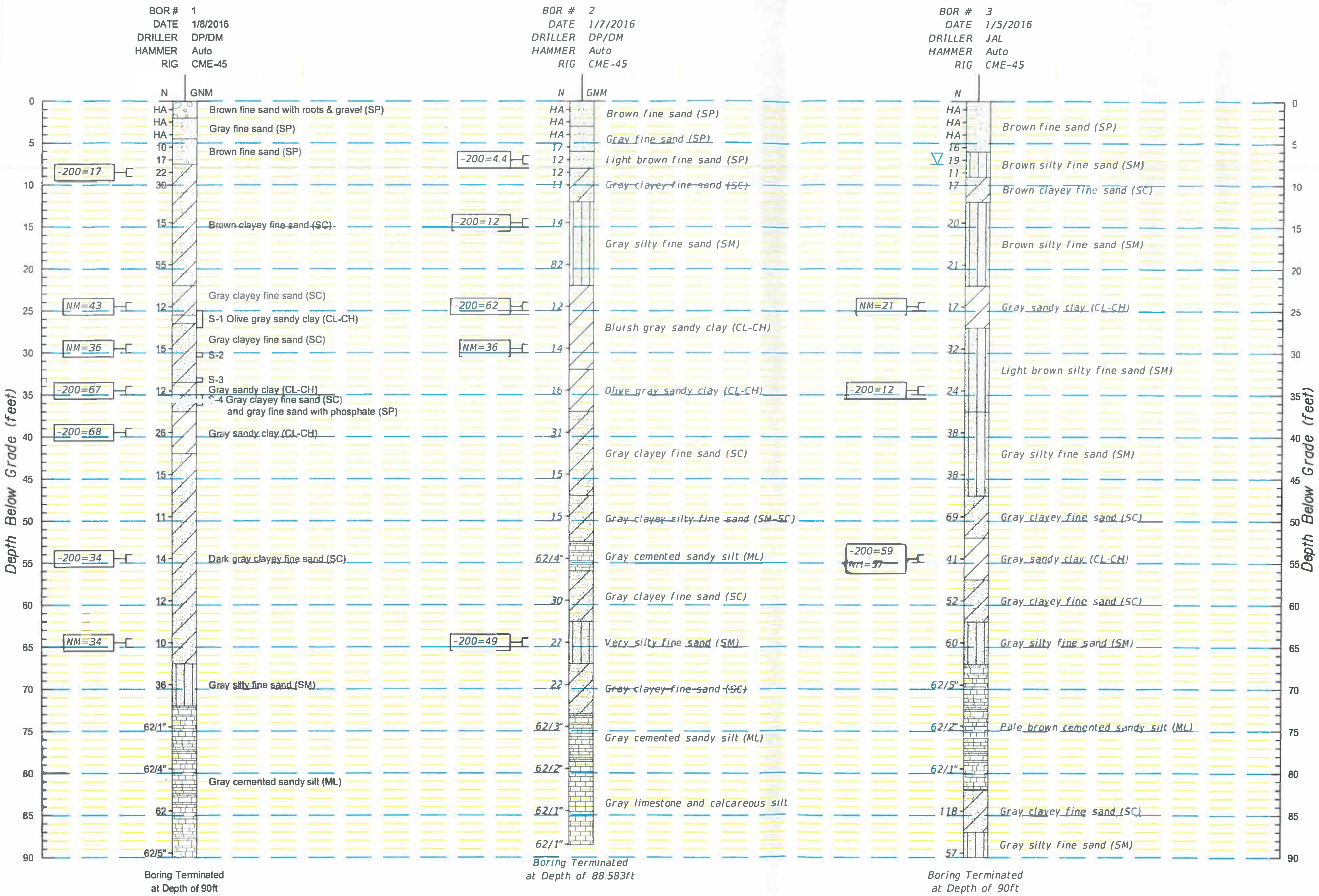

*TEST BORING LOCATIONS*

Ardaman & Associates, Inc.
Geotechnical, Environmental and Materials Consultants

*Test Locations*
*North County WRF Eq. Tank*
*69th St. E. at Erie Rd., Ellenton*
*Manatee County, Florida*

| DRAWN BY: KGS | CHECKED BY: | DATE: 1/29/16 |
|---|---|---|
| FILE NO. 15-7337 | APPROVED BY: | FIGURE: 1 |

*Base PDF Image By: Kimley Horn*




LEGEND

▽ GROUNDWATER LEVEL MEASURED ON DATE DRILLED

N SPT N-VALUE IN BLOWS PER FOOT (UNLESS OTHERWISE NOTED)

SPT N VALUES CONVERTED TO EQUIVALENT SAFETY HAMMER

▯ SHELBY TUBE SAMPLE

HA HAND AUGER

GNM GROUND WATER NOT MEASURED

NM NATURAL MOISTURE CONTENT

-200 FINES PASSING THE NO. 200 SIEVE (%)

| GRANULAR MATERIALS–RELATIVE DENSITY | SPT (BLOWS/FOOT) |
|---|---|
| VERY LOOSE | LESS THAN 4 |
| LOOSE | 4–10 |
| MEDIUM DENSE | 10–30 |
| DENSE | 30–50 |
| VERY DENSE | GREATER THAN 50 |

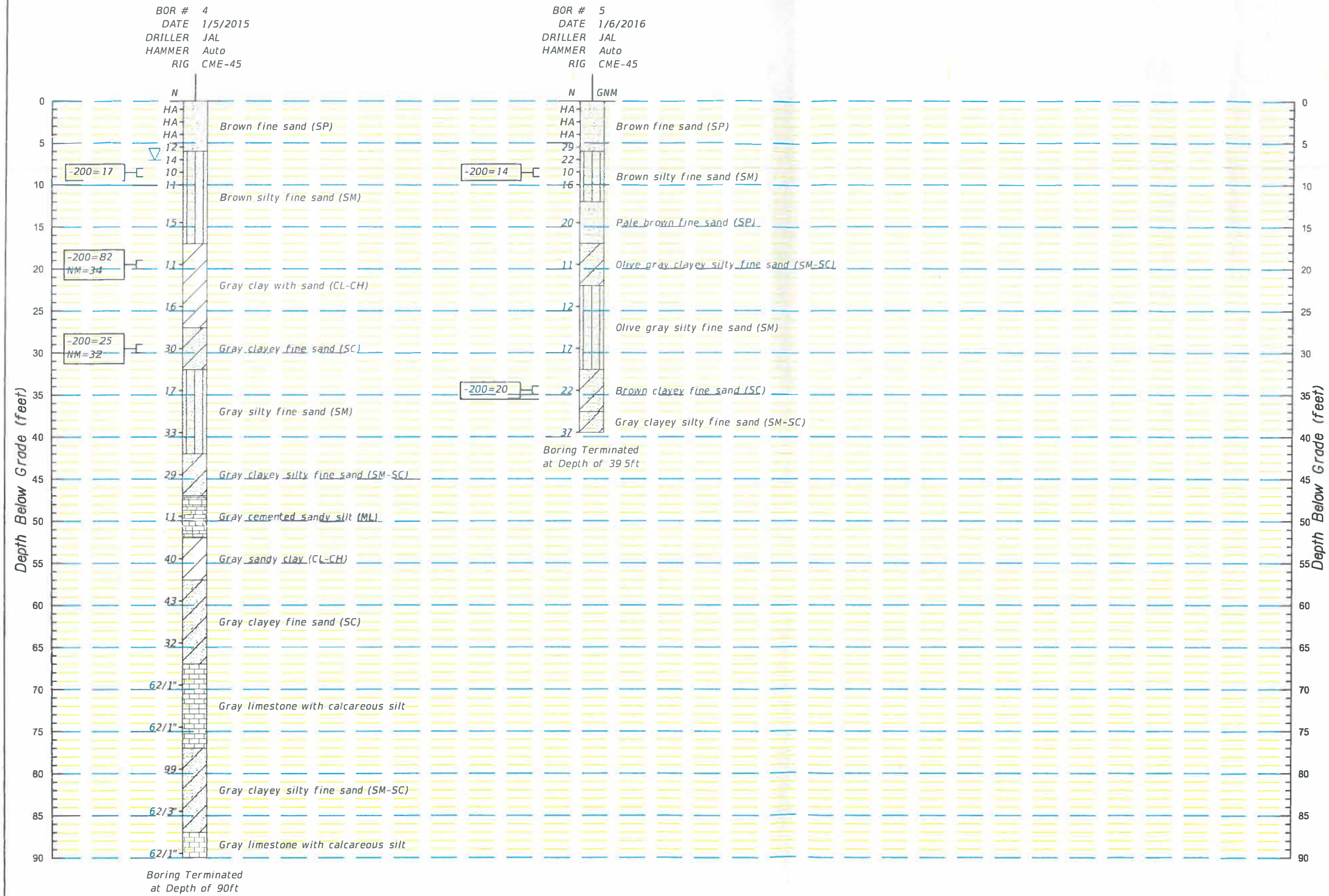
| SILTS AND CLAYS CONSISTENCY | SPT (BLOWS/FOOT) |
|---|---|
| VERY SOFT | LESS THAN 4 |
| SOFT | 2–4 |
| FIRM | 4–8 |
| STIFF | 8–15 |
| VERY STIFF | 15–30 |
| HARD | GREATER THAN 30 |

Revised 5/16/2016

Ardaman & Associates, Inc.
Geotechnical, Environmental and Materials Consultants

**Soil Boring Logs**
**North County WRF Eq. Tank**
**69th St. E. at Erie Rd., Ellenton**
**Manatee County, Florida**

| DRAWN BY: KGS | CHECKED BY: | DATE: 1/29/16 |
|---|---|---|
| FILE NO. 15-7337 | APPROVED BY: | FIGURE: 2 |




LEGEND

- ▽ GROUNDWATER LEVEL MEASURED ON DATE DRILLED
- N SPT N-VALUE IN BLOWS PER FOOT (UNLESS OTHERWISE NOTED)
- SPT N VALUES CONVERTED TO EQUIVALENT SAFETY HAMMER
- HA HAND AUGER
- GNM GROUND WATER NOT MEASURED
- NM NATURAL MOISTURE CONTENT
- -200 FINES PASSING THE NO. 200 SIEVE (%)

| GRANULAR MATERIALS– RELATIVE DENSITY | SPT (BLOWS/FOOT) |
|---|---|
| VERY LOOSE | LESS THAN 4 |
| LOOSE | 4–10 |
| MEDIUM DENSE | 10–30 |
| DENSE | 30–50 |
| VERY DENSE | GREATER THAN 50 |

| SILTS AND CLAYS CONSISTENCY | SPT (BLOWS/FOOT) |
|---|---|
| VERY SOFT | LESS THAN 4 |
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| HARD | GREATER THAN 30 |

Revised 5/16/2016

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**Soil Boring Logs**
**North County WRF Eq. Tank**
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**Manatee County, Florida**

| DRAWN BY: KGS | CHECKED BY: | DATE: 1/29/16 |
|---|---|---|
| FILE NO. 15-7337 | APPROVED BY: | FIGURE: 3 |