Geotechnical Investigation Report

Alline Avenue
Stormwater Pumping Station
City of Tampa, Florida

Prepared for: **McKim & Creed**
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Prepared By:
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Project No. T050704.084
July 2010

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GEOTECHNICAL • ENVIRONMENTAL
MATERIALS TESTING

MC^2 GEOTECHNICAL • ENVIRONMENTAL MATERIALS TESTING

July 26, 2010

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Geotechnical Engineering Services Report
Alline Avenue Stormwater Pumping Station
City of Tampa, Florida
MC^2 Inc. Project No. T050704.084

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

Often, because of design and construction details that occur on a project, questions arise concerning subsurface conditions. **MC^2** will be pleased to continue our role as geotechnical consultants during the construction phase of this project to provide assistance with construction materials testing and inspection services and to verify that our recommendations are implemented.

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

Respectfully submitted,
MC^2

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EXECUTIVE SUMMARY

MC^2 has completed its geotechnical engineering study for the proposed stormwater pump station located at the 2921 West Alline Avenue in Tampa, Florida. Project information has been provided by Mr. Mathew S. Love, PE and Mr. Bill Band, PE (Structural Engineer) of **McKim & Creed**. The project also includes proposed stormwater piping and inlets along a proposed easement between W. Bayshore Court and Asbury Place and a forcemain along Asbury Place from the Alline Avenue Pump Station to Bayshore Boulevard and eventually into Tampa Bay.

The forcemain will be 36-inches in diameter, 1250 feet in length; 850 feet of DIP installed by open cut and 400 feet installed using a trenchless construction technique such as horizontal directional drilling (HDD). The pipes included in this project consist of 24" RCP and 29" x 45" ERCP. The depth to the bottom of the footing for the pump station foundation is 16.7 feet below grade. Temporary shoring is also anticipated.

We are assuming that the bottom slab of the wet well will be poured monolithically and tied in with the lower portion of the walls.

The recommendations provided in this report are based on this information. 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.

The information that is provided in this section is a brief summary. This report <u>must be read in its entirety prior to its use</u> in the design and further development of the subject project.

In general, the borings encountered very loose to loose fine sands to slightly silty fine sands to slightly clayey fine sand (SP/SP-SM/SP-SC) and <u>organic silty fine sand (SM)</u> to depths ranging from 4 to 6 feet. The organic silty sands were encountered at depths ranging from 2 to 2-1/2 feet and have organic contents ranging from 5 to 8 percent. These sands occasionally contained traces of rock and shell fragments. Below these sands, the borings indicated very loose to loose clayey sands (SC) or firm to very stiff sandy clay to clay (CH) to depths near 12 feet. Boring SPT-4 encountered a layer of very stiff calcareous clay with weathered limestone (CL) from depths of 12 to 17 feet. Both borings indicated hard weathered limestone with calcareous clay (LS) at depths ranging from 12 to 17 feet and extending to the boring termination depth of 30 feet.

In general, the borings performed for the forcemain and RCP (AB-1 to AB-4 and SPT-1 to SPT-3) encountered very loose to loose fine sands to slightly silty fine sands to slightly clayey fine sands (SP/SP-SM/SP-SC) and silty fine sand (SM) to depths ranging from 8 to 12 feet. These sands occasionally contained traces of rock and shell fragments. Below the cleaner sands, the borings indicated very loose to loose clayey sands (SC) or firm to very stiff sandy clay to clay (CH) to depths ranging from 17 to 22 feet. Boring SPT-2 was terminated in stiff calcareous clay with weathered limestone

(CL) from depths of 17 to 20 feet and boring SPT-3 was terminated in hard weathered limestone with calcareous clay (LS) from 22 to 30 feet.

Based on the anticipated construction, and after proper subgrade preparation (including overexcavating vertically and horizontally any unsuitable materials, such as organic soils, roots and debris if encountered during construction), the proposed pump station wet well structures, building structure (generator room, electrical room, dock and porches) and outfall box structure can be designed for a net maximum allowable bearing capacity of 2,000 psf using mat foundations and strip footings, respectively.

Groundwater will have a significant effect on construction of the proposed pump station. The groundwater level, during drilling, at the location of the Standard Penetration Test (SPT) and hand auger borings ranged from 1.5 to 4.0 feet. The Soil Survey of Hillsborough County indicates that the site is in Immokalee-Urban Land complex (mapping unit 22) with seasonal high depths of 6 to 18 inches. We estimate the seasonal high groundwater level to be at a depth of about 0.5 to 1.0 feet below the existing surface.

Dewatering will be required and the pump station and outfall structure design should take into account the effect of buoyancy. The buoyancy analysis should include determination of additional methods of restraint, such as increased bottom slab thickness or slab extension, if necessary. Temporary shoring will also be required and designed by others using the soil parameters provided in this report.

It is recommended that **MC²** be retained to provide observation and testing of construction activities involved in the foundation, earthwork, and related activities of this project to ensure that the recommendations contained herein are properly interpreted and implemented. If **MC²** is not retained to perform these functions, we cannot be responsible for the impact those conditions might have on the performance of the project.

Summary of Recommendations
Alline Avenue Stormwater Pumping Station
City of Tampa, Florida

Criteria	Recommendations
Coefficient of at-rest earth pressure, K_o	0.5
Coefficient of active earth pressure, K_a	0.3
Coefficient of passive earth pressure, K_p	3.0
Friction factor of concrete over soil	0.4
Friction factor of concrete (slab) over crushed stone	1.3
Friction factor of slab-on-grade with polyethylene vapor barrier over soil	0.4
Potential expansive, deleterious, chemically reactive or corrosive materials or presence of gas	None encountered. No odors noted during drilling
Swelling potential of on-site soils	Low
Sinkhole Activity	None Identified; however not within the scope
Soil liquefaction potential	Low
Special design considerations due to unusual geological features	None
Effect of weather or equipment on soil during construction	Positive site drainage and maintenance. Keep traffic off wet soils.
Recommendations for groundwater management	Positive site drainage.
Allowable soil bearing pressures	2,000 psf or less for wet well and building
Anticipated total and differential settlement	1 inch total and ½ inch differential for slabs
Alternate foundation recommendations	None
Effect of foundation construction on adjacent property	None known
Concrete slab-on-grade subgrade modulus	Approximately 100 pci
Subgrade recommendations for concrete slab-on-grade support, wet well construction	12 inches compacted to 98% modified Proctor, with 36 inches of compacted No. 57 stone wrapped in geo/filter fabric on top of the approved subgrade soils
Pavement design for standard and heavy duty traffic	Not Applicable

GEOTECHNICAL ENGINEERING SERVICES REPORT

1.0 INTRODUCTION

1.1 Authorization

This report presents the findings of the subsurface exploration and associated recommendations based on a geotechnical engineering evaluation of the site for the proposed Alline Avenue Pump Station in Tampa, Florida. The services for this project were performed in general accordance with our revised proposal No. T050704.084 dated January 23, 2008. Authorization to perform the exploration and evaluation was in the form of acceptance of our proposal by Mr. Mathew S. Love, PE Project Manager with **McKim & Creed.**

2.0 PROJECT INFORMATION

This geotechnical report is based on information supplied to us by Mr. Matthew S. Love, PE and Mr. Bill Band, PE (Structural Engineer) with **McKim & Creed**. The borings were located in the field by representatives of **MC^2** using existing features as references and considering utility constraints.

The recommendations provided in this report are based on this information. 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.

2.1 Site Location

The proposed site evaluated and reported herein is located on West Alline Avenue, south of Asbury Place in Tampa, Florida. A **Boring Location Plan** is included as **Sheet 1** in **Appendix A.**

2.2 Project Description

Project information has been provided by Mr. Mathew S. Love, PE and Mr. Bill Band, PE (Structural Engineer) of **McKim & Creed**. The project consists of a new stormwater pumping station and a building at 2921 West Alline Avenue in Tampa, Florida. An existing house at 2921 West Alline Avenue had been demolished and the site was cleared and covered with grass.

The project also includes proposed stormwater piping and inlets along a proposed easement between W. Bayshore Court and Asbury Place and a forcemain along Asbury Place from the Alline Avenue Pump Station to Bayshore Boulevard and eventually into Tampa Bay.

The forcemain will be 36-inches in diameter, 1250 feet in length, of which about 400 feet (from the intersection of Richards Court and Asbury Place to Bayshore Blvd.) will be installed using a trenchless construction technique such as horizontal directional drilling (HDD). The pipes along the easement consist of 24" RCP and 29" x 45" ERCP with inlets.

We understand that the proposed RCP pipe will be approximately 4 to 6 feet deep, and the portion of the forcemain not installed using the HDD trenchless technology along Asbury Place will be embedded 10 to 12 feet below the existing or finished grade. From that intersection to the median in Bayshore Blvd. the installation will be using HDD and embedment depths of 20 to 30 feet below the existing ground surface or finished grade. At Bayshore Blvd. the embedment depth will be 5 to 10 feet.

Preliminary conceptual drawings and information were provided by Mr. Bill Band, PE, the project structural engineer with **McKim & Creed**. Two new structures are being proposed at the 2921 West Alline Avenue site. The proposed structure located on the north side of the site consisted of wet well and valve vault and the second structure located south consisted of a generator room, electrical room, truck loading docks and porches. The existing ground surface elevation is approximately +3.0 feet. The following preliminary information was provided;

- <u>Bar Screening, Wet Well and Valve Vault Building</u>
    1. Approximate dimensions = 38' x 34'
    2. Approximate Load = 1090 kips
    3. Approximate pressure = 844 psf
    4. Top slab of wet well = at grade
    5. Bottom of Wet well and Valve Vault = depth of 16.7 feet below grade or elevation -13.5 feet.

- <u>Generator Room, Electrical Room Building with Truck Loading Dock and Porches</u>
    1. Approximate dimensions = 44' x 37'
    2. Approximate load = 1230 kips
    3. Approximate pressure= 756 psf
    4. 2-1/2 stories tall above the existing ground surface
    5. Bottom of footings at embedment depth = 2.0 feet below grade or elev. +1.0 feet
    6. Top of Main Floor elevation = +11.5 feet
    7. Top of Generator slab elevation = +10.0 feet
    8. Loading Dock elevation = +6.0 feet

- <u>Outfall Box Structure</u>
    1. Approximate dimensions = 30' x 15'
    2. Approximate Load = 1000 kips
    3. Approximate pressure = 2,222 psf
    4. Bottom of foundation = depth of 12.0 feet below grade

We are assuming that the bottom slab of the wet well will be poured monolithically and tied in with the lower portion of the walls.

2.3 Purpose and Scope of Services

Two (2) Standard Penetration Test (SPT) borings (SPT-4 and SPT-5) were performed near the proposed pump station location and three (3) SPT borings (SPT-1 through SPT-3) were performed along the proposed forcemain to develop the recommendations presented in this report. The boring locations were selected by **McKim & Creed** and located in the field by representatives from **MC^2**.

The purpose of this exploration was to evaluate subsurface conditions at the site and to provide recommendations regarding design and general site development for the proposed pump station and forcemain construction.

Our geotechnical study and analyses consisted of a review of available subsurface test data. Sources include the USDA Hillsborough County Soil Survey, USGS Maps and previous geotechnical engineering studies performed by **MC^2** in this area. The testing program consisted of the following services:

- Conducted a visual reconnaissance of each project site. The actual location of the proposed structures was provided by **McKim & Creed** personnel. However, the final boring location was positioned considering access and utility constraints. We determined the boring locations by taping distances from boundaries and existing features; therefore, the boring locations are approximate.

- Reviewed the USDA Soil Survey for Hillsborough County and the USGS topographic maps.

- Drilled two (2) Standard Penetration Test (SPT) borings to depths of 20 feet (at the median of Bayshore Blvd.) below the existing grade and one (1) to a depth of 30 feet (at the intersection of Asbury and Richards) below the existing grade at the site to provide site-specific deeper design information for the proposed forcemain. The borings were labeled SPT-1 to SPT-3. The locations of the borings are shown on the **Boring Location Plan** (**Sheet 1**) located in **Appendix A** of this report.

- Drilled two (2) SPT borings for the proposed pump station building to a depth of 30 feet below the existing ground surface. The borings were labeled SPT-4 and SPT-5. The boring locations are also shown in **Sheet 1** in **Appendix A**.

- Performed three (3) hand auger borings to depths of 4 to 6 feet along the proposed pipeline route within the easement between W. Bayshore Court and

Asbury Place and one (1) to a depth of 6 feet (south side of Asbury Place). The boring locations are shown on **Sheet 1 i**n **Appendix A.**

AB-1 - approximate location is at 3011 Asbury Place
AB-2 - approximate location is between 2922 and 2924 Alline Avenue
AB-3 – approximate location is between 2921 and 2923 Coachman Avenue
AB-4 – approximate location is between 2929 and 2931 Bayshore Court

- Visually examined all recovered soil samples for the project using the Unified Soil Classification Systems (USCS). Performed a limited laboratory testing program to assist with the classification, including percent passing the No. 200 sieve, natural moisture, Atterberg limits testing and organic content.

- Performed soil corrosion parameters tests (sulfate, chloride, pH, conductivity and redox potential) on randomly selected soil samples along the proposed forcemain.

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

- General assessment of area geology based on our past experience, study of geological literature and boring information for the site.

- General suitability of materials within the site for use as engineered fill and general backfill.

- General location and description of potentially deleterious materials encountered in the borings, which may interfere with the pump station's construction or performance, including existing fill or surficial organics.

- Discuss critical design and/or construction considerations based on the soil and groundwater conditions encountered in from the borings.

- Address the groundwater level in the boring and estimate seasonal high groundwater. Provide recommendations for de-watering, if required.

- Recommendations for design and construction may include allowable bearing pressures for foundation design, excavation conditions, dewatering and uplift resistance, structural fill, earthwork recommendations and lateral earth pressures on below grade walls for the site and pipe bedding.

The Report of Core Borings for the site with the soil profiles is included in **Sheet 2** in **Appendix A** of this report.

The geotechnical scope of services did not include an environmental assessment for determining the presence or absence of wetlands or hazardous or toxic materials in the soil, bedrock, surface water, groundwater, or air, on or below or around this site. Any statements in this report or on the boring logs regarding odors, colors, unusual or suspicious items or conditions are strictly for the information of the client.

3.0 LABORATORY TESTING

3.1 Soil Classification Testing

Representative soil samples collected from the auger and SPT borings were visually reviewed in the laboratory by a geotechnical engineer to confirm the field classifications. The samples were classified and stratified in general accordance with the Unified Soil Classification System. Classification was based on visual observations with the results of the laboratory testing used to confirm the visual classification. Laboratory classification tests consisting of percent passing the No. 200 sieve, moisture content, Atterberg limits and organic content determinations were performed on selected soil samples believed to be representative of the materials encountered. A summary of the test results is provided in **Table 1** in **Appendix A**.

3.2 Percent passing the No. 200 Sieve

The wash gradation test measures the percentage of a dry soil sample passing the No. 200 sieve. By definition in the Unified Soil Classification System, the percentage by weight passing the No. 200 sieve is the silt and clay content. The amount of silt and clay in a soil influences it properties, including permeability, workability and suitability as fill. This test was performed in general accordance with ASTM D-1140 (Standard Test Methods for Amount of Material Finer Than the No. 200 (75 μm) Sieve).

3.3 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 FM 1-T265 (ASTM test designation D-2216). Briefly, natural moisture content is determined by weighing a sample of the selected material and then drying it in a warm oven. Care is taken to use a gentle heat so as not to destroy any organics. The sample is removed from the oven and reweighed. The difference of the two weights is the amount of moisture removed from the sample. The weight of the moisture divided by the weight of the dry soil sample is the percentage by weight of the moisture in the sample.

3.4 Atterberg Limits

The liquid limit and the plastic limit tests ("Atterberg Limits") were conducted in general accordance with the FDOT test designation FM 1-T089 and FM 1-T090, respectively (ASTM test designation D-4318). Atterberg plastic limit and liquid limit tests measure the moisture content at which a fine-grained soil changes from a semi-solid to plastic state and from a plastic to a liquid state, respectively. The plasticity index is the difference between the liquid and plastic limits. The plasticity index is a rough indication of the tendency of a soil to absorb water on the particle surfaces. Some clays have a strong affinity for water, and tend to swell when wetted and shrink when dried. The larger the plastic index, the greater the shrink-swell tendency.

3.5 Organic Content

The laboratory organic content test consists of drying the soil sample, then heating it in a small furnace to a minimum temperature of 400 degrees Centigrade for 6 hours. The high heat burns off all organic material, leaving only the soil minerals. The difference in the weight prior to and after the burning is the weight of the organics. The weight of the organics divided by the weight of the before burn dried soil is the percentage of the organics within a sample. The organic content testing procedure were conducted in general accordance with the FDOT test designation FM1-T267 (ASTM 2974 (Standard Test Methods for Moisture, Ash, and Organic Matter of Peat and Other Organic Soils)).

3.6 Environmental Classification (Corrosion Test)

Environmental corrosion tests were conducted in accordance with the FDOT test designations FM 5-550, FM 5-551, FM 5-552 and FM 5-553. These tests were performed on recovered soil samples obtained from the forcemain borings. Environmental corrosion tests measure parameters such as pH, resistivity, sulfate content, chloride content and redox potential. Test results obtained are presented in **Table 2** in **Appendix A.** Based on the laboratory test results and the FDOT Structures Design Guidelines, the environment of the pipeline alignment sub soils is classified as slightly and moderately aggressive for steel and concrete. We recommend using the FDOT Structures Design Guidelines and FDOT Standard Specifications for corrosion protection measures.

4.0 GENERAL SITE AND SUBSURFACE CONDITIONS

4.1 Regional Geology

A review of Florida Geological Survey, Report of Investigations No. 25, dated 1961 prepared by the United States Geological Survey (USGS) revealed that Hillsborough County is in the Floridian section of the Atlantic Coastal Plain. The notable physiographic features of the area are related to ancient seas, which once covered the region. Relict shorelines are evidenced by subtle linear escarpments, which have not been significantly altered by fluvial (river) processes in much of the area. Four ancient shorelines are preserved in Hillsborough County. The Pamlico, Talbot, Penholoway, and Wicomico shorelines stand at or near 25, 42, 70 and 100 feet above present mean sea level (MSL), respectively.

C. Wythe Cooke included the western and southern parts of the County in the Coastal Lowlands and the eastern part in the Central Highlands. The Coastal Lowlands are low, nearly level plains that lie next to the coast. The Central Highlands are the gently undulating to rolling areas in the eastern part of the County.

In the southwestern part of the County, Tampa Bay extends for a considerable distance inland. Its northern section is separated into Old Tampa Bay and Hillsborough Bay by a peninsula that extends southward from Tampa.

Large, nearly level plains, commonly called flatwoods, are in the western, southern, and northeastern parts of the County. These plains rise gradually from the coast to elevations of more than 100 feet in the eastern part of the County. Numerous intermittent ponds, swamps, and marshes and a few permanent lakes are in the flatwood areas. Many permanent lakes and intermittent ponds are in the northwestern and north-central parts of the County. Some of the larger lakes are Lake Thonotosassa, Lake Calrico, Mango Lake, Keystone Lake, and Lake Magdalene. Along the coast, elevations in the County range from sea level to about 144 feet at a point about 3.4 miles east of Plant City. Tampa is at an elevation of about 19 feet.

The surface drainage is toward Old Tampa Bay, Hillsborough Bay, and Tampa Bay. The principal streams are the Hillsborough, Alafia, and Little Manatee Rivers and Rocky, Sweetwater, Sixmile, and Bullfrog Creeks. Many ditches, canals and small bays extend inland from the coast for short distances.

4.2 Soil Survey of Hillsborough County

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 Hillsborough County. This information is available through the NRCS Web Soil Survey. The soil information of Hillsborough County, Florida was issued in May 1989 with updated seasonal high water levels published in June 2006. The aerial photographs used in the mapping were compiled in 2007. The soil survey indicates the extent of the improvements at the site is primarily located within an area mapped as Immokalee-Urban land complex (mapping unit 22). A soil mapping unit is an area dominated by a particular soil type.

This mapping unit consists of 45 to 60 percent Immokalee soil and 35 to 45 percent Urban land. The individual areas of Immokalee fine sand and Urban land are too small to map separately at the scale used in the mapping. Most areas are artificially drained.

Immokalee fine sand is characterized by 5 inches of black fine sand at the surface, which is followed by grayish brown fine sand. From about 13 to 35 inches, light gray fine sand is present. A seam of black fine sand then occurs to a depth of about 40 inches, overlying a layer of dark reddish brown fine sand that extends to 47 inches deep. Dark brown fine sand is then indicated to a depth of 60 inches, and light brownish gray fine sand follows below to a depth in excess of 80 inches. The depth to the seasonal high groundwater level ranges from 6 to 18 inches.

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.

4.3 Subsurface Exploration

Subsurface conditions at the proposed pump station location was obtained by drilling two (2) Standard Penetration Test (SPT) borings at the proposed pump site to a depth of 30 feet, one (1) SPT boring along the proposed forcemain to a depth of 30 feet and two (2) SPT borings to a depth of 20 feet in the vicinity of the storm main outfall box. The approximate boring locations are shown on the **Boring Location Plan (Sheet 1)** presented in **Appendix A.**

The SPT borings were conducted in general accordance with ASTM D-1586 (Standard Test Method for Penetration Test and Split Barrel Sampling of Soils) using the rotary wash method, where a clay slurry ("drill mud" or "drill fluid") was used to flush and stabilize the borehole. Standard Penetration sampling was performed at closely spaced intervals in the upper 10 feet and at 5-foot intervals thereafter. After seating the sampler 6 inches into the bottom of the borehole, the number of blows required to drive the sampler one foot further with a standard 140 pound hammer is known as the "N" value or blowcount. The blowcount has been empirically correlated to soil properties. The recovered samples were placed into containers and returned to our office for visual review.

A total of approximately four (4) hand auger borings were performed to a depth of about 4 to 6 feet below the existing grades along the proposed forcemain and the reinforced concrete pipe. The depths of the borings were limited as a result of caving of the borehole due to the groundwater. 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 **MC²** Tampa office for review by a geotechnical engineer and confirmation of the field classification.

4.4 Subsurface Conditions

The SPT soil samples were classified using the Unified Soil Classification System (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.

The following description is of a generalized nature, provided to highlight the major subsurface strata encountered in the borings performed at the site. The **Report of Core Borings** on **Sheet 2** in **Appendix A** should be reviewed for specific soil and groundwater information at the boring locations. The stratifications shown on the boring logs represent the conditions only at the actual boring location. Variations may occur and should be expected across the site. The stratifications represent the approximate boundary between subsurface materials and the transition may be gradual.

- **36” Diameter Storm Main and Outfall Box (borings SPT-1 to SPT-3 and AB-1)**

  In general, the borings encountered very loose to loose fine sands to slightly silty fine sands to slightly clayey fine sand (SP/SP-SM/SP-SC) and silty fine sand (SM) to depths ranging from 8 to 12 feet. These sands occasionally contained traces of rock and shell fragments. Below the cleaner sands, the borings indicated very loose to loose clayey sands (SC) or firm to very stiff sandy clay to clay (CH) to depths ranging from 17 to 22 feet. Boring SPT-2 was terminated in stiff calcareous clay with weathered limestone (CL) from depths of 17 to 20 feet and boring SPT-3 was terminated in hard weathered limestone with calcareous clay (LS) from 22 to 30 feet.

- **Proposed Wet Well and Valve Vault and Generator Room, Electrical Control Room, Truck dock and Porches Buildings (borings SPT-4 and SPT-5)**

  In general, the borings encountered very loose to loose fine sands to slightly silty fine sands to slightly clayey fine sands (SP/SP-SM/SP-SC) and <u>organic silty fine sand (SM)</u> to depths ranging from 4 to 6 feet. The organic silty sands were encountered at depths ranging from 2 to 2-1/2 feet and have organic contents ranging from 5 to 8 percent. These sands occasionally contained traces of rock and shell fragments. Below the these sands, the borings indicated very loose to loose clayey sands (SC) or firm to very stiff sandy clay to clay (CH) to depths of 12 feet. Boring SPT-4 encountered a layer of very stiff calcareous clay with weathered limestone (CL) from depths of 12 to 17 feet. Both borings indicated hard weathered limestone with calcareous clay (LS) from 12 to 17 feet and extends to the boring termination depth of 30 feet.

- **Proposed Reinforced Concrete Pipe (RCP) (borings AB-2 to AB-4)**

  In general, the borings encountered fine sands to slightly silty fine sands to slightly clayey fine sands (SP/SP-SM/SP-SC) and silty fine sand (SM) to depths ranging from 4 to 6 feet.

4.5 Groundwater Information (Wet Well and Valve Vault and Generator Room, Electrical Control Room buildings, Outfall Box structure and Piping)

The depth below the existing ground surface to the groundwater level along the proposed stormwater piping (boring AB-2, AB-3 and AB-4) and proposed forcemain (AB-1, SPT-1, SPT-2 and SPT-3) ranged from 1.5 to 4.0 feet, during a relative dry period (February 22 and 23, 2010). The groundwater level at the proposed pump site (borings SPT-4 and SPT-5) was encountered at a depth of 3.5 feet below the existing ground surface and was also measured during a dry period.

The water table can be expected to vary at times and will fluctuate seasonally based on rainfall quantities, area geology, surface drainage conditions and other factors. The Soil Survey of Hillsborough County indicates that the site is in Immokalee-Urban Land complex (mapping unit 22). About 35 to 45 percent is Urban land or covered with buildings and pavements and contains soils altered by development so that their identification is not feasible and seasonal high water tables are not provided. However, we estimate the seasonal high groundwater level to be at a depth of about 0.5 to 1.0 feet below the existing surface.

Dewatering will be required and the pump station and outfall box structures design should take into account the effect of buoyancy. <u>The buoyancy analysis should include determination of additional methods of restraint, such as increased bottom slab thickness or slab extension, if necessary.</u>

5.0 EVALUATION AND RECOMMENDATIONS

The following design recommendations have been developed on the basis of the previously described project characteristics and subsurface conditions encountered. If there are any changes in these project criteria, including project location on the site, a review must be made by **MC²** to determine if any modifications in the recommendations will be required. The findings of such a review should be presented in a supplemental report.

Once final design plans and specifications are available, a general review by **MC²** is strongly recommended as a means to check that the evaluations made in preparation of this report are correct and that earthwork and foundation recommendations are properly interpreted and implemented.

5.1 Building, Pump Station and Outfall Box Structure

5.1.1 General Site Development Considerations

We understand that about 3 to 4 feet will be excavated for the building, 17 feet for the wet well and influent pipes and 12 feet for the outfall structure of soil will be excavated to. Based on the findings of our test borings, our understanding of the proposed structures, and our geotechnical engineering evaluation, monolithically poured foundations can be used for the proposed construction. However, there are some issues that will need to be addressed during design and construction especially with regards to the somewhat high groundwater table at this location.

The following sections further discuss specific geotechnical, foundation, design, and site grading concerns at the site.

5.1.2 Site Preparation

Prior to construction, the site (building and pump station) should be stripped of any surface vegetation and any organic soils should be removed extending out at least 10 feet beyond the construction limits. A layer of organic silty sand (SM), which will cause long term settlement was encountered in the two borings at depths of about 2.0 to 2.5 feet and should be overexcavated within the footprint of both structures plus 5 feet and replaced with clean sands (SP/SP-SM/SP-SC) free of detrimental materials.

Any areas requiring at grade structures or areas requiring fill should be proofrolled with a heavily loaded dump truck if accessible, to determine areas that may need additional removal of unsuitable bearing materials. In addition to stripping the site, the location of any existing underground utility lines within the construction area should be established. Provisions should then be made to relocate any interfering utility lines within the construction area to appropriate locations. In this regard, it should be noted that if abandoned underground pipes are not properly removed or plugged, they may serve as conduits for subsurface erosion which subsequently may result in excessive settlement. Any underground utility pipes not removed and being greater than 4 inches in diameter should be filled with "flowable" fill (lean concrete grout), while the ends of utility pipes less than 4 inches in diameter should be plugged with concrete to prevent the inadvertent introduction of fluids into the construction area. All utility lines that are removed outside of the excavation limits should be backfilled with acceptable fill material. Fill placement and subgrade preparation recommendations are presented in the Construction Considerations, Fill Placement and Subgrade Preparation Section of this report.

In addition, deeper organic soils and clayey soils should be removed (if encountered) within 48 inches from the bottom of the wet well and replaced with properly compacted clean sands (SP/SP-SM/SP-SC).

5.1.3 Groundwater Considerations and Dewatering

The groundwater level, during drilling, at the location of the Standard Penetration Test (SPT) borings and at the hand auger borings ranged from a depth of 1.5 to 4.0 feet. We estimate the SHWT to be at 0.5 to 1.0 feet below the existing ground surface at the site. The contractor should determine the actual groundwater levels at the time of construction. The contract documents should indicate that dewatering design and implementation is the sole responsibility of the Contractor and should also contain the performance criteria for assessing the effectiveness of the dewatering system actually installed. Dewatering consisting of cutoff walls (temporary shoring), cased well points and/or vacuum well points or a combination thereof, should be designed and installed to lower the groundwater table at least to a depth of 3 or more feet below the bottom of the excavation. The dewatering should be maintained continuously (7 days per week/ 24 hours per day) throughout the construction period, until the backfill has reached the existing grade, and until sufficient structural weight is in place to resist uplift pressures due to the existing groundwater levels. Temporary shoring is anticipated. The soil parameters to be used by others to design the temporary shoring are included in **Table 3** in **Appendix A.**

In addition to the primary dewatering system, pumping of miscellaneous inflow of water should be performed from sumps excavated and placed outside and just below the elevation of the proposed foundation area. Placement of compacted No. 57 stone wrapped in geo/filter fabric in the bottom of the excavation, beneath a pre-cast or cast in place concrete slab, will act as a medium for rainwater and groundwater inflows which will be pumped out of the recommended sump areas.

We recommend the use of 36 inches of No. 57 Stone wrapped in geo/filter fabric be placed on the approved subgrade to support the wet well foundation concrete. The No. 57 stone should be extended 3 feet beyond the perimeter of the foundation footprint. The gravel will provide a stable working platform, will help to preserve the subgrade and will be used to facilitate dewatering of the excavation.

Depending upon shallow groundwater levels and the effectiveness of dewatering at the time of construction, seepage may enter the excavated trenches from the bottom and sides. Such seepage will act to loosen soils and create difficult working conditions. Groundwater levels should be determined immediately prior to construction.

5.1.4 Excavation Considerations

Excavation will be required to construct the pump station, outfall box and any series of pipelines associated with the project. The dewatering system should be in place and functioning prior to any excavation taking place. Piezometers installed prior to excavation should be used to verify that the dewatering system is performing adequately.

The existing soils being excavated at this site generally consist of very loose to loose fine sands (SP/SP-SM/SP-SC) in the top 4 to 8 feet followed by loose clayey sands (SC) or firm to very stiff sandy clays to clays (CH). We do not anticipate that excavation of these materials will be a problem. However, shallow very stiff to hard calcareous clay with weathered limestone (CL) and weathered limestone with calcareous clay (LS) was encountered in borings SPT-2 to SPT-5 at depths ranging from 12 to 22 feet. Excavation of this material will be difficult but can be done with heavy specialized equipment or blasting. Temporary shoring is anticipated. The soil parameters to be used by others to design the temporary shoring are included in **Table 3** in **Appendix A.**

We recommend that the bottom of the wet well or the outfall box be overexcavated approximately 36 inches and 3 feet wider than the perimeter of the foundation and replaced with compacted No. 57 stone, wrapped in geo/filter fabric.

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 excavation appears suitable as load bearing materials, the soils should be prepared for construction by compaction to a dry density of at least 98% of the modified Proctor maximum dry density (ASTM D-1557) for a depth of at least 1 foot below the compacted No. 57 stone wrapped in geo/filter fabric, which will serve as a 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% of the modified Proctor maximum dry density (ASTM D-1557), as previously described. Gravel, or crushed #57 or #67 stone, if used, should be compacted and the compaction confirmed by visual observation.

It is possible that the proposed construction will consist of both open sloped excavation and the installation of bracing and/or sheet walls. Our scope of services did not include analysis of slope stability or sheet piling; however, for soils of the type present at the site we recommend that all excavations be sloped no steeper than 2H:1V. Please refer to the Federal Temporary Excavation Regulations reported below.

5.1.5 Federal Temporary Excavation Regulations

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

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

We are providing this information solely as a service to our client. **MC²** is not assuming responsibility for construction site safety or the contractor’s activities; such responsibility is not being implied and should not be inferred.

5.1.6 Uplift Resistance for Pump Station and Outfall Box Structure

The structures should be designed to resist the hydrostatic pressure and uplift of the anticipated maximum groundwater levels. Maximum groundwater levels should be the highest of the proposed seasonal high groundwater level or the 100 year flood level for this site. Uplift resistance can be created by both the dead weight of the structure as well as any backfill on any projecting parts of the base slab.

Uplift resistance from extension of the pump station slab should be calculated using a wedge from the outside upper edge of the base of the extended slab upward at a 30 degree angle to the ground surface. Below the water table, the backfills buoyant weight should be used. We estimate, based on other projects in this area, that the buoyant weight of the fine sands is approximately 48 pcf.

5.1.7 Foundation Recommendations

Wet Well and Valve Vault.
In general, the soil beneath the proposed bottom of the wet well consists of very stiff calcareous clay with weathered limestone fragments (CH). We anticipate that the wet well structure will impose less foundation pressure than the weight of the material being removed. Based on the results of our test borings and subgrade preparation and placement of any fill as discussed in this report, a mat foundation system can be used to support the wet well. The net allowable bearing pressure should not exceed 2,000 psf. The mat design may be conducted using a modulus of subgrade reaction (k_s) of 100 pounds per cubic inch (pci). Any structures or utilities founded within excavated areas placed on properly compacted structural fill should be designed for a net allowable bearing capacity of 2,000 psf.

We recommend that 36 inches of No. 57 stone wrapped in geo/filter fabric be placed on the approved subgrade to support the wet well foundation concrete. The No. 57 stone should be extended 3 feet beyond the perimeter of the foundation footprint. The gravel will provide a stable working platform, will help to preserve the subgrade and will be used to facilitate dewatering of the excavation.

Generator Room, Electrical Room, Truck Loading Dock and Porches.
Based on the anticipated light construction for the building, shallow foundations should be designed for a net maximum allowable bearing pressure not to exceed **2,000** pounds per square foot (psf). The foundation and floor slab should bear on properly placed and compacted cohesionless (sand) structural fill. The existing near surface sandy soils should be improved by heavy static compaction after clearing operations to improve foundation support and reduce total and differential settlement.

All continuous wall (strip) footings should be embedded so that the bottom of the foundation is a minimum of 18 inches below adjacent compacted grades on all sides. Strip or wall footings should be a minimum of 24 inches wide based on a requirement of 2 kips per linear foot. Based on an assumed load of 20 kips, individual column footings should be a minimum of 48 inches wide and embedded a minimum of 24 inches below adjacent compacted grades on all sides. The minimum footing sizes should be used regardless of whether or not the foundation loads and allowable bearing pressures dictate a smaller size. These minimum footing sizes tend to provide adequate bearing area to develop bearing capacity and account for minor variations in the bearing materials. All footings should be constructed in a "dry" fashion. All footing excavations should be covered during rain events. During rain events, uncovered excavations may become oversaturated and difficult to compact. Surface run-off water should be drained away from the excavations and not allowed to pond. Top and bottom reinforcement of the foundations should be utilized to minimize the effect of settlement on the foundations and to limit any cracking of the concrete.

Outfall Box Structure.
In general, the soil beneath the proposed bottom of the outfall box consists of firm sandy clay to clay with shell fragments (CH). We anticipate that the outfall box structure will impose less foundation pressure than the weight of the material being removed. Based on the results of our test borings and subgrade preparation and placement of any fill as discussed in this report, a mat foundation system can be used to support the outfall box structure. The net allowable bearing pressure should not exceed 2,000 psf. The mat design may be conducted using a modulus of subgrade reaction (k_s) of 100 pounds per cubic inch (pci). Any structures or utilities founded within excavated areas placed on properly compacted structural fill should be designed for a net allowable bearing capacity of 2,000 psf.

We recommend that 36 inches of No. 57 stone wrapped in geo/filter fabric be placed on the approved subgrade to support the outfall box. The No. 57 stone should be extended 3 feet beyond the perimeter of the foundation footprint. The gravel will provide a stable working platform, will help to preserve the subgrade and will be used to facilitate dewatering of the excavation.

After proper subgrade preparation, including the removal of any unsuitable materials, soils and debris, replaced with clean compacted sand and with top and bottom steel reinforcement of the footings, we estimate maximum total and differential foundation settlements of less than 1 inch and ½ inch, respectively, across a distance of 50 feet.

Footing evaluations should be performed prior to reinforcement and concrete placement. If unsuitable bearing soils are encountered, these soils will need to be recompacted in place, removed and replaced with properly compacted fill, or foundations deepened, to achieve suitable bearing.

Foundation bearing surface evaluations should be performed and concrete placed as quickly as possible after the footings are excavated. Footing concrete should be poured the same day footing excavations are made. If it is required that foundation excavations be left open for more than one day, they should be protected by the placement of a thin (2-3 inch) mud mat of lean concrete. Soils left exposed will soften and will require additional excavation.

A subgrade modulus of 100 pci should be used for designing the mat foundation and slabs.

5.1.8 Earth Slope and Lateral Earth Pressures

Formal analysis of slope stability was beyond the scope of work for this project. Based on the soil types encountered at the site, we recommend that temporary or permanent slopes not exceed 2(H) to 1(V) for this project. The crest or toe of slopes should be no closer than 10 feet to any structure foundation and no closer than 5 feet to the nearest edge of pavement.

Below grade walls such as the pump station walls and loading docks must be designed to resist lateral earth pressures. The "at rest" earth pressure state should be used for soils supporting rigidly restrained walls such as those for the wet well structure. The soils at the site consisting of fine sands (SP/SP-SM) are suitable materials for use as backfill. The table below presents recommended values of earth pressure coefficients for the select backfill materials, assuming an approximate angle of internal friction of 30 degrees. Equivalent fluid densities are frequently used for the calculation of lateral earth pressures. Equivalent fluid densities for the "at-rest" and active conditions based upon a total unit weight of 110 pcf and a fluid unit weight of 62.4 pcf are shown below.

Earth Pressure State	Earth Pressure Coefficient	Equivalent Fluid Density (pcf)		
		Above Water Table	Below Water Table (No Hydrostatic Pressure)	Below Water Table (with Hydrostatic Pressure)
At-Rest (soil backfill)	0.5	55	24	86
Active	0.3	33	14	77
Passive	3.0	330	143	205

The design values and recommendations presented on the previous page assume that the backfill behind the wall will be horizontal with no surcharge loads. Equivalent fluid densities for *no hydrostatic pressure and including hydrostatic pressure are given above. Walls below the groundwater level should include hydrostatic pressures.*

5.1.9 On-Site Soil Suitability and Structural Fill

Soil Types SP/SP-SM/SP-SC, which were encountered in the borings performed, can be categorized as relatively clean fine sands or slightly silty fine sands or slightly clayey fine sands based on the Unified Soil Classification System (USCS). Typically, these materials are deemed suitable for reuse as fill. These soils can be used for grading purposes, site leveling, general engineered fill, structural fill and backfill against the structure wall as well as in other areas, provided the fill is free of organic materials, clays, debris or any other material deemed unsuitable for construction. These soil types will possess improved permeability or drainage characteristics as compared to the underlying soils with increased fines content. These fine sands should require minimal processing in order to properly place and compact. Moisture contents will probably require adjustment in order to affect maximum densification, depending upon specification requirements. It is anticipated that the majority of these soil types will be excavated below the water table and can occur in a relatively saturated state, but should effectively drain within stockpiles. Soils not meeting these requirements will need to be evaluated by **MC²** during construction.

If off-site sources of fill are needed, they should consist of fine sand (SP/SP-SM/SP-SC) with less than 12% passing the No. 200 sieve, free of rubble, organics, clays, debris and other unsuitable material. The moisture content of fill soils at the time of placement and compaction should generally be within 2 percentage points of their optimum moisture content. All materials to be used for backfill or compacted fill construction should be evaluated and, if necessary, tested by **MC²** prior to placement to determine if they are suitable for the intended use. In general, based on the boring results, the majority of the on-site sandy materials excavated for the drainage improvements (with the exception of the clayey fine sands (SC), sandy clays (CH) and organic soils are suitable for use as structural fill and as general subgrade fill and backfill.

The fill material placed around the pump stations structures is critical to support any upper piping. Proper compaction and control of the fill being placed will be required from the bottom of the excavation to the surface in order to properly support utilities or other structures.

Fill material placed adjacent to the walls and beneath structures and piping should be placed in 6 to 8 inch loose lifts compacted using a static roller if near existing structures. Within small excavations such as in utility trenches, around manholes, or within 5 feet of any of the structure walls, we recommend the use of smaller, hand or remote-guided equipment. Placement of loose lift thickness of 4 inches is recommended when using such equipment. All structural fill should be compacted to a dry density of at least 98 percent of the modified Proctor maximum dry density (ASTM D-1557). A representative of **MC²** should perform field density tests on each lift as necessary to assure that adequate compaction is achieved.

5.1.10 Construction Considerations

5.1.10.1 General

It is recommended that **MC²** be retained to provide observation and testing of construction activities involved in the foundation, earthwork, and related activities of this project to ensure that the recommendations contained herein are properly interpreted and implemented. If **MC²** is not retained to perform these functions, we cannot be responsible for the impact of those conditions on the performance of the project.

5.1.10.2 Fill Placement and Subgrade Preparation

The following are our general recommendations for overall site preparation and mechanical densification work for the proposed project based on the anticipated construction and our boring results. These recommendations should be used as a guideline for the project general specifications by the Design Engineer.

1. The excavated subgrade (dewatered trench bottom) for the pipes and associated structures should be leveled, cut to grade if necessary, and then compacted with a vibratory compactor. Careful observations should be made during compaction to help identify any areas of soft yielding soils that may require overexcavation and replacement. If unsuitable material, such as organic or clayey soils, is encountered at the bottom of the pipe or structure embedment depth, overexcavation of an additional 2 and 3 feet of the material is recommended for the pipe and structure, respectively. The excavation should then be backfilled to foundation grade with clean sands in controlled lifts not exceeding 6-inches and compacted to a density of at least 98 percent of the maximum density as determined by ASTM D-1557. Care should be used when operating the compactor to avoid transmission of vibrations to existing structures or other construction operations that could cause settlement damage or disturb occupants. Dewatering may also have an effect on adjacent structures. A preconstruction survey with video and/or photographs of adjacent residences/structures is recommended to check for existing cracking prior to construction and during construction. Vibration and groundwater levels monitoring are also recommended.

2. Prior to beginning compaction, soil moisture contents may need to be controlled in order to facilitate proper compaction. A moisture content within 2 percentage points of the optimum indicated by the modified Proctor test (ASTM D-1557) is recommended.

3. Following satisfactory completion of the initial compaction on the excavation bottom, the construction areas may be brought up to finished subgrade levels. Fill should consist of fine sand with less than 12% passing the No. 200 sieve, free of rubble,

organics, clay, debris and other unsuitable material. Fill should be tested and approved prior to acquisition and/or placement. Approved sand fill should be placed in loose lifts not exceeding 6-inches in thickness and should be compacted to a minimum of 98% of the maximum modified Proctor dry density (ASTM D-1557). Density tests to confirm compaction should be performed in each fill lift before the next lift is placed.

4. It is recommended that a representative from our firm be retained to provide on-site observation of earthwork activities. The field technician would monitor the placement of approved fills and compaction efforts and provide compaction testing. Density tests should be performed in subgrade sands after rolling and in each fill lift. It is important that **MC²** be retained to observe that the subsurface conditions are as we have discussed herein, and that construction and fill placement is in accordance with our recommendations.

5.2 Storm Main and Reinforced Concrete Pipes (RCP)

5.2.1 General

Based on the information provided by **McKim & Creed**, the forcemain and RCP pipes will be installed at depths ranging from 5 to 30 feet.

5.2.2 Pipeline Considerations

In general, the majority of the borings (AB-1 to AB-4 and SPT-1 to SPT-3) indicated very loose to loose fine sand and/or slightly silty fine sand and/or slightly clayey fine sand (SP/SP-SM/SP-SC) or silty fine sand (SM) extending to depths ranging from 8 to 12 feet. Next, very loose to loose clayey fine sand (SC) and/or firm to stiff clay to sandy clay (CH) were encountered in borings SPT-1 through SPT-3, and AB-1 extending to depths ranging from 8.0 feet to the boring termination depth of 20 feet. Next, the borings indicated very stiff calcareous clay with weathered limestone (CL) and very soft to very hard weathered limestone (LS) with calcareous clay was encountered in boring SPT-3 extending from a depth of approximately 22.0 feet to the boring termination depth of 30 feet.

Settlement due to the presence of the pipeline should be minimal unless the subsoil is excessively disturbed during the installation, or the phreatic surface is lowered for a substantial period of time, or if new loads are placed above or near the pipeline. Uplift pressure from the groundwater may be considered when the bottom of the pipeline is significantly below the existing groundwater level.

Surface water and groundwater control will be necessary during construction of the pipeline to establish a stable sand bottom on which to bed the pipeline. Dewatering consisting of sump pumps and/or well pointing has been successful in the past. Dewatering must be conducted with care to avoid settlement of nearby structures, roads or utilities, and in such a manner that the areas possibly affected are as small as possible.

Depending upon shallow groundwater levels and the effectiveness of dewatering at the time of construction, seepage may enter the excavated trenches from the bottom and sides. Such seepage will act to loosen soils and create difficult working conditions. Groundwater levels should be determined immediately prior to construction. Shallow groundwater should be kept at least 24 inches below the working area to facilitate proper material placing and compaction. Organic soils and clayey soils should be removed (if encountered) within 24 inches from the bottom of the pipeline and replaced with properly compacted clean sands (SP/SP-SM/SP-SC).

A density of at least 98% of the modified Proctor maximum dry density (ASTM D-1557) is recommended for all fill materials and natural subgrade under the pipeline. The subgrade soils should be firm and stable prior to placement of the pipe. Once the pipeline is placed, it is recommended that backfill around the sides of the pipe be placed and compacted in equal lifts with a vibratory tamper in lifts not to exceed 6-inches (loose) to avoid laterally displacing the pipeline. Failure to compact the backfill will result in future settlement of the ground surface.

Pipeline backfill should be clean fine sand (free of clay, rubble, organics and debris) with less than 12% passing the No. 200 sieve and placed in compacted lifts. Some contractors like to place a gravel working bed in wet areas. Fine gravel, such as No. 57, and No. 67 stone may be used in limited areas. A continuous gravel bed should not be placed for the full pipe length to prevent a flow conduit under the pipeline. The gravel, where used, should be compacted and the compaction confirmed by visual observation.

The non-organic clean fine sands and slightly silty fine sands and slightly clayey fine sands (SP/SP-SM/SP-SC) encountered in the project with less than 12 percent passing the No. 200 sieve will be suitable for backfill.

5.2.3 Pipe Structure Excavations

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 excavation appears suitable as load bearing materials, the soils should be prepared for construction by compaction to a dry density of at least 98% of the modified Proctor maximum dry density (ASTM D-1557) for 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% of the modified Proctor maximum dry density (ASTM D-1557), as previously described. Gravel, or crushed #57 or #67 stone, if used, should be compacted and the compaction confirmed by visual observation.

5.2.4 Pipe Lateral Earth Pressures

The pipeline will be subject to lateral earth pressures. For pipes which are restrained and adjacent to moderately compacted backfill, design is usually based on "at-rest" earth pressures. Active pressures are usually employed for unrestrained retaining wall design. Several earth pressure theories could be utilized. One of the most straightforward is the equivalent fluid pressure or Rankine Theory.

The pressures presented in Section 5.1.8 of this report should be used for designing the pipes.

5.2.5 Pipe Structural Fill

All materials to be used for backfill or compacted fill construction should be evaluated and tested prior to placement to determine if they are suitable for the intended use. In general, based on the boring results, the majority of the on-site clean sandy materials (SP/SP-SM/SP-SC) excavated for the drainage improvements are suitable for use as structural fill and as general subgrade fill and backfill. Suitable structural fill materials should consist of fine to medium sand with less than 12% passing through the No. 200 sieve (SP/SP-SM/SP-SC) and be free of rubble, organics, clay, debris and other unsuitable material.

All structural fill should be compacted to a dry density of at least 98 percent of the modified Proctor maximum dry density (ASTM D-1557). In general, the compaction should be accomplished by placing the fill in maximum 6-inch loose lifts and mechanically compacting each lift to at least the specified minimum dry density. Field density tests should be performed on each lift as necessary to assure that adequate compaction is achieved.

5.2.6 Pipe Construction Considerations

5.2.6.1 General

It is recommended that observation and testing of construction activities involved in the foundation, earthwork, and related activities of this project be performed by qualified personnel to ensure that the recommendations contained herein are properly interpreted and implemented.

5.2.6.2 Fill Placement and Subgrade Preparation

The recommendations for the Pipes Construction Considerations are the same as the General Construction Considerations in Section 5.1.10.2.

5.2.7 Pipe Groundwater Control

Dewatering will be necessary to achieve the required depth of excavation and compaction of backfill. Groundwater can normally be controlled in excavations with a sump pump and/or well pointing as previously discussed. For deep excavations, dewatering using temporary well points or temporary sheet pile walls may be necessary.

Surface water and groundwater control will be necessary during construction to permit establishment of a stable sand bottom. If a pump is used, a standby pump is recommended.

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

Groundwater levels should be determined by the contractor immediately prior to construction. Shallow groundwater should be kept at least 24 inches below the lowest working area to facilitate proper material placement and compaction.

5.2.8 Pipe Temporary Slopes

Side slopes for temporary excavations may stand near one (1) horizontal to one (1) vertical (1H:1V) for short dry periods of time and a maximum excavation depth of four (4) feet. Where restrictions do not permit slopes to be constructed as recommended above, the excavation should be shored in accordance with current OSHA requirements. In addition, any open cut excavations adjacent to existing structures should be evaluated by a geotechnical engineer on a case by case basis. During construction, excavated materials should not be stockpiled at the top of the slope within a horizontal distance equal to the excavation depth.

Excavation slopes should conform to OSHA, State of Florida and any other local regulations. The dewatering system chosen for use on this project should consider the nature of the permeable upper sands encountered at the project site. The contractor should also assess equipment loads and vibrations when considering slopes or excavation bracing.

5.2.9 Federal Temporary Excavation Regulations

Recommendations for the Federal Temporary Excavation Regulations are presented in Section 5.1.5 of this report.

We are providing this information solely as a service to our client. **MC²** does not assume responsibility for construction site safety or the contractor's or other party's compliance with local, state, and federal safety or other regulations.

5.2.10 Pipe On-Site Soil Suitability

All materials to be used for backfilling or compacted fill construction should be evaluated and tested prior to placement to determine if they are suitable for the intended use. In general, based on the boring results, the majority of the on-site clean, non-organic sandy material (SP/SP-SM/SP-SC) can be used as structural fill and as general subgrade fill and backfill in other areas, provided the fill material is free of rubble, clay, rock, roots and organics. Suitable structural fill materials should consist of fine sand with less than 12% passing the No. 200 sieve, and be free of rubble, organics, clay, debris and other unsuitable material. Any off-site materials used as fill should be approved prior to acquisition.

6.0 ENVIRONMENTAL CLASSIFICATION

We performed corrosion tests on random samples from the forcemain auger and SPT borings. Test results obtained are presented in **Table 2, Appendix A**. Based on the FDOT's "Structures Design Guidelines, the forcemain subsurface environment is classified as moderately aggressive and slightly aggressive for steel and concrete. We recommend using the FDOT Structures Design Guidelines and FDOT Standard Specifications for corrosion protection measures.

7.0 REPORT LIMITATIONS

The recommendations detailed herein are based on the available soil information obtained by **MC^2** and information provided by **McKim & Creed** 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 foundations or other recommendations are required. In the event that **MC^2** is not retained to perform these functions, **MC^2** cannot be 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 the final design plans and specifications 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 **McKim & Creed.**

APPENDIX A

Boring Location Plan – Sheet 1

Report of Core Borings – Sheet 2

Summary of Laboratory Tests - Table 1

Summary of Corrosion Parameters Tests – Table 2

Soil Parameters for Shoring– Table 3

LEGEND

- SPT BORING LOCATION.
- HAND AUGER BORING LOCATION.

NOTES:

BORING LOCATIONS ARE APPROXIMATE.

BASE MAP PROVIDED BY MCKIM & CREED

DATE	NAME	REVISION	APPROVED BY:

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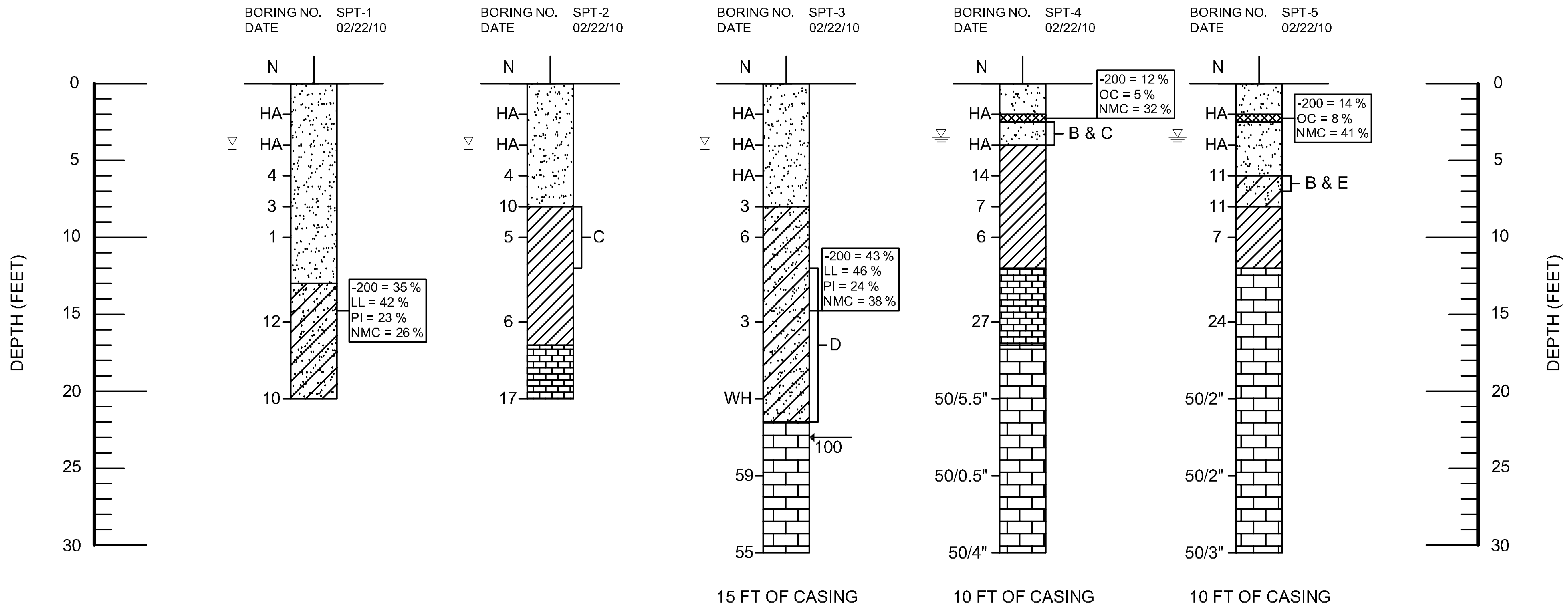
	NAME	DATE
DESIGNED BY:	IR	02/10
DRAWN BY:	IR	02/10
CHECKED BY:	KS	02/10
SUPERVISED BY:		

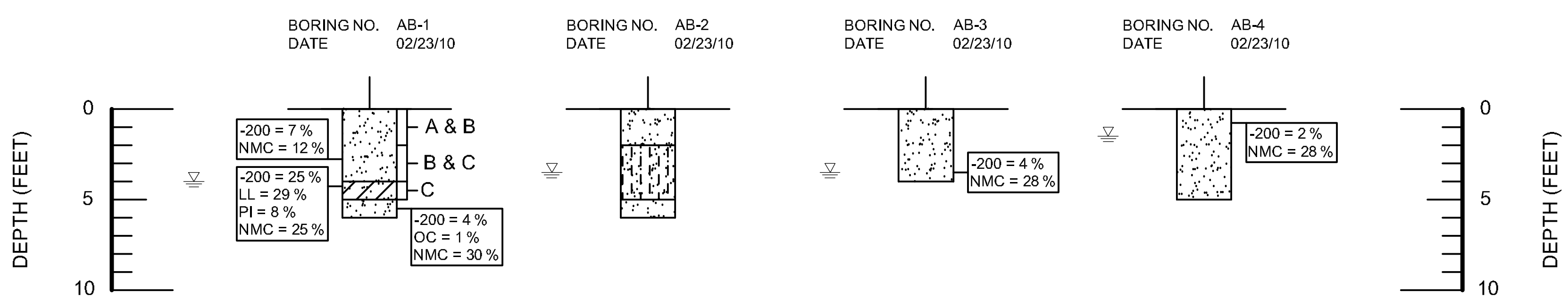
BORING LOCATION PLAN	PROJECT NO.	SHEET NO.
Alline Avenue Stormwater Pumping Station Tampa, FL	T050704.084	1



LEGEND

- (SP/SP-SM/SP-SC) GRAY, PALE BROWN, OR BROWN FINE SAND, SLIGHTLY SILTY FINE SAND, TO SLIGHTLY CLAYEY FINE SAND.
- (SM) GRAY OR DARK BROWN SILTY FINE SAND.
- (SM) BLACK ORGANIC SILTY FINE SAND.
- (SC) GREEN OR GREENISH GRAY CLAYEY FINE SAND.
- (CH) GREEN OR GREENISH GRAY SANDY CLAY TO CLAY.
- (CL) PALE BROWN CALCAREOUS CLAY WITH WEATHERED LIMESTONE FRAGMENTS .
- (LS) PALE BROWN WEATHERED LIMESTONE WITH CALCAREOUS CLAY.

A WITH ROOTS
B WITH TRACES TO SOME ROCK
C WITH TRACES TO SOME SHELL
D WITH TRACES TO SOME LIMESTONE
E WITH CHERT



NOTES:

▽	WATER TABLE	-200	FINES PASSING NO. 200 SIEVE (%)
N	SPT N-VALUE	NMC	NATURAL MOISTURE CONTENT (%)
HA	HAND AUGER	OC	ORGANIC CONTENT (%)
100	LOSS OF CIRCULATION	LL	LIQUID LIMIT (%)
WH	WEIGHT OF HAMMER	PI	PLASTIC INDEX (%)

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

0' 10

SCALE: 1" = 10'
HORIZONTAL:
N.T.S

DATE	NAME	REVISION	APPROVED BY:

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	NAME	DATE
DESIGNED BY:	IR	02/10
DRAWN BY:	IR	02/10
CHECKED BY:	KS	02/10
SUPERVISED BY:		

REPORT OF CORE BORINGS	PROJECT NO.	SHEET NO.
Alline Avenue Stormwater Pumping Station Tampa, FL	T050704.084	2

Table 1 **Summary of Laboratory Tests** **Proposed Alline Avenue Pumping Station** **MC^2 Project No. T050704.084**														
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-1	2.0 – 3.5	SP-SM							7				12
AB-1	4.0 – 4.5	SC							25	29	8		25
AB-1	5.0 – 6.0	SP							4			1	30
AB-3	3.0 – 4.0	SP							4				28
AB-4	1.0 – 1.5	SP							2				28
SPT-1	14 – 15.5	SC							35	42	23		26
SPT-3	14 – 15.5	SC							43	46	24		38
SPT-4	2.0 – 2.5	Organic SM							12			5	32
SPT-5	2.0 - 2.5	Organic SM							14			8	41
NP = Non-plastic or no plasticity													

Table 2
Summary of Corrosion Parameters Tests
Proposed Alline Avenue Pumping Station
MC^2 Project No. T050704.084

Boring No./Station and Offset (Feet)	Sample Depth (Feet)	Stratum Number	Unified Soil Classifi.	pH	Resistivity (ohm-cm)	Sulfates (ppm)	Chlorides (ppm)	Redox Potential (mV)	Environmental Classification*	
									Steel	Concrete
AB-1	4.5 – 5.0	-	SM	8.03	2010	68	44	432.8	Moderately Aggressive (Resistivity)	Moderately Aggressive (Resistivity)
SPT-1	4.0 – 6.0	-	SP/SP-SM/SP-SC	7.89	3825	64	<10	436.4	Moderately Aggressive (Resistivity)	Slightly Aggressive
SPT-3	4.0 – 6.0	-	SP/SP-SM/SP-SC	7.20	4870	44	40	436.2	Moderately Aggressive (Resistivity)	Slightly Aggressive

*= As per the latest version of the FDOT Structures Design Guidelines, Table 1.1a

**** Any reading represented as "0.0" is below the detection limit of 4.8 ppm**

Table 3
Summary of Soil Parameters
Proposed Alline Avenue Pumping Station
MC^2 Project No. T050704.084

Boring No.	Depth range, ft	SPT "N" Value Range	Unified Soil Classification	Approximate Soil Unit Weight (pcf)		Soil Angle of Friction (degrees)	Cohesion (psf)	Earth Pressure Coefficient	
				saturated	submerged			Active (K_a)	Passive (K_p)
SPT-4	0 – 2	HA	SP/SP-SM/SP-SC	100.0	37.6	28	0	0.361	2.77
	2 – 2.5	HA	Organic SM	90	27.6	26	0	0.391	2.56
	2.5 - 4	HA	SP/SP-SM/SP-SC	100.0	37.6	28	0	0.361	2.77
	4 – 12	6 – 14	CH	115.0	52.6	0	1000	1.000	1.00
	12 – 17	27	CL	125.0	62.6	0	3350	1.000	1.00
	17 - 30	50/1"- 50/6"	LS	135.0	72.6	0	8000	1.000	1.00
SPT-5	0 - 2	HA	SP/SP-SM/SP-SC	100.0	37.6	28	0	0.361	2.77
	2 – 2.5	HA	Organic SM	90	27.6	26	0	0.391	2.56
	2.5 – 6	11	SP/SP-SM/SP-SC	110.0	47.6	30	0	0.333	3.00
	6 – 8	11	SC	110.0	47.6	30	0	0.333	3.00
	8 – 12	7 – 14	CH	115.0	52.6	0	1000	1.000	1.00
	12 - 30	50/2" – 50/3"	LS	135.0	72.6	0	8000	1.000	1.00

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.

Hollow Stem Augering: 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.

Core Drilling: 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 Penetration Testing
b) Undisturbed Sampling
c) Dynamic Cone Penetrometer Testing
d) Water Level Readings

The procedures utilized for this project are presented below.

Standard Penetration Testing: 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, 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.

Water Level Readings: 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 is 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		Consistency of Cohesive Soils	
Very Loose	≤ 4 bpf	Very Soft	≤ 2 bpf
Loose	5 - 10 bpf	Soft	3 - 4 bpf
Medium Dense	11 - 30 bpf	Firm	5 - 8 bpf
Dense	31 - 50 bpf	Stiff	9 - 15 bpf
Very Dense	> 50 bpf	Very Stiff	16 - 30 bpf
		Hard	30 – 50 bpf
(bpf = blows per foot, ASTM D 1586)		Very Hard	> 50 bpf

Relative Hardness of Rock		Particle Size Identification	
Very Soft	Hard Rock disintegrates or easily compresses to touch; can be hard to very hard soil.	Boulders	Larger than 12"
		Cobbles	3" - 12"
Soft	May be broken with fingers.	Gravel	
		Coarse	3/4" - 3"
Moderately Soft	May be scratched with a nail, corners and edges may be broken with fingers.	Fine	4.76mm - 3/4"
		Sand	
		Coarse	2.0 - 4.76 mm
Moderately Hard	Light blow of hammer required to break samples.	Medium	0.42 - 2.00 mm
		Fine	0.42 - 0.074 mm
Hard	Hard blow of hammer required to break sample.	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 %