Geotechnical Investigation Report

Southeast Water Reclamation Facility Septage Station
Manatee County, Florida

Prepared for: **Cardno TBE**
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Prepared By:
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Project No. T121211.252
October 2013

October 10, 2013

Mr. David O'Connor, PE, BCEE
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Clearwater, Florida 33759

Geotechnical Engineering Services Report
Southeast Water Reclamation Septage Station
Manatee County, Florida
MC Squared Project No. T121211.252

Dear Mr. O'Connor:

MC Squared, Inc. (MC^2) has completed our geotechnical engineering services for the referenced project. The results of this subsurface investigation, together with our recommendations, are included in the accompanying report.

We trust that this report will assist you in further design development of the proposed project. We appreciate the opportunity to be of service on this project. Should you have any questions, please do not hesitate to contact us.

Respectfully submitted,
MC^2

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1.0 PROJECT INFORMATION

MC^2 has completed its geotechnical engineering study for the Southeast Water Reclamation Facility Septage Station, located in Manatee County, Florida. Included in this report are the results of the subsurface exploration performed and our recommendations concerning foundations for the proposed new structures included in the project which we have evaluated.

1.1 Project Authorization

Authorization to proceed with this project was issued by **Cardno TBE** through a subcontract agreement for services dated March 29, 2013. A formal contract has been executed between **Cardno TBE** and **MC^2** for these services.

1.2 Project Location and Description

Project information has been provided by Mr. David O'Connor, PE of **Cardno TBE** through verbal and email communications including an aerial photo showing anticipated areas for improvements. Based on our understanding, the proposed work includes the design of a Septage Station directly east of the existing rectangular shaped pond located along the northern border of the existing facility. We further understand that the proposed improvements at the Septage station will include above ground equipment on slabs on grade, below ground structures, above grade storage tank(s), paved areas and associated other miscellaneous improvements.

Conceptual plans for the proposed Station were provided by **Cardno TBE** and in general included the following structures:

- One (1) Vacuum Truck Receiving Area for Jet/Vac Trucks (slab/mat at grade) and one (1) sloped concrete ramp (mat foundation at grade).
- Two (2) Septage receiving stations (partially below grade), supported on a slab/mat at grade and a concrete mat below the ground.
- Two (2) Septage/Grease Tanker areas supported on a slab/mat on grade.
- One (1) Wet well and a grease receiving station supported on mat foundation below grade.
- Two (2) Above ground grease storage tanks supported on mat foundations at grade.
- Two (2) Overhead Canopies (pre-engineered galvanized steel) supported on strip footings and/or isolated columns.

In addition, a new entrance roadway to the proposed facility may be required if existing roadways cannot be utilized. The new roadway is proposed to be located north of the existing pond. Structural loads were not available to **MC^2** for this report.

If any of the noted information is incorrect or has changed, please inform **MC^2** so that we may amend the recommendations presented in this report, if appropriate or necessary.

2.0 SCOPE OF SERVICES

Our geotechnical study began with a review of any available subsurface test data provided to us for previous work performed in the area or at the site. We also reviewed available sources of information including the USDA Manatee County Soil Survey and USGS Maps. The testing program consisted of the following services:

1. Conducted a visual reconnaissance of the project site. Reviewed the USDA Soil Survey for Manatee County and the USGS topographic maps.

2. Cleared utilities in the vicinity of the proposed boring locations.

3. Performed six (6) Standard Penetration Test (SPT) borings in areas of proposed improvements to depths of 30 feet. The locations and depths of borings were determined by **Cardno TBE** using the approved Septage Station site plan. The borings were labeled B-1S through B-6S.

4. Performed ten (10) hand auger borings to a depth of approximately 5 feet in areas of proposed new roadway improvements**.** The borings were performed along the proposed roadway. The borings were labeled AB-1S through AB-10S.

5. Performed two (2) pavement cores in the existing roadway located south of the pond to evaluate existing pavement structure and conditions. The cores were labeled PC-1S and PC-2S.

6. Collected two (2) samples and provided Limerock Bearing Ratio (LBR) values of existing in-place material in proposed area of new roadway north of the existing pond.

7. Visually examined all recovered soil samples in the laboratory and performed laboratory tests on selected representative samples to develop the soil legend for the project using the Unified Soil Classification Systems, as appropriate. The laboratory testing included percent passing the No. 200 sieve, organic and natural moisture content determination. In addition, performed limerock bearing ratio tests.

The data was used in performing engineering evaluations, analyses, and for developing geotechnical recommendations in the following areas:

1. General assessment of area geology based on our past experience, study of geological literature and boring information.

2. General location and description of potentially deleterious materials encountered in the borings, which may interfere with the proposed construction or performance, including existing fills or surficial organics.

3. Discuss critical design and/or construction considerations based on the soil and groundwater conditions developed from the borings including dewatering, hard soil conditions etc.

4. Address groundwater levels in the borings and estimate seasonal high groundwater.

5. Recommendations for construction including a summary of findings and analysis. Provided a summary of the pavement cores results.

6. Recommendations for shallow foundation design and construction including recommended horizontal earth pressures (active, passive and at-rest) for below grade walls.

All information has been provided in a Geotechnical Investigation Report which will generally include the following:

a. Description of the proposed project.
b. Plot showing location of borings performed.
c. Boring logs including water table where encountered.
d. Description of surface and subsurface conditions encountered.
e. Internal friction angles, cohesion.
f. Active, passive and at rest soil pressures.
g. Recommendations for site preparation and engineered fill.
h. Recommendations for support of slab-on-grade and below grade structures.
i. Recommendations for temporary sheet pile shoring design.
j. Analysis for deep foundations is not anticipated and is not included.

3.0 SUBSURFACE CONDITIONS

3.1 MANATEE COUNTY SOIL SURVEY

The U.S. Department of Agriculture - Soil Conservation Service, now known as Natural Resources Conservation Service (NRCS), has mapped the shallow soils in this area of Manatee County. This information was outlined in a report titled *The Soil Survey of Manatee County, Florida* using Version 8, dated July 6, 2012. The aerial images were photographed in February 10, 2010. The proposed reject water pipeline alignment is within areas mapped as Eau Gallie fine sand (No. 20). Small areas of other similar and dissimilar soils may be present in the mapping unit.

Typically the surface layer of the Eau Gallie soil is black fine sand. The surface layer is underlain by gray fine sand to a depth of 22 inches. Dark reddish brown sand grading to dark brown fine sand is usually indicated to a depth of 44 inches and is followed by gray fine sand. From depths of 48 to 66 inches, grayish brown sandy loam occurs, which grades to gray sandy loam that continues to a depth of about 80 inches or more. The Eau Gallie soil in its natural state has a seasonal high water table at a depth of 6 to 18 inches for 1 to 3 months and within a depth of 40 inches for 2 to 6 months. The water table recedes to a depth of more than 40 inches during extended dry periods. This soil and the estimated SHWT are summarized in **Table 1 in Appendix A.**

The USDA Soil Survey is not necessarily an exact representation of the soils on the site. The mapping is based on interpretation of aerial maps with scattered shallow borings for confirmation. Accordingly, borders between mapping units are approximate and the change may be transitional. Differences may also occur from the typical stratigraphy, and small areas of other similar and dissimilar soils may occur within the soil-mapping unit. As such, there may be differences in the mapped description and the boring descriptions obtained for this report. The survey may, however, serve as a good basis for evaluating the shallow soil conditions of the area.

3.2 Standard Penetration Test (SPT) and Hand Auger Borings

A total of six (6) SPT borings were performed to depths of 30 feet and labeled B-1S through B-6S. The SPT boring procedure was conducted in general accordance with the ASTM test designation D-1586. Closely spaced samples using 4 inch ID split-barrel samples were performed in the upper 10 feet with 5-foot sample intervals used thereafter. After seating the SPT samples 6” into the soil, the number of successive blows required to drive the sampler 12” into the soil constitutes the test results commonly referred to as the “N” valve. The “N” valve has been empirically correlated with various soil properties and is considered to be indicative of the soil density of cohesionless soils and the consistency of cohesive soils. The recovered split spoon samples were visually classified in the field with representative portions of the samples placed in jars and transported to our Tampa office for review by a geotechnical engineer and confirmation of the field classifications. Rock coring was not performed due to the poor quality of the rock encountered and for economical reasons.

In addition, a total of ten (10) hand auger borings were performed to depths of about 5 feet below the existing grades. The hand auger borings were performed by manually twisting and advancing a bucket auger into the ground in 4 to 6-inch increments. As each soil type was revealed, representative samples were placed in air-tight jars and returned to the **MC^2** Tampa office for review by a geotechnical engineer and confirmation of the field classification.

3.3 Generalized Subsurface Conditions

The following subsurface description is of a generalized nature, provided to highlight the major soil strata encountered. The boring logs, included in **Appendix A**, should be reviewed for specific information as to individual test locations. The stratifications shown on the boring logs represent the conditions only at the actual test locations. Variations may occur and should be expected between test locations. The stratifications represent the approximate boundary between and among subsurface materials; however, the transition may be gradual.

The following includes the proposed Septage Station components planned with the corresponding boring number(s) that were performed in their respective areas.

3.3.1 Proposed One Vacuum Truck Receiving Area (B-1S)

In general, the subsurface conditions and materials encountered consisted of medium dense to dense fine sands to slightly silty fine sands to slightly clayey fine sands (SP/SP-SM/SP-SC) to a depth of 12 feet below ground surface (BGS) (elev. 23.5 ft), followed by medium dense clayey fine sand (SC) with traces to some phosphate and cemented clay fragments to a depth of 17 feet (elev. 18.5 ft). Next, the boring entered firm to stiff sandy clay to clay (CL/CH) extending to the boring termination depth of 30 feet (elev. 5.5 ft).

3.3.2 Proposed One Wet Well (B-2S)

In general, the subsurface conditions and materials encountered generally consisted of medium dense fine sands to slightly silty fine sands to slightly clayey fine sands (SP/SP-SM/SP-SC) to a depth of 12 feet BGS (elev. 23.5 ft), followed by very loose to medium dense clayey fine sand (SC) with occasional traces to some phosphate to a depth of 27 feet (elev. 8.5 ft). Next, the boring entered very stiff sandy clay to clay (CL/CH) extending to the boring termination depth of 30 feet (elev. 5.5 ft).

3.3.3 Proposed Above Grade Septage Storage Tanks (B-3S and B-4S)

In general, the subsurface conditions and materials encountered generally consisted of (No N-value top 6 feet assume very loose) very loose to dense fine sands to slightly silty fine sands to slightly clayey fine sands (SP/SP-SM/SP-SC) with occasional traces to some phosphate to depths ranging from 12 to 17 feet BGS (elev. 25.5 ft to 20 feet). This cleaner sand was followed by a layer of very loose clayey fine sand (SC) with occasional traces to some phosphate to a depth of 17 feet (elev. 20.0 ft) in both borings. Next, the borings entered firm to stiff sandy clay to clay (CL/CH) extending to the boring termination depth of 30 feet (elev. 5.5 ft).

3.3.4 Proposed Septage Receiving Stations (B-5S and B-6S)

In general, the subsurface conditions and materials encountered consisted of (No N-value top 6 feet assume very loose) very loose to medium dense fine sands to slightly silty fine sands to slightly clayey fine sands (SP/SP-SM/SP-SC) to a depth of 12 feet below ground surface (BGS) (elev. 24.5 ft), followed by medium dense clayey fine sand (SC) with large phosphate fragments or cemented clay fragments to a depth of 17 feet (elev. 19.5 ft). Next, the boring entered firm to very stiff sandy clay to clay (CL/CH) extending to the boring termination depth of 30 feet (elev. 6.0 ft).

3.4 Groundwater Information

Groundwater was encountered in the majority of borings performed at depths ranging from about 4.0 to 6.0 feet BGS and the groundwater table was not encountered within the depth explored of 5 feet in borings AB-3S and AB-4S. It should be noted that our field investigation was done during what is considered to be "the wet season" but dry period. Fluctuations of groundwater levels should be anticipated and we expect that the actual water level will be higher than noted in our borings. We recommend that the contractor determine the actual groundwater levels at the time of construction to determine groundwater impact on the construction procedures. According to the Manatee County Soil Survey, a Seasonal High Water (SHW) table of 0.5 to 1.5 foot BGS is reported. The results of the groundwater levels in the borings and estimated seasonal high groundwater level is presented in **Table 1, Appendix A**. Need for dewatering is not anticipated but may be required if below grade construction is expected.

For limited, relatively shallow excavations below the groundwater level, pumping from the excavation or sumps should be sufficient to control groundwater seepage. Deeper and larger excavations, such as the ones required for the septage/grease receiving station will require more sophisticated dewatering measures such as well points or cut-off walls, which may be difficult but possible.

3.5 Pavement Cores Information

Two (2) pavement cores were performed and labeled PC-1S and PC-2S and the information is included in the table below. The information, measurements and photos for all the cores taken are presented in **Appendix A**.

Summary of Pavement Core Results			
Core No.	**Core Location (See Sheet 1)**	**Type of Material and Averaged Thickness**	
		Asphalt Thickness[1] (in.)	**Field Measurement of Base and Type (in.)**
PC-1S	-	2.43	10.00 – Shell
PC-2S	-	2.48	9.00 - Shell
Avg. Layer Thickness (in.)		**2.46**	**9.50**
Layer Thickness Range (in.)		**2.43 to 2.48**	**9.00 to 10.00**
Notes:			
1.	**The thicknesses were measured at four locations around the core using a calibrated caliper and averaged.**		

4.0 LABORATORY TESTING

4.1 General

Laboratory tests were assigned to aid in the classification of the explored soils. These tests included organic and moisture content determination tests, percent passing the -200 Sieve and limerock bearing ratio tests. The laboratory test results are presented in **Tables 2 and 3 in Appendix A.**

4.2 Soil Classification

The SPT soil samples were classified using the USCS in general accordance with ASTM test designation D-2488. This test method classifies soils into specific categories based upon the results of the laboratory testing program. The assignment of a group name and symbol is then used to aid in the evaluation of the significant engineering properties of a soil.

4.3 Organic Content Test

The organic content testing procedure generally followed ASTM 2974 (Standard Test Methods for Moisture, Ash, and Organic Matter of Peat and Other Organic Soils).

4.4 Moisture Content

The laboratory moisture content test consists of the determination of the percentage of moisture contents in selected samples in general accordance with FDOT test designation 1-T265 (ASTM test designation D-2216).

4.5 Percent Passing the No. 200 sieve

The wash gradation test this was performed in general accordance with ASTM D 1140 (Standard Test Methods for Amount of Material Finer Than the No. 200 (75 μm) Sieve).

4.6 Limerock Bearing Ratio (LBR) Tests

LBR tests were performed in accordance with the Florida Department of Transportation Standard FM 5-515 including a Modified Proctor test. The LBR test is a measure of the bearing capacity of a soil. The test consists of measuring the load required to cause a 3 in^2 circular plunger to penetrate a specimen at a specified rate. The LBR is the load required to force the plunger into the soil 0.1 inches expressed as a percentage of the load in psi required to force the plunger the same depth into the standard sample of crushed limerock. The average penetration load for a typical crushed limerock found in Florida has been standardized to 800 psi. Results of the LBR tests results are presented in **Table 3 in Appendix A.**

5.0 EVALUATION AND RECOMMENDATIONS

5.1 Site Preparation

Any organic or other deleterious material at or just below the existing ground surface should be removed from development areas. Demolition, site grubbing and stripping should be performed during dry weather conditions to reduce the potential for the operation of heavy equipment, which may cause rutting and mixing of surficial debris with otherwise suitable underlying soils.

We recommend that any proposed construction areas to receive fill be evaluated by proofrolling prior to fill placement. Areas of mass excavation below existing grades should be proofrolled after final grade has been achieved. Proofrolling should be performed by traversing the construction areas with a loaded dump truck or similar vibratory compactor equipment weighing at least 20 tons. Proofrolling operations should be observed by a Manatee County designated representative. Unstable soils which are revealed by proof rolling and which cannot be adequately densified in place should be removed and replaced under the recommendations of the Manatee County designated representative. The contractor should exercise caution during the proof rolling and compaction of soils as not to cause settlement of the existing structures due to vibrations.

In areas where shallow groundwater is encountered, proofrolling should not be performed due to the potential for degrading an otherwise acceptable subgrade, and alternate means of evaluating the subgrade soils should be used. The contractor must control and adjust the vibration as not to disturb any existing structures and/or subsurface utilities that may be in the vicinity of the project. The contractor is solely responsible for any settlement caused by his/her actions.

While not as effective as vibratory compaction, non-vibratory options for compacting soils include the use of pneumatic tire rollers.

Burn pits, trash pits or other isolated disposal areas may be encountered outside the boring locations. Any such material encountered during site work or foundation construction should be excavated and removed from the site. Abandoned underground pipes, if left in place, should be filled with flowable concrete.

5.2 Selection and Placement of Structural Fill

After the stripped site (with a sand subgrade) has been proofrolled and accepted by the Geotechnical Engineer, fill required to bring the site to final grade may be placed and properly compacted. Fill material should be inorganic, non-plastic granular soil (clean to slightly silty or slightly clayey sands, Unified Soil Classification (SP, SP-SM, SP-SC) with less than 12 percent passing the No. 200 sieve. The material should be free of detrimental materials such as clay clods, debris, roots, rocks larger than 1 inch in greatest dimension, etc. Materials selected to be used as structural fill should not contain more than 3 percent by weight of organic matter. The majority of the upper portion of the on-site near surface sandy soils (thickness varies from place-to-place) other than root laden topsoils will meet this requirement; careful evaluation should be made of any clayey soils prior to use.

The fill should be placed in level lifts not to exceed 12 inches loose thickness. The fill should be compacted to a minimum of 95% of the soil’s modified Proctor maximum dry density as determined by ASTM designation D-1557. We suggest the following minimum testing frequency, per layer of fill placed: one test per 2500 square feet of structure area and one test per 5000 square feet of pavement area. This fill should extend a minimum of 10 feet beyond building/structure lines to prevent possible erosion or undermining of footing bearing soils. Furthermore, fill slopes should not exceed 2 horizontal to 1 vertical.

All fill placed in utility line trenches and adjacent to footings beneath slabs-on-grade should also be properly placed and compacted to the specifications stated above. However, in these restricted working areas, compaction should be accomplished with light weight, hand-guided compaction equipment, and lift thickness should be limited to a maximum of 6 inches loose thickness. To facilitate compaction, the fill moisture content should be controlled to within 2 percentage points of the optimum determined by the modified Proctor test (ASTM D-1557).

We also recommend that proposed fill materials be tested prior to beginning earthwork to determine if their material characteristics meet the above criteria.

The moisture content of fill soils at the time of placement and compaction should generally be within plus or minus two percentage points of their optimum moisture content. More stringent moisture limits may be necessary with certain soils. Localized dewatering may be required depending on the time of the year in order to control moisture.

We recommend that structural fill and backfill be compacted to a minimum of 95 percent of the Modified Proctor maximum dry density (ASTM D-1557). The upper 12 inches of floor slab and pavement subgrade soils should be compacted to at least 98 percent of the Modified Proctor maximum dry density (ASTM D-1557). The Manatee County designated representative should observe fill placement operations and perform density tests concurrently to indicate if the specified compaction is being achieved.

5.3 Reuse of Excavated Soils as Structural Fill

The residual soils at the site will, in our opinion, be suitable for reuse as structural fill materials. Routine adjustment of moisture content will generally be necessary to allow compaction in accordance with project specifications. The planned fill soils should be evaluated to see that they meet the recommended material properties.

5.4 Federal Temporary Excavation Regulations

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

5.5 Drainage and Groundwater Considerations

Groundwater may be a concern dependant on final grades and the time of year construction is performed. For limited, relatively shallow excavations below the groundwater level, pumping from the excavation or sumps should be sufficient to control groundwater seepage. Deeper and larger excavations, such as the ones required for the Septage/Grease receiving station will require more sophisticated dewatering measures such as well points or cut-off walls, which may be difficult but possible. We understand that temporary sheet pile walls are anticipated for dewatering and for construction of some of these structures. The walls will be designed by others using the parameters shown on the table on Page 14 of this report.

Undercut or excavated areas should be sloped toward one corner to facilitate removal of any collected rainwater, groundwater or surface water runoff. Positive site drainage should be provided to reduce infiltration of surface water around the perimeter of any above-ground structures and beneath the floor slabs. All grades should be sloped away from the structures and surface drainage should be collected and discharged such that water is not permitted to infiltrate the immediate area surrounding structures.

5.6 Temporary Structure Excavations

Where possible, temporary excavations should be "opened" and should have minimum slopes on the order of 1.5 (H):1.0 (V). Deeper and larger excavations, such as the ones required for the proposed septage/grease receiving station structures, may require more sophisticated dewatering measures such as well points or cut-off walls.

All structure excavations should be observed by the Geotechnical Engineer or his representative to explore the extent of any fill and excessively loose, soft, or otherwise undesirable materials. If the excavated subgrade appears suitable as load bearing materials, the soils should be prepared for construction by compaction to a dry density of at least 98 percent of the modified Proctor maximum dry density (ASTM D-1557) to a depth of at least 1 foot below the foundation base.

If soft pockets are encountered in the bottom of the structure excavations, the unsuitable materials should be removed and the proposed foundation elevation re-established by backfilling after the undesirable material has been removed. This backfilling may be done with a very lean concrete or with a well-compacted, suitable fill such as clean sand, gravel, or crushed #57 or #67 stone. Sand backfill should be compacted to a dry density of at least 98 percent of the modified Proctor maximum dry density (ASTM D-1557).

5.7 Existing Facilities

An existing Septage/Grease and Vacuum Truck material receiving facility is located along the southern paved access road. The type of foundation of the existing structures at the Southeast Water Reclamation Facility is not available.

5.8 Foundation Alternatives

Several foundations systems were considerated based upon the planned construction, noted subsurface conditions and our experience with similar projects. In general, the following foundation alternatives are being considered:

- Shallow Foundations – Strip Footings, Isolated columns
- Shallow Foundations – Mat Foundations
- Slab on grade

Specifics of our recommendations for site work and foundation design and construction are presented in the sections that follow.

a. Shallow Foundation Systems and Slab on Grade – The use of slab on grade for the proposed improvements at this site is unfeasible for the majority of heavily loaded structures. However, the proposed new canopies can be supported on shallow foundation systems (strip footings and/or column footings).

5.9 Foundation Recommendations

a. Shallow Foundations - Strip Footings and/or Isolated Columns (Canopies)

- Two (2) Overhead Canopies (pre-engineered galvanized steel).

Based on the results of the test borings, our engineering evaluation and recommendations, it is our opinion that the proposed canopy structures may be supported on shallow foundations. Foundations or bottom slabs of below grade structures may bear on newly placed, properly compacted fill. Without knowing specific grades and being able to correlate them to existing conditions, we cannot make specific foundation bearing recommendations at this time for each structure. However, the following can be used as a guideline for design of foundations:

Footings Bearing on Compacted Fill **2500 psf**

It may be possible to utilize existing on site fill materials for foundation support depending upon the type and quality of the fill material and the expected loads. An allowable bearing pressure of 3000 psf may be used for existing fills properly compacted, if encountered.

Isolated column foundations should have a minimum width of 30 inches, even though allowable bearing pressures may not be fully developed in all cases. Continuous wall foundations should have a minimum width of 24 inches. All foundations should bear at a minimum depth of 24 inches below the lowest adjacent final ground surface.

We estimate maximum total and differential settlements on the order of 1 inch and ½ inch, respectively. These values for estimated total and differential settlements should be increased by 50 percent for foundations bearing on existing fill materials.

It is possible that some soft or loose soils will not be identified and properly remediated during site preparation. We recommend that the bearing soils at the bottom of and below the footing excavations be checked to assess the suitability of the soils. Footing evaluations should be performed prior to reinforcement and concrete placement. If unsuitable bearing soils are encountered, these soils will need to be re-compacted in place, removed and replaced with properly compacted fill (or foundations deepened) to achieve suitable bearing.

After footings are excavated, foundation bearing surface evaluations should be performed and concrete placed as quickly as possible to avoid exposure of the footing bottoms to changes in moisture contents. If it is required that foundation excavations be left open for more than one day, they should be protected to reduce evaporation or entry of moisture.

b. Shallow Foundations - Mat Foundation Recommendations

Based on the results of our test borings, a mat foundation system can be used to support the proposed following structures after proper subgrade preparation and placement of fill:

- One (1) Vacuum Truck Receiving Area for Jet/Vac Trucks (slab/mat at grade) and one (1) sloped concrete ramp (mat foundation at grade).
- Two (2) Septage receiving stations (partially below grade), supported on a slab/mat at grade and a concrete mat below the ground.
- Two (2) Septage/Grease Tanker areas supported on a slab/mat on grade.
- One (1) Wet well and a grease receiving station supported on mat foundation below grade.
- Two (2) Above ground grease storage tanks supported on mat foundations at grade.

The net allowable bearing pressure should not exceed 2,500 psf. The mat design may be conducted using a modulus of subgrade reaction (k_s) of 150 pounds per cubic inch for densified in-place soils or compacted structural fill. Minimum embedment depths for this type of foundation system should be 3 feet, which can be achieved with the use of a perimeter key extending below the mat. We suggest the placement of a minimum of 4 inches of stone such as FDOT No. 57 or FDOT No. 67 beneath the slab if conditions warrant (i.e., wet conditions). An impermeable vapor barrier may be utilized; however, the final decision to use a vapor barrier is left to the owner and designer.

Existing sandy soils may be reused after removing all organic matter to build up the grade, if required. Alternatively, durable crushed stone may be used below the groundwater and would not require compaction. The soil subgrade in the area of concrete mat support is often disturbed during foundation and superstructure construction. We recommend that the floor slab subgrade be evaluated by the Manatee County designated representative immediately prior to placing stone and beginning floor slab construction. If low consistency soils are encountered which cannot be adequately densified in place, such soils should be removed and replaced with well-compacted structural fill material.

For the purpose of estimating the settlement for the mat foundation, we used a total contact pressure of about 2800 psf. We also assumed, based on the soil conditions encountered in our borings, that most of the settlement will occur during load applications. Based on these assumptions, we anticipate that the total settlement should not exceed one (1) inch and differential settlement should not exceed one half (1/2) inch.

While the proposed new tanks may be able to withstand the anticipated settlement, the utility connections to the tanks will likely experience failure. As a result, we recommend that the above ground septage and grease storage tanks be filled with water prior to utility connections in order to reduce the differential settlement between the tanks and the utilities. The tanks should be filled with water in stages and should not be filled completely at once. Careful monitoring of settlement should be implemented, as discussed later. It is important to note that some additional differential settlement will likely occur if the tanks are emptied and re-filled; the fact remains that the magnitude of settlement will be greatly reduced due to the effect of preloading the tanks.

c. Below Grade Walls

Below grade walls restrained at the top should be designed for "at rest" earth pressure conditions. Retaining walls that are free to deflect should be designed for "active" earth pressure conditions. The "passive" earth pressure state should be used for soils supporting the retaining structure, such as toe backfill.

The table below presents recommended values of earth pressure coefficients based on our experience with soils in the area. Equivalent fluid pressures are also provided for conditions above and below the water table.

Earth Pressure State	**Earth Pressure Coefficient**	**Equivalent Fluid Pressure (pcf)**	
		Above Water	Below Water
At-Rest	0.53	60	90
Active	0.36	45	80
Passive	2.75	--	--

These design recommendations have assumed that the wall has horizontal backfill and no surcharge loads, using soils with an approximate angle of internal friction of 28 degrees, no cohesion, a total unit weight of 120 pcf, no factor of safety. Since a permanent drainage system behind the below grade walls of the structure will not be practical, the design needs to include hydrostatic pressures also. For analysis of sliding resistance of the base of the retaining walls, the ultimate coefficient of friction may be taken as 0.34 between concrete and soil. If the walls are designed using earth pressure coefficients, the hydrostatic pressure due to groundwater must be included.

d. Settlement Monitoring for Tanks

As an integral part of our foundations recommendations, settlement monitoring of the septage and grease storage tanks should be implemented and carefully documented, particularly during the initial loading of the tanks (filling with water in stages). If settlement values exceed tolerable limits for the structures or the utility connections, then remedial measures should be implemented which will include adjustment of utility connections, sealing any cracks (which may have developed) or any other appropriate measures.

We recommend that four reference points be established around the perimeter of the tanks in order to monitor their settlement during the initial filling of the tanks with water. The reference points should be placed at about 90 degrees from each other to facilitate the monitoring of all corners. A bench mark should also be established at a distance greater than 120 feet from the foot print of the tanks. A Florida licensed surveyor should be retained to document the change in elevations due to settlement and to establish the initial bench mark and reference points. A Florida licensed, Geotechnical engineer should be retained to establish the settlement monitoring system and monitor the settlement during the filling of the tanks.

We recommend that settlement measurements be taken at stages during the filling of the septage and grease storage tanks with water. We recommend that measurements be taken at 25%, 50%, 75% and 100% of tank volume. We further recommend that a daily monitoring of the tanks settlement be taken for the first week after the tanks are filled to 100% capacity and then once a week for 3 to 4 weeks thereafter. If settlement has stabilized after the third week, then no further monitoring will be required. All settlement monitoring should be documented by the Geotechnical Engineer and a final report submitted prior to the installations of any mechanical devices and utility connections being made to the tanks

e. Slab-on-Grade Recommendations for Lightly Loaded Slabs (since loads were not available to MC^2, the decision to use this system should be made by the structural engineer based on actual loads)

A concrete slab-on-grade may be utilized as needed for the project that will bear at or below grade. We suggest the placement of a minimum of 4 inches of stone such as FDOT No. 57 or FDOT No. 67 beneath the slab if conditions warrant (i.e., wet conditions). An impermeable vapor barrier may be utilized; however, the final decision to use a vapor barrier is left to the owner and designer.

We recommend that the upper 12 inches of the subgrade in the area of the concrete slab-on-grade be re-compacted prior to stone placement. The soil subgrade in the area of concrete slab-on-grade support is often disturbed during foundation and superstructure construction. We recommend that the floor slab subgrade be evaluated by a Manatee County designated representative immediately prior to placing stone and beginning floor slab construction. If low consistency soils are encountered which cannot be adequately densified in place, such soils should be removed and replaced with well-compacted fill material or with well-compacted crushed stone materials.

5.10 Concrete and Asphalt Pavement Design Considerations

LBR test results are presented in **Table 3, Appendix A** and ranged from 24 to 34.

6.0 REPORT LIMITATIONS

The recommendations submitted are based on the available soil information obtained by **MC^2** and design details furnished by **Cardno TBE** for the proposed project. If there are any revisions to the plans for this project or if deviations from the subsurface conditions noted in this report are encountered during construction, **MC^2** should be notified immediately to determine if changes in the foundation, or other, recommendations are required. If **MC^2** is not retained to perform these functions, **MC^2** cannot be held responsible for the impact of those conditions on the performance of the project.

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

After the plans and specifications are more complete, the geotechnical engineer should be provided the opportunity to review them to assess that our engineering recommendations have been properly incorporated into the design documents. At that time, it may be necessary to submit supplementary recommendations. This report has been prepared for the exclusive use of **Cardno TBE** for the specific application to the proposed improvements at the **Southeast Water Reclamation Facility Septage Station in Manatee County, Florida.**

APPENDIX A

Table 1 - Summary of Boring Information, Groundwater Levels and Estimated Seasonal High Groundwater Level

Table 2 -Summary of Laboratory Test Results

Table 3 - Summary of Limerock Bearing Ratio Test Results

Sheet 1 - Boring Location Plan

Sheets 2 and 3 - Report of Core Borings

Table 1
Summary of Boring Information, Groundwater Levels and Estimated Seasonal High Groundwater Level
Southeast Water Reclamation Facility Septage Station
Manatee County, Florida
MC² Project No. T121211. 252

Boring No.	Approximate Boring Location	USDA Soil Type	USDA SHWT Depth (feet)	Existing Ground Surface Elevation (feet) (See Note 1).	Measured Groundwater Depth on August 2013 (feet)	Measured Groundwater level Elevation (ft)	Estimated SHGWT Levels	
							Depth (feet)	Elevation (ft)
Southeast Water Reclamation Facility Septage Station								
B-1S	See Sheet 1	(No. 20) Eau Gallie fine sand	0.5 -1.5	35.5	4.0	31.5	-	-
B-2S	See Sheet 1			35.5	4.0	31.5	-	-
B-3S	See Sheet 1			37.5	6.0	31.5	-	-
B-4S	See Sheet 1			37.0	6.0	31.0	-	-
B-5S	See Sheet 1			36.0	4.0	32.0	-	-
B-6S	See Sheet 1			36.5	4.0	32.5	-	-
AB-1S	See Sheet 1			36.5	4.5	32.0	-	-
AB-2S	See Sheet 1			35.0	4.5	30.5	-	-
AB-3S	See Sheet 1			37.0	GNE	<32.0	-	-
AB-4S	See Sheet 1			35.5	GNE	<30.5	-	-
AB-5S	See Sheet 1			35.5	4.0	31.5	2.0	33.5
AB-6S	See Sheet 1			37.5	4.0	33.5	-	-
AB-7S	See Sheet 1			36.0	4.5	31.5	2.0	34.0
AB-8S	See Sheet 1			35.5	4.0	31.5	2.0	33.5
AB-9S	See Sheet 1			37.0	4.5	32.5	-	-
AB-10S	See Sheet 1			37.0	4.0	33.0	-	-

Notes:

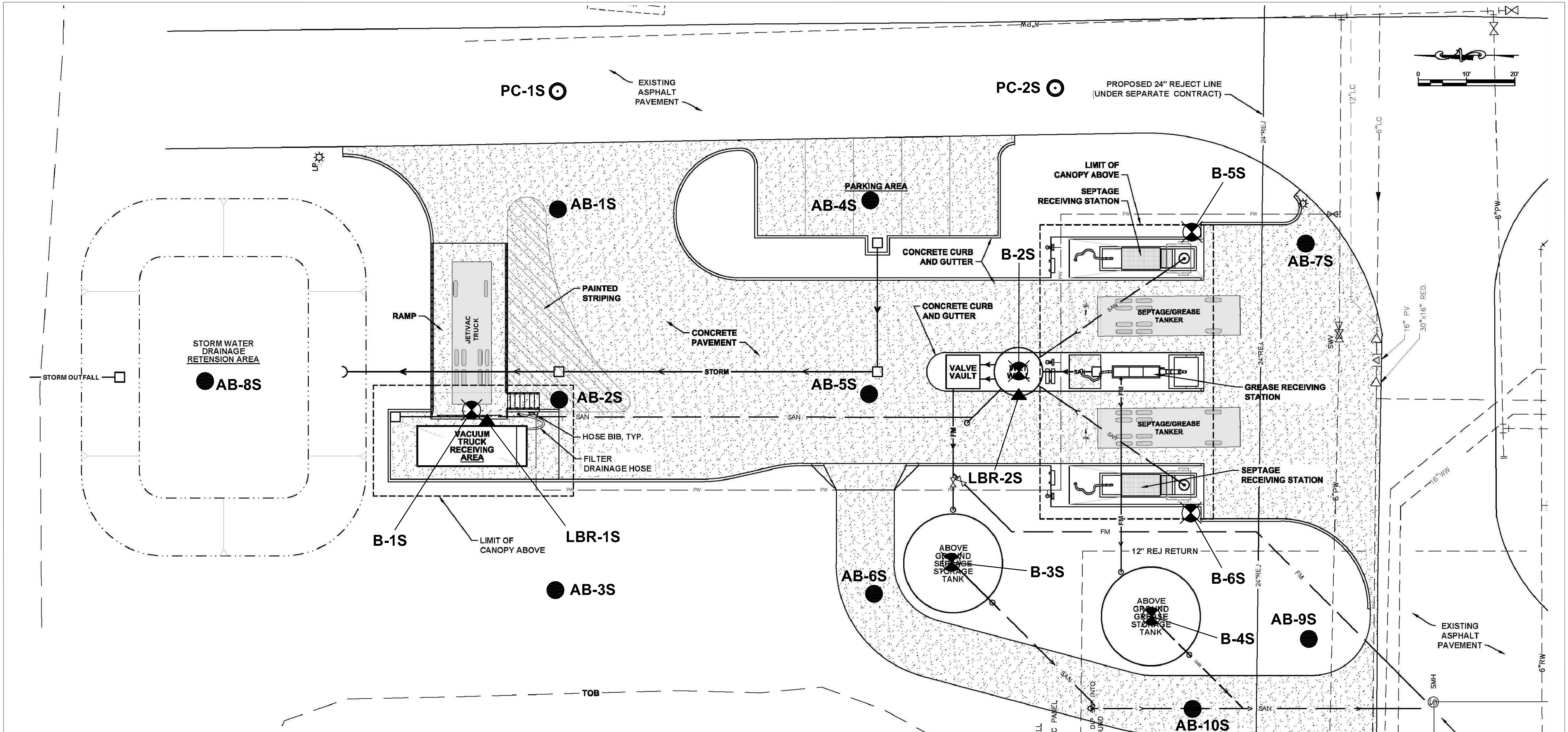
1. Boring ground surface elevation obtained from a spot elevation drawing provided by **Cardno TBE** and is approximate.
2. GNE = Groundwater elevation not encountered within the depth explored
3. Seasonal High Groundwater Levels were determined in the areas of the proposed stormwater pond (AB-8S), septage/grease receiving stations (AB-7S), and in between the vacuum truck receiving and septage/grease receiving areas (AB-5S).

Table 2 Summary of Laboratory Test Results Southeast Water Reclamation Facility Septage Station Manatee County, Florida MC² Project No. T121211. 252														
Boring No.	**Depth (ft)**	**USCS Classi.**	**Sieve Analysis (% Passing)**							**Liquid Limit (%)**	**Plastic Index (%)**	**Organic Content (%)**	**Natural Moisture Content (%)**
			#10	**#20**	**#40**	**#60**	**#100**	**#140**	**#200**				
AB-1S	3.0 – 3.5	SP/SP-SM/SP-SC (slightly organic)							10			7	30
AB-2S	2.5 – 3.0	SP/SP-SM/SP-SC (slightly organic)							10			6	33
AB-8S	1.5 – 4.5	SP/SP-SM/SP-SC							8			3	15
B-2S	24.0 – 25.5	SC							42				37
B-3S	19.0 – 20.5	CL/CH							51				44

Table 2 Summary of Laboratory Test Results Southeast Water Reclamation Facility Septage Station Manatee County, Florida MC^2 Project No. T121211. 252														
Boring No.	**Depth (ft)**	**USCS Classi.**	**Sieve Analysis (% Passing)**							**Liquid Limit (%)**	**Plastic Index (%)**	**Organic Content (%)**	**Natural Moisture Content (%)**
			#10	**#20**	**#40**	**#60**	**#100**	**#140**	**#200**				
B-4S	14.0 – 15.5	SP/SP-SM/SP-SC							11				28
B-5S	19.0 – 20.5	CL/CH							98				54

Table 3
Summary of Limerock Bearing Ratio Test Results
Southeast Water Reclamation Facility Septage Station
Manatee County, Florida
MC^2 Project No. T121211. 252

LBR No. and Approx. Location	Soil Description	Test Method	LBR Value	Maximum Dry Density (pcf)	Optimum Moisture Content (%)	Unified Soil Classification
LBR-1S (See Sheet No. 1)	Brown Fine Sand with traces of rock and shell	FM-515	24	110	12	SP/SP-SM/SP-SC
LBR-2S (See Sheet No. 1)	Brown Fine Sand with traces of rock and shell	FM-515	34	111	11	SP/SP-SM/SP-SC



LEGEND

- APPROXIMATE SPT BORING LOCATION
- APPROXIMATE HAND AUGER BORING LOCATION
- APPROXIMATE PAVEMENT CORE LOCATION
- APPROXIMATE LIMEROCK BEARING RATIO (LBR) SAMPLE LOCATION

DATE	NAME	REVISION	APPROVED BY:

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MC² GEOTECHNICAL • ENVIRONMENTAL MATERIALS TESTING

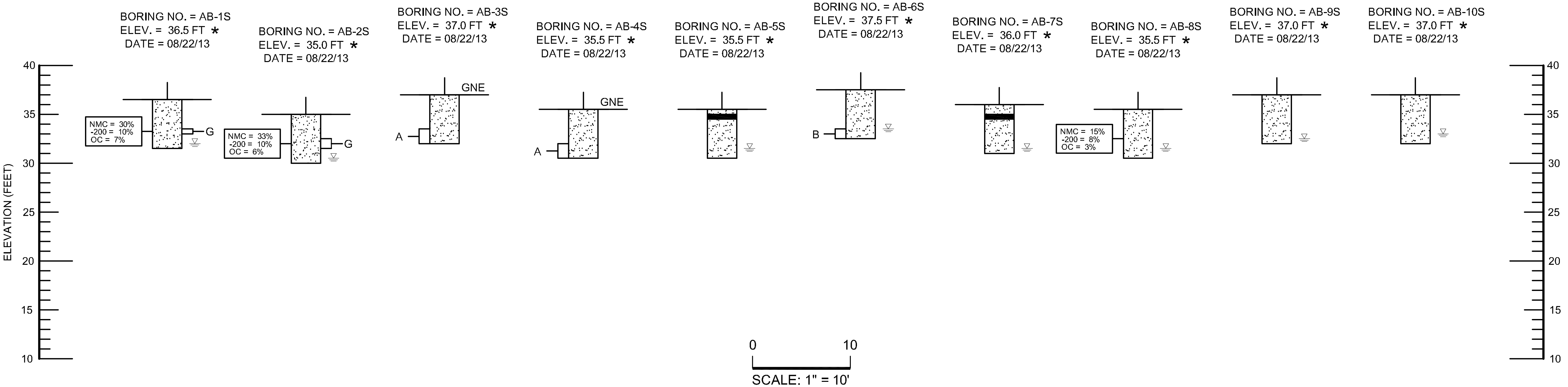
FLORIDA ENGINEERING CERTIFICATE OF AUTHORIZATION No. 9191
Kermit Schmidt, P.E.
FLORIDA LICENSE No. 45603

	NAME	DATE
DESIGNED BY:	IR	09/13
DRAWN BY:	IR	09/13
CHECKED BY:	KS	09/13
SUPERVISED BY:		

BORING LOCATION PLAN

Southeast Water Reclamation Facility
Septage Station
Manatee County, Florida

PROJECT NO.	SHEET NO.
T121211.252	1



LEGEND

(SP/SP-SM/SP-SC) BROWN, LIGHT BROWN, TO DARK BROWN FINE SAND, SLIGHTLY SILTY FINE SAND, TO SLIGHTLY CLAYEY FINE SAND.

SHELL BASE MATERIAL.

(SC) PALE BROWN TO GRAY CLAYEY FINE SAND.

(CL/CH) BROWN TO GRAY SANDY CLAY TO CLAY.

A WITH ROOT FRAGMENTS
B WITH CLAY CLODS
C WITH TRACES TO SOME SHELL
D WITH TRACES TO SOME PHOSPHATE
E WITH CEMENTED CLAY FRAGMENTS
F WITH LARGE PHOSPHATE FRAGEMENTS
G SLIGHTLY ORGANIC

* ELEVATIONS WERE OBTAINED FROM PLANS PROVIDED BY CARDNO TBE AND ARE APPROXIMATE.

NOTES:

WATER TABLE
HA HAND AUGER
GNE GROUNDWATER NOT ENCOUNTERED
WH WEIGHT OF HAMMER
N SPT N-VALUE
100 LOSS OF CIRCULATION (%)
NMC NATURAL MOISTURE CONTENT (%)
-200 FINES PASSING A NO. 200 SIEVE (%)

GRANULAR MATERIALS- RELATIVE DENSITY	SPT (BLOWS/FT)
VERY LOOSE	LESS THAN 4
LOOSE	5-10
MEDIUM	11-30
DENSE	31-50
VERY DENSE	GREATER THAN 50

SILTS AND CLAYS CONSISTENCY	SPT (BLOWS/FT)
VERY SOFT	LESS THAN 2
SOFT	3-4
FIRM	5-8
STIFF	9-15
VERY STIFF	16-30
HARD	30-50
VERY HARD	GREATER THAN 50

DATE	NAME	REVISION	APPROVED BY:

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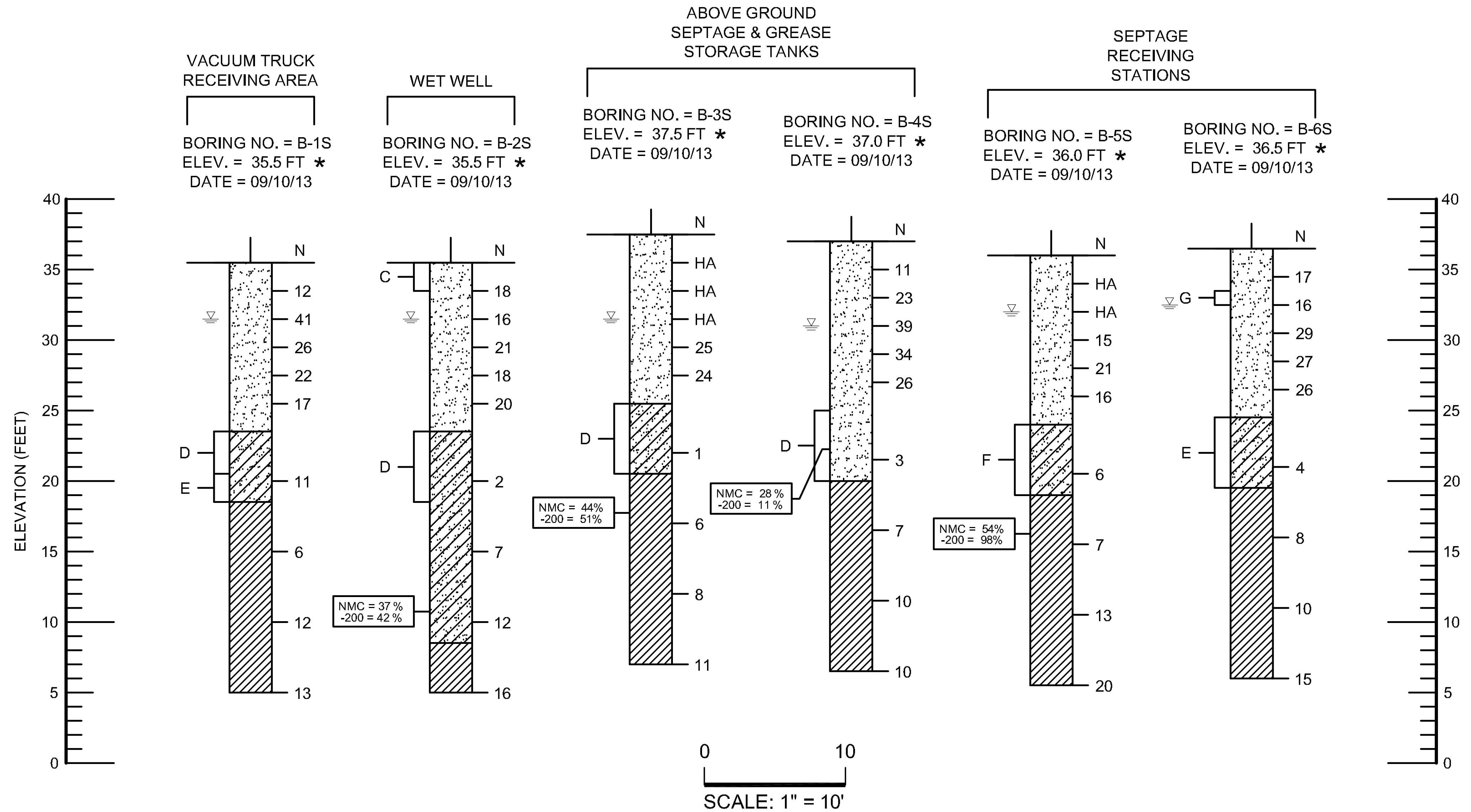
FLORIDA LICENSE No. 45603

	NAME	DATE
DESIGNED BY:	IR	09/13
DRAWN BY:	IR	09/13
CHECKED BY:	KS	09/13
SUPERVISED BY:		

REPORT OF CORE BORINGS

Southeast Water Reclamation Facility
Septage Station
Manatee County, Florida

PROJECT NO.	SHEET NO.
T121211.252	2



LEGEND

(SP/SP-SM/SP-SC) BROWN, LIGHT BROWN, TO DARK BROWN FINE SAND, SLIGHTLY SILTY FINE SAND, TO SLIGHTLY CLAYEY FINE SAND.

(SC) PALE BROWN TO GRAY CLAYEY FINE SAND.

SHELL BASE MATERIAL.

(CL/CH) BROWN TO GRAY SANDY CLAY TO CLAY.

A WITH ROOT FRAGMENTS
B WITH CLAY CLODS
C WITH TRACES TO SOME SHELL
D WITH TRACES TO SOME PHOSPHATE
E WITH CEMENTED CLAY FRAGMENTS
F WITH LARGE PHOSPHATE FRAGEMENTS
G SLIGHTLY ORGANIC

* ELEVATIONS WERE OBTAINED FROM PLANS PROVIDED BY CARDNO TBE AND ARE APPROXIMATE.

NOTES:

⊽	WATER TABLE
HA	HAND AUGER
GNE	GROUNDWATER NOT ENCOUNTERED
WH	WEIGHT OF HAMMER
N	SPT N-VALUE
100 →	LOSS OF CIRCULATION (%)
NMC	NATURAL MOISTURE CONTENT (%)
-200	FINES PASSING A NO. 200 SIEVE (%)

GRANULAR MATERIALS- RELATIVE DENSITY	SPT (BLOWS/FT)
VERY LOOSE	LESS THAN 4
LOOSE	5-10
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VERY DENSE	GREATER THAN 50

SILTS AND CLAYS CONSISTENCY	SPT (BLOWS/FT)
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SOFT	3-4
FIRM	5-8
STIFF	9-15
VERY STIFF	16-30
HARD	30-50
VERY HARD	GREATER THAN 50

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	NAME	DATE
DESIGNED BY:	IR	09/13
DRAWN BY:	IR	09/13
CHECKED BY:	KS	09/13
SUPERVISED BY:		

REPORT OF CORE BORINGS

Southeast Water Reclamation Facility
Septage Station
Manatee County, Florida

PROJECT NO.	SHEET NO.
T121211.252	3

APPENDIX B

Test Procedures

TEST PROCEDURES

The general field procedures employed by MC Squared, Inc. (**MC²)** are summarized in the American Society for Testing and Materials (ASTM) Standard D420 which is entitled "Investigating and Sampling Soil and Rock". This recommended practice lists recognized methods for determining soil and rock distribution and groundwater conditions. These methods include geophysical and in-situ methods as well as borings.

Standard Drilling Techniques
To obtain subsurface samples, borings are drilled using one of several alternate techniques depending upon the subsurface conditions. Some of these techniques are:

In Soils:

a) Continuous hollow stem augers.
b) Rotary borings using roller cone bits or drag bits, and water or drilling mud to flush the hole.
c) "Hand" augers.

In Rock:

a) Core drilling with diamond-faced, double or triple tube core barrels.
b) Core boring with roller cone bits.

The drilling method used during this exploration is presented in the following paragraph.

<u>Hollow Stem Augering:</u> A hollow stem augers consists of a hollow steel tube with a continuous exterior spiral flange termed a flight. The auger is turned into the ground, returning the cuttings along the flights. The hollow center permits a variety of sampling and testing tools to be used without removing the auger.

<u>Core Drilling</u>: Soil drilling methods are not normally capable of penetrating through hard cemented soil, weathered rock, coarse gravel or boulders, thin rock seams, or the upper surface of sound, continuous rock. Material which cannot be penetrated by auger or rotary soil-drilling methods at a reasonable rate is designated as “refusal material”. Core drilling procedures are required to penetrate and sample refusal materials.

Prior to coring, casing may be set in the drilled hole through the overburden soils, to keep the hole from caving and to prevent excessive water loss. The refusal materials are then cored according to ASTM D-2113 using a diamond-studded bit fastened to the end of a hollow, double or triple tube core barrel. This device is rotated at high speeds, and the cuttings are brought to the surface by circulating water. Core samples of the material penetrated are protected and retained in the swivel-mounted inner tube. Upon completion of each drill run, the core barrel is brought to the surface, the core recovery is measured, and the core is placed, in sequence, in boxes for storage and transported to our laboratory.

Sampling and Testing in Boreholes
Several techniques are used to obtain samples and data in soils in the field; however the most common methods in this area are:

a) Standard Penetrating Testing
b) Undisturbed Sampling
c) Dynamic Cone Penetrometer Testing
d) Water Level Readings

The procedures utilized for this project are presented below.

<u>Standard Penetration Testing:</u> At regular intervals, the drilling tools are removed and soil samples obtained with a standard 2 inch diameter split tube sampler connected to an A or N-size rod. The sampler is first seated 6 inches to penetrate any loose cuttings, and then driven an additional 12 inches with blows of a 140 pound safety hammer falling 30 inches. Generally, the number of hammer blows required to drive the sampler the final 12 inches is designated the "penetration resistance" or "N" value, in blows per foot (bpf). The split barrel sampler is designed to retain the soil penetrated, so that it may be returned to the surface for observation. Representative portions of the soil samples obtained from each split barrel sample are placed in jars, sealed and transported to our laboratory.

The standard penetration test, when properly evaluated, provides an indication of the soil strength and compressibility. The tests are conducted according to ASTM Standard D1586. The depths and N-values of standard penetration tests are shown on the Boring Logs. Split barrel samples are suitable for visual observation and classification tests but are not sufficiently intact for quantitative laboratory testing.

<u>Water Level Readings:</u> Water level readings are normally taken in the borings and are recorded on the Boring Records. In sandy soils, these readings indicate the approximate location of the hydrostatic water level at the time of our field exploration. In clayey soils, the rate of water seepage into the borings is low and it is generally not possible to establish the location of the hydrostatic water level through short-term water level readings. Also, fluctuation in the water level should be expected with variations in precipitation, surface run-off, evaporation, and other factors. For long-term monitoring of water levels, it is necessary to install piezometers.

The water levels reported on the Boring Logs are determined by field crews immediately after the drilling tools are removed, and several hours after the borings are completed, if possible. The time lag is intended to permit stabilization of the groundwater level that may have been disrupted by the drilling operation.

Occasionally the borings will cave-in, preventing water level readings from being obtained or trapping drilling water above the cave-in zone.

BORING LOGS

The subsurface conditions encountered during drilling are reported on a field boring log prepared by the Driller. The log contains information concerning the boring method, samples attempted and recovered, indications of the presence of coarse gravel, cobbles, etc., and observations of groundwater. It also contains the driller's interpretation of the soil conditions between samples. Therefore, these boring records contain both factual and interpretive information. The field boring records are kept on file in our office.

After the drilling is completed a geotechnical professional classifies the soil samples and prepares the final Boring Logs, which are the basis for our evaluations and recommendations.

SOIL CLASSIFICATION

Soil classifications provide a general guide to the engineering properties of various soil types and enable the engineer to apply his past experience to current problems. In our investigations, samples obtained during drilling operations are examined in our laboratory and visually classified by an engineer. The soils are classified according to consistency (based on number of blows from standard penetration tests), color and texture. These classification descriptions are included on our Boring Logs.

The classification system discussed above is primarily qualitative and for detailed soil classification two laboratory tests are necessary; grain size tests and plasticity tests. Using these test results the soil can be classified according to the AASHTO or Unified Classification Systems (ASTM D-2487). Each of these classification systems and the in-place physical soil properties provides an index for estimating the soil's behavior. The soil classification and physical properties are presented in this report.

The following table presents criteria that are typically utilized in the classification and description of soil and rock samples for preparation of the Boring Logs.

Relative Density of Cohesionless Soils From Standard Penetration Test	
Very Loose	≤ 4 bpf
Loose	5 - 10 bpf
Medium Dense	11 - 30 bpf
Dense	31 - 50 bpf
Very Dense	> 50 bpf

(bpf = blows per foot, ASTM D 1586)

Consistency of Cohesive Soils	
Very Soft	≤ 2 bpf
Soft	3 - 4 bpf
Firm	5 - 8 bpf
Stiff	9 - 15 bpf
Very Stiff	16 - 30 bpf
Hard	30 – 50 bpf
Very Hard	> 50 bpf

Relative Hardness of Rock	
Very Soft	Hard Rock disintegrates or easily compresses to touch; can be hard to very hard soil.
Soft	May be broken with fingers.
Moderately Soft	May be scratched with a nail, corners and edges may be broken with fingers.
Moderately Hard	Light blow of hammer required to break samples.
Hard	Hard blow of hammer required to break sample.

Particle Size Identification	
Boulders	Larger than 12"
Cobbles	3" - 12"
Gravel	
Coarse	3/4" - 3"
Fine	4.76mm - 3/4"
Sand	
Coarse	2.0 - 4.76 mm
Medium	0.42 - 2.00 mm
Fine	0.42 - 0.074 mm
Fines (Silt or Clay)	Smaller than 0.074 mm

Rock Continuity

$$\textbf{RECOVERY} = \frac{\text{Total Length of Core}}{\text{Length of Core Run}} \times 100\ \%$$

Description	Core Recovery %
Incompetent	Less than 40
Competent	40 - 70
Fairly Continuous	71 - 90
Continuous	91 - 100

Relative Quality of Rocks

$$\textbf{RQD} = \frac{\text{Total core, counting only pieces > 4" long}}{\text{Length of Core Run}} \times 100\ \%$$

Description	RQD %
Very Poor	0 - 25 %
Poor	25 - 50 %
Fair	50 - 75 %
Good	75 - 90 %
Excellent	90 - 100 %