APPENDIX A

GEOTECHNICAL REPORT

Geotechnical Investigation Report. Lift Stations 28, 54, 60, and 67
Replacements Orlando, FL AEA Project No. 201513-2.
Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a civil engineer may not fulfill the needs of a constructor — a construction contractor — or even another civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared solely for the client. No one except you should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. And no one — not even you — should apply this report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical-engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

Geotechnical Engineers Base Each Report on a Unique Set of Project-Specific Factors

Geotechnical engineers consider many unique, project-specific factors when establishing the scope of a study. Typical factors include: the client’s goals, objectives, and risk-management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical-engineering report that was:

- not prepared for you;
- not prepared for your project;
- not prepared for the specific site explored; or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical-engineering report include those that affect:

- the function of the proposed structure, as when it’s changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the composition of the design team; or
- project ownership.

As a general rule, always inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.

Subsurface Conditions Can Change

A geotechnical-engineering report is based on conditions that existed at the time the geotechnical engineer performed the study. Do not rely on a geotechnical-engineering report whose adequacy may have been affected by: the passage of time; man-made events, such as construction on or adjacent to the site; or natural events, such as floods, droughts, earthquakes, or groundwater fluctuations. Contact the geotechnical engineer before applying this report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ — sometimes significantly — from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide geotechnical-construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report’s Recommendations Are Not Final

Do not overrely on the confirmation-dependent recommendations included in your report. Confirmation-dependent recommendations are not final, because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual subsurface conditions revealed during construction. The geotechnical engineer who developed your report cannot assume responsibility or liability for the report’s confirmation-dependent recommendations if that engineer does not perform the geotechnical-construction observation required to confirm the recommendations’ applicability.

A Geotechnical-Engineering Report Is Subject to Misinterpretation

Other design-team members’ misinterpretation of geotechnical-engineering reports has resulted in costly
problems. Confront that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team’s plans and specifications. Constructors can also misinterpret a geotechnical-engineering report. Confront that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing geotechnical construction observation.

**Do Not Redraw the Engineer’s Logs**

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical-engineering report should never be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, but recognize that separating logs from the report can elevate risk.

**Give Constructors a Complete Report and Guidance**

Some owners and design professionals mistakenly believe they can make constructors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give constructors the complete geotechnical-engineering report, but preface it with a clearly written letter of transmittal. In that letter, advise constructors that the report was not prepared for purposes of bid development and that the report’s accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. Be sure constructors have sufficient time to perform additional study. Only then might you be in a position to give constructors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

**Read Responsibility Provisions Closely**

Some clients, design professionals, and constructors fail to recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled “limitations,” many of these provisions indicate where geotechnical engineers’ responsibilities begin and end, to help others recognize their own responsibilities and risks. Read these provisions closely. Ask questions. Your geotechnical engineer should respond fully and frankly.

**Environmental Concerns Are Not Covered**

The equipment, techniques, and personnel used to perform an environmental study differ significantly from those used to perform a geotechnical study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. Unanticipated environmental problems have led to numerous project failures. If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. Do not rely on an environmental report prepared for someone else.

**Obtain Professional Assistance To Deal with Mold**

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the express purpose of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold-prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, many mold-prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical-engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; none of the services performed in connection with the geotechnical engineer’s study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.

**Rely, on Your GBC-Member Geotechnical Engineer for Additional Assistance**

Membership in the Geotechnical Business Council of the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project. Confer with you GBC-Member geotechnical engineer for more information.
June 3, 2016

Tetra Tech, Inc.
201 East Pine Street, Suite 1000
Orlando, Florida 32801

Attention: Brett Kuziak, E.I.

Reference: Geotechnical Investigation Report
Lift Stations 28, 54, 60, and 67 Replacements
Orlando, Florida
AEA Project No. 201513-2

Dear Mr. Kuziak:

Antilllian Engineering Associates, Inc. has completed geotechnical engineering investigations for the Lift Stations 28, 54, 60, and 67 Replacements project in Orlando, Florida. The work on this project was authorized under Tetra Tech’s Continuing Wastewater Engineering Services contract with the City of Orlando, and done in general accordance with the scope of services presented in our proposal dated March 16, 2015.

This report contains the results of our investigations, our recommendations for lift-station design, earthwork, groundwater control, building foundation stabilization, and other concerns as appropriate.

It has been our pleasure to serve Tetra Tech and the City of Orlando on this project. Please do not hesitate to call if you have any questions or if you need additional information.

Antilllian Engineering Associates, Inc.

3331 Bartlett Boulevard • Orlando, FL 32811 • (407) 422-1441 • Fax (407) 422-2226

Attachments: Figures
Appendix A: Field and Laboratory Investigations
Appendix B: Important Information About This Geotechnical-Engineering Report
Appendix C: Constraints and Restrictions
PROJECT DESCRIPTION

The City of Orlando Public Works Department (“the City”) is planning to replace four sanitary-sewer lift stations on the western side of the city. Those lift stations were designated as follows:

- Lift Station 28: Gulfstream Road, between Marathon Avenue and Gulf Stream Court
- Lift Station 54: Sun Court, about 250 feet south of the intersection with West South Street
- Lift Station 60: Barley Street, between Dorcas Court and Argos Avenue
- Lift Station 67: Raleigh Street, about 100 feet west of the intersection with Broome Avenue.

It is our understanding that the existing lift stations are about 60 years old and will be replaced with new reinforced-concrete lift stations. In addition, Lift Station 28 is housed in a single-story service building with concrete-masonry-block walls. Some walls are cracked and out of plumb, but City staff intends to preserve the building as part of the new lift station. As a result, City staff requested an assessment of the soils adjacent to the building foundations in an effort to identify the causes of the visible distress and develop an appropriate remedy.

The City retained Tetra Tech, Inc. (“Tetra Tech”) to design this project. TetraTech retained Antillian Engineering Associates, Inc. to investigate the subsurface conditions at each pump station site and provide geotechnical-engineering recommendations for design and construction of the replacement lift stations and restoration of the building foundations at Lift Station 28.

AVAILABLE INFORMATION

United States Geological Survey (USGS) quadrangle topographic maps of the area and the May, 2009 USGS map “Potentiometric Surface of the Upper Florida Aquifer in the St Johns River Water Management District” were examined for general information about the lift station sites. Tetra Tech provided a draft set of plans dated March 9, 2016, which we examined for additional information.

The USGS topographic map (part of which is reproduced as Figure 1) showed the general area of the four lift stations a broad, low-lying plain bordered by, and interspersed with, areas of marsh on the western side of the city. In general, the ground surface was mapped below the Elevation 100 feet NGVD (El. 100) contour. Land usage on the plain was shown as mostly residential. Lake Mann and Clear Lake were shown within the project vicinity. The surface of Lake Mann was mapped at El. 90, while the surface of Clear Lake was mapped at El. 92.

The potentiometric surface map showed the potentiometric surface of the Upper Floridan Aquifer near the El. 50 NGVD contour in the project vicinity.

The preliminary plans dated March 9, 2016 by Tetra Tech (the “Tetra Tech plans”) showed each existing lift station as a reinforced-concrete wet well connected by a horizontal, intake (“suction”) line to a dry, cylindrical, underground, metal structure (a “dry well” or “can”) containing pumps, valves, and electrical controls. The drawings showed the replacement wet wells as precast concrete
structures containing submersible pumps. Electrical controls for the replacement lift stations were housed in panel boxes supported on posts at the ground surface. Diesel-powered, standby generators, above-ground fuel tanks and new concrete driveways were also shown.

Elevations shown on the Tetra tech plans indicated that the existing wet wells were cast-in-place concrete structures with inside diameters between seven feet and eight feet and inside depths between 13 feet and 18 feet. The new wet wells will be precast-concrete structures with eight-foot inside diameters and inside depths between 16 feet and 24 feet. Each new wet well will include a precast concrete monolithic base with a footing thickness of 18 inches. The base of the wet wells will bear on 24 inches of “FDOT Granite No 57 stone.” Joints between the precast concrete wet-well sections will be filled with a sealant. A heat-shrink sealing band will be applied around the outsides of the joints and a “5 mm black agru-lined wet well liner” will be installed in the completed wet well.

FIELD INVESTIGATIONS

Site reconnaissances were conducted to prepare for the drilling program. Subsurface-exploration locations were established and staked for underground utility location and marking as required by Florida Statutes and to facilitate identification by the field crews. Subsurface explorations included Cone Penetration Test (“CPT”) soundings at Lift Station 28 and Standard Penetration Test (“SPT”) borings at the other three lift stations. CPT soundings were designated “CPT-1” through “CPT-3” while test borings were designated using the lift station numbers, i.e., “54-1,” “60-1,” and “67-1.” Approximate subsurface-exploration locations are shown on Figure 3 and Figure 4.

The CPT soundings were conducted by InSitu Group of Orlando, Florida under contract to this firm, and were supervised by a geotechnical engineer from our office. Soundings were conducted in accordance with ASTM D 5778-95 and were completed at depths between 40 feet and 75 feet below the existing ground surface. Cone tip resistance, side friction and porewater pressure behind the base of the cone (the “U2” position) were measured and recorded electronically at intervals of two centimeters (0.066 feet) from the ground surface to the indicated completion depths.

SPT Borings 54-1 and 67-1 were drilled to a depth of 35 feet as intended. Boring 60-1 had to be drilled to 40 feet to define the depth of potentially unfavorable soil conditions encountered at 35 feet. All three boreholes were initially drilled by hand to a depth of four feet using a bucket auger to reduce the likelihood of damaging unmarked underground utilities. Auger drilling and sampling were conducted in accordance with ASTM D 1452. The boreholes were then advanced from four feet to ten feet by continuous split-spoon soil sampling. The Standard Penetration Test (SPT) with split-spoon sampling was conducted in accordance with ASTM D 1586. All three borings were advanced from ten feet to the indicated completion depths by split-spoon sampling and mud-rotary drilling, with testing and split-spoon sampling conducted at five-foot intervals to the indicated completion depths.
Sampler penetration resistance expressed in hammer blows per foot (“SPT N-values”), soils recovered from the auger and the samplers, and other noteworthy conditions were logged by the field crew. Depth to groundwater in each borehole was measured and recorded on the field logs. The depth to groundwater in the CPT soundings was deduced using the porewater pressure measurements that had been recorded electronically. Representative soil samples from the test borings were sealed in clean, airtight containers for transportation to our Orlando office. At the completion of drilling, the boreholes were backfilled with soil cuttings. The CPT soundings were left to close on their own.

LABORATORY TESTING

The recovered soil samples were examined in our office by a geotechnical engineer who confirmed the descriptions on the field logs, classified the soils visually in accordance with the Unified Soil Classification System (ASTM D 2488) and developed a representation of the soil stratigraphy at each boring location. Representative soil samples were selected for laboratory testing, which consisted of 18 percent fines tests, one Atterberg limits series and one moisture content test. Test results are presented on the boring logs and on the Summary of Laboratory Test Results sheet in Appendix A.
SURFACE CONDITIONS

All four lift stations were operational, wastewater lift stations. Three were fenced and locked, while Lift Station 54 was not. Lift Station 28 was the only station with a building. The walls on the eastern side and northeastern corner of that building had cracks and other signs of structural distress. Above-ground piping, electrical panels, utility poles and tops of the buried wet wells and vaults were visible. Plastic flags, paint markings and manhole covers indicated the presence of buried, sanitary-sewer force mains and other underground utilities.

SUBSURFACE CONDITIONS

The stratigraphy, soil types and groundwater levels described below are based on the results of the field testing and laboratory testing programs. SPT N-values were used as empirical indications of soil condition. Unified Soil Classification System group names and group symbols were used for soil classification. The descriptions below are general and describe the major soil types that were encountered. Detailed subsurface characteristics at each boring location are shown on the boring logs and on the Summary of Laboratory Test Results sheet in Appendix A.

Lift Station 28 (CPT Soundings CPT-1 through CPT-3)

Examination of the plotted CPT data revealed soils that were generally consistent between the three sounding locations. Three anomalies were also clearly visible. In general, the penetrated soils behaved like sands that contained varying amount of silt and clay. The uppermost layer was about ten feet thick and consisted of soils with low penetration resistance values. Cone tip resistance values in this layer ranged from less than 1 ton per square foot (tsf) to about 160 tsf, with most values between 10 tsf and 100 tsf. The ratio of soil friction to cone resistance ($f_s/q_c$, also known as the “friction ratio”) was typically about 1 percent. The penetrated materials were classified mostly as “sand to silty sand,” and “silty sand to sandy silt” using the soil behavior type classification developed by Robertson et al. (1986).

Beneath the uppermost layer was a layer of materials with higher resistance to penetration that extended to depths between 29 feet and 33 feet below the ground surface. Cone tip resistance values in this layer ranged from 17 tsf to 178 tsf, with most values between 60 tsf and 150 tsf. The friction ratio was typically lower than 1 percent in these soils. The penetrated materials were classified mostly as “sand,” and “sand to silty sand” using the soil behavior type classification developed by Robertson et al. (1986).

Beneath the sands was an interbedded zone of materials that behaved mostly like silty sands and clayey sands with isolated bands of sand, silt and clay. The actual thickness of these materials could not be confirmed because the soundings were terminated as planned before fully penetrating them. Cone resistance values ranged from 20 tsf to more than 300 tsf and the friction ratio was generally between 1 percent and 2 percent. Materials in this layer were classified as “sand to silty sand,” “silty
sand to sandy silt,” “sandy silt to clayey silt,” “clayey silt to silty clay,” and “silty clay to clay” using the soil behavior type classification developed by Robertson et al. (1986).

The three observed anomalies were observed in this interval. The first was a zone of material between three feet and ten feet below the ground surface in sounding CPT-1 which behaved like “sensitive fine-grained” and “clayey silt to silty clay” using the soil behavior type classification developed by Robertson et al. (1986). The second was a zone of material between two feet and about six feet below the ground surface in CPT-2 which behaved like “soft clays to organic,” while the third was between 28 feet and 36 feet below the ground surface in CPT-2, which behaved like “clayey silt to silty clay,” “sandy silt to clayey silt,” and soft/loose - possible voids” using the Robertson et al classifications. All three anomalies appeared to have been caused by tip resistance values lower than 8 tsf. Closer examination of the tabulated data revealed that these values were the lowest tip resistance values in each layer described in the previous paragraphs. Low tip resistance can cause soil response that would mimic clayey soils, organic soils, or possible voids even though the side friction values did not increase appreciably.

Groundwater was estimated in the soundings at a depth of about three feet below the existing ground surface, based on the porewater pressure profiles. Hydrostatic conditions were indicated in the near-surface soils in all three soundings. Graphical representations of the subsurface characteristics at each location are shown on the CPT plots in Appendix A.

**Lift Station 54 (Boring 54-1)**

The uppermost material in this borehole was dark grayish brown sand about a foot thick. Beneath the sand was grayish brown to dark grayish brown sand that contained clay. Its encountered thickness was about 12 feet. SPT N-values ranged from 1 blow per foot (“bpf”) to 4 bpf, indicating very loose to loose conditions. Percent fines testing of three samples indicated fines contents (fraction by dry weight passing the U.S. Standard No. 200 sieve) that ranged from 15 percent to 29 percent. Based on visual examination and laboratory testing, this soil was classified as clayey sand (“SC”).

Beneath the very loose to loose clayey sand was gray clay. The encountered thickness of this soil was about five feet. The only SPT N-value obtained in this soil was 6 bpf, indicating a firm consistency. Percent fines testing of one sample indicated a fines content of 51 percent. Additional laboratory testing indicated a liquid limit of 59, plasticity index of 36 and natural moisture content of 23 percent. Based on visual examination and the laboratory testing results, this soil was classified as high-plasticity clay (“CH”).

Beneath the firm clay was light gray fine sand that contained silt. The encountered thickness of this soil was about nine feet. SPT N-values in this soil were 4 bpf and 7 bpf, indicating loose conditions. Percent fines testing of one sample indicated a fines content of 12 percent. Based on visual examination and laboratory testing, this soil was classified as silty sand (“SM”).

Beneath the loose silty sand was light gray fine sand that contained silt. The encountered thickness of this soil was about three feet. Its actual thickness could not be confirmed because the boring had
been terminated at its intended depth before fully penetrating this soil. The SPT N-value in this soil was 8 bpf, indicating a loose condition. Percent fines testing of one sample indicated a fines content of 7 percent. Based on visual examination and laboratory testing, this soil was classified as sand with silt (“SP-SM”).

Groundwater was encountered in this borehole about three feet below the existing ground surface.

**Lift Station 60 (Boring 60-1)**

The uppermost material in this borehole was dark brown to very dark brown and gray fine sand that contained silt. The encountered thickness of this soil was about 18 feet. SPT N-values in this soil ranged from Weight-of-Hammer (“WOH”) to 4 bpf, indicating very loose conditions. Percent fines testing of three samples indicated fines contents that ranged from 9 percent to 12 percent. Based on visual examination and laboratory testing, this soil was classified as sand with silt (“SP-SM”).

Beneath the very loose sand with silt was gray fine sand that contained silt. Its encountered thickness was about five feet. The SPT N-value was 4 bpf, indicating a very loose condition. Percent fines testing of one sample indicated a fines content of 16 percent. Based on visual examination and laboratory testing, this soil was classified as silty sand (“SM”).

Beneath the very loose silty sand was light gray fine sand that contained silt. Its encountered thickness was about five feet. The SPT N-value in this soil was 7 bpf, indicating a loose condition. Percent fines testing indicated a fines content of 7 percent. Based on visual examination and laboratory testing, this soil was classified as sand with silt (“SP-SM”).

Beneath the loose sand with silt was green fine sand that contained silt. Its encountered thickness was about five feet. The SPT N-value was 4 bpf, indicating a very loose condition. Percent fines testing of one sample indicated a fines content of 16 percent. Based on visual examination and laboratory testing, this soil was classified as silty sand (“SM”).

Beneath the very loose silty sand was greenish gray fine sand that contained silt. Its encountered thickness was about two feet. Its actual thickness could not be confirmed because the borehole had been terminated five feet past its intended depth without fully penetrating this soil. The SPT N-value in this soil was 11 bpf, indicating a medium dense condition. Percent fines testing indicated a fines content of 7 percent. Based on visual examination and laboratory testing, this soil was classified as sand with silt (“SP-SM”).

Groundwater was encountered in this borehole about three feet below the existing ground surface.

**Lift Station 67 (Boring 67-1)**

The uppermost material in this borehole was mixed brown and yellowish brown to dark yellowish brown and dark brown fine sand that contained silt. The encountered thickness of this soil was about 13 feet. SPT N-values in this soil ranged from 6 bpf to 10 bpf, indicating loose conditions. Percent
fines testing of one sample indicated a fines content of 12 percent. Based on visual examination and laboratory testing, this soil was classified as sand with silt (“SP-SM”).

Beneath the loose sand with silt was grayish brown fine sand that contained silt. Its encountered thickness was about five feet. The SPT N-value was 3 bpf, indicating a very loose condition. Percent fines testing of one sample indicated a fines content of 14 percent. Based on visual examination and laboratory testing, this soil was classified as silty sand (“SM”).

Beneath the very loose silty sand was gray fine sand that contained clay. Its encountered thickness was about five feet. The SPT N-value in this soil was 7 bpf, indicating a loose condition. Percent fines testing indicated a fines content of 36 percent. Based on visual examination and laboratory testing, this soil was classified as clayey sand (“SC”).

Beneath the loose clayey sand was light brownish gray fine sand that contained silt. Its encountered thickness was about five feet. The SPT N-value in this soil was 13 bpf, indicating a medium dense condition. Percent fines testing indicated a fines content of 18 percent. Based on visual examination and laboratory testing, this soil was classified as silty sand (“SM”).

Beneath the medium dense silty sand was light greenish gray fine sand that contained silt. Its encountered thickness was about two feet. Its actual thickness could not be confirmed because the borehole had been terminated at its intended depth before fully penetrating this soil. The SPT N-value in this soil was 10 bpf, indicating a loose condition. Percent fines testing indicated a fines content of 10 percent. Based on visual examination and laboratory testing, this soil was classified as sand with silt (“SP-SM”).

Groundwater was encountered in this borehole about seven feet below the existing ground surface.
GENERAL COMMENTS ON RECOMMENDATIONS

The following recommendations are based upon a review of the available information, the field and laboratory test results, our understanding of the proposed improvements and our experience with similar projects and subsurface conditions. Soils are natural materials, so variations in composition and other physical characteristics are normal and should be expected. Because of natural variations in depth, composition and consistency of soils and the limited number of borings drilled for this investigation, unsuitable materials and other soils not encountered by the borings may exist beyond each boring location, and should be anticipated. If subsurface conditions encountered during construction differ significantly from those encountered in the borings, those conditions should be reported to us for our observation and comment.

The recommendations contained in this report are based on our understanding that conventional open-cut and trenching and backfilling (“cut-and-cover”) construction methods will be used to install the wet wells and below-grade piping. If plans for the proposed construction change from those discussed in this report, we request the opportunity to review our recommendations and amend them as needed to accommodate those changes. We recommend a review of the project plans and geotechnical-related specifications by this firm to ensure that the geotechnical-engineering recommendations presented in this report are properly interpreted and presented in those documents.

GENERAL ASSESSMENT OF ENCOUNTERED SOILS

As discussed in the SUBSURFACE CONDITIONS section of this report, the soils encountered by the test borings were mostly very loose to loose sands that contained varying amounts of silt and clay. Boring 54-1 encountered sandy clay between 13 feet and 18 feet below the existing ground surface. Groundwater was encountered between three feet and about seven feet below the existing ground surface in the boreholes, and about three feet below the ground surface in the CPT soundings.

The soils penetrated by the CPT soundings at Lift Station 28 behaved like “sand to silty sand,” “silty sand to sandy silt,” “sandy silt to clayey silt”, “clayey silt to silty clay,” and “silty clay to clay” using the soil behavior type classifications developed by Robertson et al. (1986). Three anomalies were observed in soundings CPT-1 and CPT-2, i.e., the sounding outside the north perimeter fence and the sounding immediately to the east of the northeastern corner of the service building. No anomaly was observed in sounding CPT-3, which was about ten feet south of CPT-2.

The shallow anomalies in CPT-1 and CPT-2 were characterized as “sensitive fine-grained,” “clayey silt to silty clay,” “soft clays to organic,” while the deeper anomaly in CPT-2 was characterized as “clayey silt to silty clay,” “sandy silt to clayey silt,” and soft/loose - possible voids” using the Robertson et al (1986) classifications. All three anomalies were associated with tip resistance values lower than 8 tsf, which would cause soil responses similar to clayey soils, organic soils, or possible voids, even though the side friction values did not change appreciably.
As mentioned, planned improvements for the lift stations included new, precast-concrete wet wells that are likely to be between 16 feet and 24 feet deep, and bear between 20 feet and 30 feet below the existing ground surface. New driveways are anticipated, and the existing single-story service building at Lift Station 28 will be renovated.

In general, the soil types and conditions encountered in the borings are suitable for construction of the planned improvements. However, the soils with low resistance values adjacent to the building at Lift Station should be addressed if the City intends to preserve that building. Recommendations for that purpose are provided in the BUILDING FOUNDATION STABILIZATION section of this report. In addition, very loose to loose soils near the anticipated bearing level for the wet wells should be densified to provide adequate support for those structures. Recommendations for that purpose are provided in the SUPPORT OF BURIED STRUCTURES section of this report.

The contractor should expect some difficulty when compacting silty sands, clayey sands and clays, because these soils may retain excess moisture and hamper below-grade construction activity. Excavation dewatering systems should be carefully installed, operated and monitored to ensure that below-grade work areas are stable and dry. Difficult excavation should be anticipated in medium dense to dense soils and clayey soils. Soils beneath the planned piping, slab-on-grade foundations, and driveways will need to be densified to provide adequate support, in accordance with the recommendations in the appropriate EARTHWORK sections of this report.

GENERAL COMMENTS ON BEARING PRESSURE

The vertical design load of a structure distributed over the area of its foundation is known as “gross bearing pressure.” Excavating to install a buried structure reduces the stress on the intended bearing area by the weight of the soil (“overburden”) that is removed. The actual stress increase induced by the structure in the bearing soils is the difference between gross bearing pressure and overburden stress and is known as the “net bearing pressure.” Structural design of a foundation is based on gross bearing pressure, while settlement and bearing capacity are based on net bearing pressure.

Bearing capacity is the foundation bearing pressure that would induce a sudden, shear failure in the underlying soils. It is a function of the size and depth of the foundation and the properties of the bearing soils. Bearing capacity failure is generally not considered a concern for large footings, mat foundations, foundations bearing more than four feet below the ground surface, or foundations bearing in medium dense to very dense soils.

BUILDING FOUNDATION STABILIZATION

As discussed in the SUBSURFACE CONDITIONS of this report, soils with low tip resistance were encountered in sounding CPT-1, which was closest to the side of the building where the structural distress was observed. Those soils extended to a depth of about six feet, which would be within the likely zone of influence of the building foundations. Because of the localized nature of the distress,
the low foundation load that is expected from this structure and the presence of more competent soils about ten feet below the ground surface, we recommend underpinning the foundations on the side of the building where the distress was observed. This can be accomplished at reasonable cost and within a short time frame by a contractor specializing in this type of work. The City should select a qualified and experienced contractor who can include with their estimate the engineering analysis needed to substantiate the remedial work. The City should review that analysis before authorizing the contractor to mobilize to the site.

LIFT STATION WET WELL DESIGN

As discussed earlier in this report, the replacement wet wells for the lift stations are expected to bear between 20 feet and about 30 feet below the ground surface. For design of the wet-well walls, the groundwater should be assumed to be level with existing grade. Under those conditions, we recommend a saturated soil unit weight of 120 pounds per cubic foot (pcf), soil friction angle of 30 degrees and lateral earth pressure coefficient of 0.5. That coefficient represents the “at-rest” condition because enclosed structures like wet wells tend to be self-bracing, and may not allow the soil to displace to the extent needed to attain the active condition. The lateral earth pressure coefficient of 0.5 should be applied to loads on the ground surface around the wet well, including any nearby shallow foundations and incidental vehicular traffic. In the absence of specific load information, incidental traffic should be represented by a uniformly distributed vertical load of 250 psf. If the groundwater level is assumed to be at the ground surface, the lateral earth pressure induced by the soil only may be represented by an equivalent fluid pressure of 29 pcf for structural design purposes only. The unit weight of water should be added to that value in order to represent the full lateral load being imposed by the saturated soils on the wet well walls.

Wet wells should be designed to ensure that there is no uplift when they are empty. Uplift resistance should be derived from the overall weight of the empty structures, to which thick concrete bases will probably be fitted to provide sufficient mass. The buoyant weight of backfill resting on any parts of the foundations projecting horizontally beyond the side walls may be used to augment the uplift resistance. However, side friction against the exterior walls of the wet well should not be considered as contributing to uplift resistance, despite the large surface area in contact with the soil.

The worst-case loading condition on the wet well foundations was assumed to occur during hydro-testing, when the structures are full of water but not backfilled and the groundwater level has been lowered to at least two feet below the lowest foundation bearing surface. Using the assumed wet-well depth discussed earlier in this report and a thick concrete base to enhance the uplift resistance, we estimated a gross bearing pressure of about 2,700 psf. Excavating soils to the depth needed to install the clarifier foundations should reduce the overburden stress by about 1,700 psf to 1,900 psf, which would result in an estimated net bearing pressure between 800 psf and 1,000 psf.

For the purposes of settlement analysis, the wet well bases were treated as circular footings. Short-term and long-term settlements were estimated using a net bearing pressure of 1,000 psf (to model the hydro-testing condition described above) and soil properties developed from empirical
correlations with the SPT N-values. Those analyses yielded estimated short-term settlement of about one inch. Significant long-term settlement is not anticipated because of the generally granular nature of the encountered soils. Differential settlement was not considered because of the small area of the wet-well foundations.

Based on those results, it is our opinion that the wet wells can be supported on spread footings or mat foundations, provided the recommendations in the EARTHWORK FOR BELOW-GRADE CONSTRUCTION section of this report are followed.

**PIPELINE DESIGN**

A minimum modulus of soil reaction (E’) value of 1,000 pounds per square inch (psi) may be used for force main pipeline design provided the earthwork, compaction and subgrade preparation recommendations described in the EARTHWORK FOR BELOW-GRADE CONSTRUCTION section of this report are implemented.

**SUPPORT FOR BURIED STRUCTURES**

Manholes, thrust blocks, anchor blocks and other underground structures should be supported on natural soils or backfill compacted as recommended in the EARTHWORK FOR BELOW-GRADE CONSTRUCTION section later in this report. Soils compacted to that condition should support bearing pressures up to 1,500 pounds per square foot (psf) with total settlements less than an inch.

**UPLIFT RESISTANCE**

All buried pipes and structures should be designed to resist hydrostatic pressure corresponding to the design high groundwater level. Uplift resistance calculations should consider the weight of the structure, the weight of any soils directly above the structure and the weight of backfill over any parts of the foundation that project horizontally beyond the side walls. Side friction resistance along the walls should not be considered.

**SOIL RESISTANCE TO HORIZONTAL PIPELINE FORCES**

Changing fluid pressure inside a pipeline can induce horizontal forces at junctions with buried structures and in locations where the pipe changes direction. Those forces can cause the pipe to move uncontrollably and eventually lead to distress, so anchor blocks or thrust blocks are typically provided to restrain the pipe. Those blocks resist horizontal forces by virtue of their mass as well as the ability to mobilize the shear resistance of the soil beneath their bases and the passive resistance of the soil in contact with their vertical faces.
In order to provide effective resistance, soils in contact with anchor blocks or thrust blocks should be in a medium dense to dense condition. Naturally loose soils (and all fill or backfill materials) should be compacted as recommended in the EARTHWORK FOR BELOW-GRADE CONSTRUCTION section later in this report to at least two feet below the base of any block or structure and at least five feet beyond its vertical face in contact with the soil. The soils should be continuous with no voids or other discontinuities.

Shear resistance beneath the base of any block or structure may be estimated using the following expression:

\[
S = \frac{(W + \gamma_s A H_t - U) \tan (0.67 \phi)}{FS_b}
\]

where

- \( S \) = allowable shear resistance, in pounds
- \( W \) = total weight of the block, in pounds
- \( \gamma_s \) = unit weight of the soil above the block, in pounds per cubic foot
- \( A \) = area of base of structure, in square feet
- \( H_t \) = depth from ground surface to the top of the block, in feet
- \( U \) = total uplift force, in pounds
- \( \phi \) = soil friction angle (30 degrees typically assumed)
- \( FS_b \) = desired factor of safety for base shear (1.5 typically assumed)

The unit weight for compacted soil in central Florida is often estimated to be about 110 pounds per cubic foot (pcf) for moist soil and about 120 pcf for saturated soil.

Passive soil resistance against the face of any block or structure may be calculated conventionally using the estimated soil properties and the desired factor of safety for passive resistance. Surcharges, traffic loads and the weight of construction equipment should not be considered in these analyses.

**SLAB-ON-GRADE FOUNDATIONS**

As discussed earlier in this report, proposed above-ground piping will be supported on slab-on-grade foundations bearing near existing grade at the lift station sites. Detailed structural information was not available at the time, so a gross bearing pressure (the increase in vertical pressure induced in the soil beneath a structure by the total weight on its foundation) of 500 psf was assumed for settlement analysis of the slab-on-grade foundations.

Potential settlement was calculated by applying the gross bearing pressure to the soil stratification developed from the boring logs. The foundations were assumed to bear on compacted soils. A ten-foot-square foundation size was assumed in order to establish a zone of influence beneath each foundation. Engineering properties were then estimated for each soil type within the anticipated zone of foundation influence using empirical correlations with the SPT N-values.
The result of the settlement analysis indicated that the slab-on-grade foundations should settle less than an inch under the assumed bearing pressure. Since the actual bearing pressure is expected to be lower than the assumed value, the actual settlement also should be less. Measurable long-term foundation settlement is not expected because fine-grained soils and plastic soils were not encountered within the anticipated zones of influence of the foundations.

**EARTHWORK FOR BELOW-GRADE CONSTRUCTION**

All below-grade construction should be conducted in accordance with the recommendations for excavation safety and groundwater control presented later in this report. Below-grade construction is likely to require temporary excavation support systems to withstand the anticipated lateral loads and limit unacceptable movement of surrounding soils and adjacent structures. Dewatering will probably be needed in order to establish and maintain dry, stable, safe, below-grade work areas.

Careful attention must be paid to the selection, installation, operation, monitoring, maintenance and removal of temporary excavation support systems. They should provide sufficient working room for anticipated below-grade activities such as installation of formwork and compaction of backfill. Temporary excavation support systems should be removed so as not to disturb completed structures, the backfill nor adjacent structures. The contractor should prepare contingency plans so that the cause(s) of any observed distress to excavation support systems, surrounding soils, or adjacent structures can be identified promptly and accurately, and addressed decisively.

Pavement materials, grass and other vegetation, roots, topsoil or any other unsuitable materials within the limits of the proposed construction should be removed and either discarded or stockpiled away from the immediate work areas for reuse as appropriate, possibly as landscaping material. Any organic materials encountered deeper below the ground surface should be treated in a similar fashion.

Conventional construction equipment should be able to dig excavations for the anticipated lift station improvements. However, the some medium dense soils and clayey sands may not be easy to excavate. In addition, roots, organic materials, debris, dense to very dense soils and cemented soils are also possible and should be expected, even though they were not encountered in the borings.

The excavations should be dug to the depths and widths needed for installation of the wet well, valve vault, piping, excavation support systems, and any below-grade equipment or materials that may be needed. This work should be closely supervised to ensure that excavations are not being over-dug and that the bearing soils are not being disturbed. Any soft, loose or muddy materials should be carefully and completely removed to expose uniform, undisturbed soil.

Below-grade concrete foundations need uniform support to function effectively, even when lightly loaded. Exposed subgrade soils at the bearing depths should be examined and probed by a geotechnical engineer or qualified representative to locate soft or yielding areas, hard spots or other non-uniform conditions. Non-uniform conditions should be treated as directed by the City of Orlando on-site representative in consultation with the examining geotechnical engineer.
Exposed subgrade soils at the bottoms of excavations for the valve vaults and pipes should be thoroughly and uniformly compacted to achieve not less than 95 percent of the maximum dry density obtained by the Modified Proctor method (ASTM D 1557) to a depth at least one foot below subgrade level. Because of the sandy nature of the soils, this work should be done just before foundation concrete is placed to reduce the risk of disturbance. Immediately after subgrade soils have been adequately compacted, a thin layer of non-structural concrete (a “mud mat”) may be placed to protect the bearing soils from disturbance and to provide a stable, durable working surface for the remainder of the construction activity.

Backfill should be placed uniformly on all sides of the proposed valve vaults and manhole in loose lifts approximately eight inches thick before initiating compaction. Each lift should be compacted to not less than 95 percent of the maximum dry density obtained by the Modified Proctor method (ASTM D 1557). Trench backfill for pipes should be placed and compacted in a similar manner.

**EARTHWORK FOR SLAB-ON-GRADE FOUNDATION AND CONCRETE DRIVEWAYS**

All vegetation, topsoil, organic matter and debris within the foundation and driveway areas should be removed to expose clean, undisturbed soils. Clearing and grubbing should extend at least five feet beyond the edges of the foundation and driveway areas and should be expected to a depth of at least one foot.

Cleared ground surfaces should be examined and probed by a geotechnical engineer or designated representative to locate soft or yielding areas, hard spots or other non-uniform conditions. Non-uniform conditions should be treated as directed by the City of Orlando on-site representative in consultation with the examining geotechnical engineer. The cleared ground surfaces should be compacted to not less than 95 percent of the maximum dry density obtained by the Modified Proctor method (ASTM D 1557) to a depth at least two feet.

Minor filling and regrading of the site is anticipated. Fill soils should be placed in uniform lifts approximately 10 to 12 inches in loose thickness and compacted to not less than 95 percent of the maximum dry density obtained by the Modified Proctor method (ASTM D 1557).

**CONSTRUCTION MONITORING**

A program should be established to ensure that excavation, backfilling and compaction operations are conducted in accordance with the project plans and specifications. In-place density testing should be conducted at the bottoms of excavations and during backfilling and compaction operations. Trench subgrade and trench backfill should be tested for adequate compaction at a frequency not less than one test per vertical foot per 300-foot run of pipe. Subgrade soils beneath buried structures should be tested for adequate compaction at a minimum of one location. Backfill around buried structures should be tested for adequate compaction at a frequency not less than one test per vertical foot of backfill. The moisture content of the subgrade soils and backfill soils should
be within the range that will optimize the densification process. The contractor should be prepared to adjust the moisture content and change equipment, procedures and lift thickness as needed at no additional cost to the Owner in order to achieve the recommended compaction.

We also recommended that a geotechnical engineer or the City of Orlando on-site representative be present during construction to confirm that the contractor complies with the intent of the earthwork recommendations and that excavation, excavation support and dewatering activities are executed in accordance with the plans, specifications, and approved submittals.

**REUSE OF EXCAVATED MATERIALS**

It is anticipated that excavated soils will be reused as backfill and fill. Most of the soils encountered in the borings should be suitable for reuse, although some may be too wet or too dry and will require proper moisture conditioning to achieve the recommended degree of compaction. The clayey sands and clays may also be too difficult to work efficiently. Fill and backfill should consist of sand with a fines content less than 12 percent that is free from debris and rubbish, topsoil, mud, muck, peat, stumps, roots, vegetable matter or other unsuitable materials that might decompose or cause excessive settlement. It should be non-plastic and contain no more than five percent by dry weight of organic matter.

Dewatering in preparation for excavation may reduce in-situ water contents to values that are more favorable for earthwork, but the contractor should still expect the soils to be unacceptably wet. The contractor should not to be able to stockpile excavated soils to drain or spread them to dry, because of the limited room that is anticipated at each site. The use of suitable, off-site, “borrow” materials should be considered despite the cost, because they can be used immediately. The resulting increase in earthwork efficiency can help to reduce the duration of below-grade construction, the risk of potential delays and costs associated with inclement weather, and the costs of dewatering.

Because a limited number of borings were drilled for this investigation, variations in consistency and fines content of the uppermost soils are likely, and should be expected. As a result, soil types encountered during excavations may vary. Possible soil types that might be encountered within the planned depths of excavation and general recommendations for their reuse are discussed below for general guidance. These guidelines should not over ride the project specifications. There is the possibility that other soils may be encountered during construction that do not fall into one of the categories discussed below.

**Poorly Graded Sands (SP)**

These soils had fines contents of 5 percent or less, and are commonly referred to as “clean” sands. They are highly desirable for use as fill and backfill in central Florida because they drain freely. That characteristic allows these soils to be placed and compacted even if they have been excavated from below the groundwater level. Satisfactory levels of compaction can be achieved using a wide variety of compaction equipment and across a relatively broad range of moisture contents. Instability or “pumping” should be expected if these soils are compacted near saturation.
Sands with Silt (SP-SM)
These soils consisted of sands with fines contents between 5 percent and 12 percent. Although these soils do not drain as freely as clean sands, they are still quite suitable for use as fill. If excavated from below the groundwater surface, they may have to be stockpiled and allowed to drain (or spread to dry) before being placed as fill. Satisfactory compaction can be achieved using a variety of compaction equipment and across a moderate to wide range of moisture contents. However, efforts should be made during compaction to maintain the moisture content below the optimum. Some instability or “pumping” should also be expected if these soils are compacted near saturation.

Silty Sands (SM), Clayey Sands (SC)
These soils consisted of sands with fines contents higher than 12 percent. They do not drain as well as sands. These soils can be reused successfully as fill, but they will require very close attention to moisture content and careful selection of compaction equipment. Excavated soils of these types can be stockpiled to drain and/or possibly spread to dry before being used as fill. However, the contractor should be discouraged from considering this option because of the limited room that is likely to available at each lift station site. Suitable compaction is generally achieved in these soils only across a narrow range of moisture contents, and this range narrows even further as the fines content increases. Silty sands should be compacted below the optimum moisture content to reduce the potential for moisture-related instability. Soils with more than 20 percent fines should not be used as backfill material.

GROUNDWATER CONTROL

The contractor should expect that groundwater will influence below-grade construction activities. The contract documents should require the contractor to verify groundwater levels before starting construction, and to be responsible for all dewatering, regardless of those groundwater levels. The contractor should be responsible for all aspects of dewatering, regardless of groundwater levels at the time of construction. That responsibility includes not just the installation and operation of an effective dewatering system, but also all permits needed to satisfy applicable environmental regulations, and all groundwater volume and quality monitoring systems.

All excavations and below-grade construction should be conducted in the dry. The contractor should be prepared to lower and maintain the groundwater level at least two feet below the bottoms of all excavations for the duration of below-grade construction activity. Groundwater should be lowered to target levels prior to excavation to minimize the potential for instability of excavations, bottom heave or a quick condition within the excavation. Dewatering systems should be maintained in operation until buried pipes and any buried structures have been placed and completely backfilled in a satisfactory manner such that sufficient dead weight exists on and around buried pipes and structures to prevent uplift. Decommissioning of dewatering systems should be addressed in the contractor’s dewatering submittal.

Water from dewatering pumps should be discharged as far as practically possible away from the work area to prevent return flow or erosion into the excavations. The contractor should also have
submersible pumps ready on site to intercept and remove any localized inflows. The ground surface around excavations should be graded to minimize inflow of runoff.

**EXCAVATION SAFETY**

In accordance with the latest regulations promulgated by the Occupational Safety and Health Administration (“OSHA”), the sides of all excavations more than four feet deep must be sloped or supported to withstand lateral forces exerted by the existing soils. Excavation support systems must also be able to support possible hydrostatic pressures and surcharge loads. For calculating the lateral loads due to the site soils, we recommend a soil unit weight of 125 pounds per cubic foot (pcf) and a lateral earth pressure coefficient of 0.4 for unbraced temporary excavation support systems. This factor should be increased to 0.5 if the system is braced. The same coefficients should be applied to loads on the ground surface from construction equipment and other vehicular traffic in the vicinity of the excavations. Traffic loads should be represented by a uniformly distributed surcharge of 250 pounds per square foot (psf).

All excavations should be kept dry so that work can proceed safely and efficiently. The design of the excavation support systems should be in conjunction with the design of the dewatering systems. As indicated in the GROUNDWATER CONTROL section, groundwater should be maintained at least two feet below the bottom of excavations for the duration of below-grade construction activity. However, dewatering systems can fail, allowing the groundwater to return to its pre-construction level and possibly fill the excavations. Subsequent rapid removal of the water by pumping out the excavation to resume work could create a “rapid drawdown” condition which raises hydrostatic pressure in the soil outside the excavation to a maximum, and reduces soil strength to its minimum. This condition should be analyzed using the design groundwater level.

**DRIVEWAY PAVEMENT**

Pavement for the driveways should be designed in accordance with accepted procedures, using the worst-case loading that can be expected during the design life of the pavement. Shear loading from the tires of turning, heavily-loaded service trucks should be taken into consideration. Portland cement concrete pavement with a minimum thickness of ten inches should be considered for high-use areas. The pavement should be designed using a modulus of subgrade reaction of 150 pounds per cubic inch. Concrete pavements should have control joints spaced as recommended by the Florida Concrete Products Association (or other appropriate agency) to minimize the likelihood of unwanted cracking in response to excessive shrinkage or thermal stress.
LIMITATIONS

This report presents an evaluation of the subsurface conditions on the basis of accepted geotechnical procedures for site characterization. The recovered soil samples were not examined or tested in any way for chemical composition or environmental hazards. The investigation was confined to the zone of soil which is likely to be affected by the proposed construction, and did not address the potential of surface expression of deep geologic activity such as sinkholes. This type of evaluation requires a more extensive range of services than those performed for this study.

Because of the natural limitations inherent in working with the subsurface, a geotechnical engineer cannot predict and address all possible problems. During construction, geotechnical issues not addressed in this report may arise. The bulletin “Important Information About This Geotechnical-Engineering Report” published by the Geoprofessional Business Administration (GBA) is presented in Appendix B to help explain the nature of geotechnical issues. Additional information is presented in Appendix C to discuss the potential concerns and the basic limitations of a typical geotechnical investigation report.
FIGURES
APPROXIMATE LOCATION OF PROJECT

Figure developed from map titled "Potentiometric Surface of the Upper Floridan Aquifer in the St. Johns River Water Management District and Vicinity, Florida, May 2009"

POTENTIOMETRIC SURFACE MAP OF THE UPPER FLORIDAN AQUIFER

201513-2 LIFT STATIONS 28, 54, 60 AND 67 IMPROVEMENTS FIG. 2