GEO HYDRO ENGINEERS

Report of Subsurface Exploration and Geotechnical Engineering Services

Kensington Lift Station
3550 Kensington Road
Dekalb County, Georgia
Geo-Hydro Project Number 191248.20

Prepared for R2T, Inc.
January 28, 2019

Mr. David Preissler, P.E.
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January 28, 2020

Report of Subsurface Exploration
and Geotechnical Engineering Service
Kensington Lift Station
3550 Kingston Road
Dekalb County, Georgia
Geo-Hydro Project Number 191248.20

Dear Mr. Preissler:

Geo-Hydro Engineers, Inc. has completed the authorized subsurface exploration for the above referenced project. The scope of services for this project was outlined in our proposal number 24191.2 dated December 18, 2019.

PROJECT INFORMATION

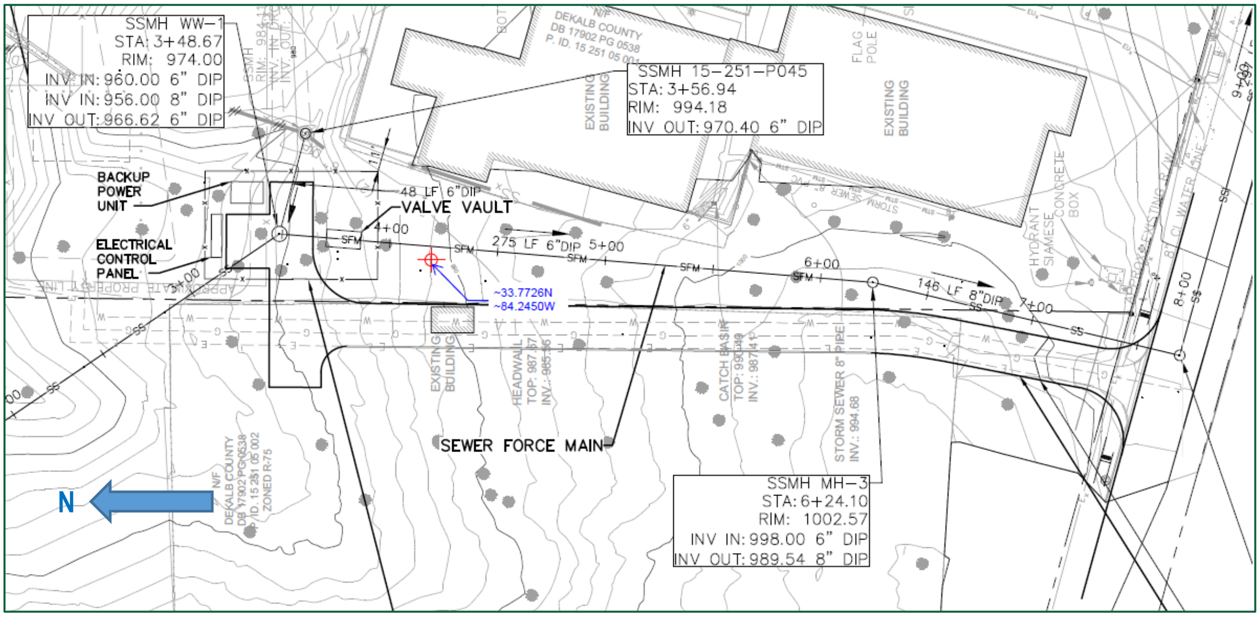
We understand that a new sanitary sewer lift station is planned for the Kensington sewer line extension project. The new lift station will be located at 3550 Kensington Road in Decatur, Georgia.

We understand that the lift station will have a bottom elevation about 30 feet below current grade (elevation 952). The site plan excerpt on the following page shows the planned lift station footprint. The following loading criteria was provided to us as part of the request for proposal for geotechnical exploration:

1. Wet Well
   A. Maximum anticipated bearing pressure: 3,500 psf
   B. Maximum acceptable total settlement: 1 inch
   C. Maximum acceptable differential settlement: ¼ inch

2. Valve Vault
   A. Maximum anticipated bearing pressure: 1,000 psf
   B. Maximum acceptable total settlement: ½ inch
   C. Maximum acceptable differential settlement: ¼ inch

3. Miscellaneous Equipment/Structures at Grade
   A. Maximum anticipated bearing pressure: 500 psf
   B. Maximum acceptable total settlement: 1 inch
   C. Maximum acceptable differential settlement: ½ inch



EXPLORATORY PROCEDURES

The subsurface exploration consisted of one machine-drilled test boring performed at the approximate location shown on Figure 2 included in the Appendix. The test boring was located in the field by Geo-Hydro by measuring angles and distances from existing site reference points. The elevation shown on the test boring record was interpolated from the site plan provided to us. In general, the location and elevation of the boring should be considered approximate.

Standard penetration testing, as provided for in ASTM Dl586, was performed at select intervals in the soil test boring. Soil samples obtained from the drilling operation were examined and classified in general accordance with ASTM D2488 (Visual-Manual Procedure for Description of Soils). Soil classifications include the use of the Unified Soil Classification System described in ASTM D2487 (Classification of Soils for Engineering Purposes). The soil classifications also include our evaluation of the geologic origin of the soils. Evaluations of geologic origin are based on our experience and interpretation and may be subject to some degree of error.

Descriptions of the soils encountered, groundwater conditions, standard penetration resistances, and other pertinent information are provided in the test boring record included in the Appendix.

REGIONAL GEOLOGY

The project site is located in the Southern Piedmont Geologic Province of Georgia. Soils in this area have been formed by the in-place weathering of the underlying crystalline rock, which accounts for their classification as "residual" soils. Residual soils near the ground surface that have experienced advanced weathering frequently consist of red brown clayey silt (ML) or silty clay (CL). The thickness of this surficial clayey zone may range up to roughly 6 feet. For various reasons, such as erosion or local variation of mineralization, the upper clayey zone is not always present.

With increased depth, the soil becomes less weathered, coarser grained, and the structural character of the underlying parent rock becomes more evident. These residual soils are typically classified as sandy micaceous silt (ML) or silty micaceous sand (SM). With a further increase in depth, the soils eventually become quite hard and take on an increasing resemblance to the underlying parent rock. When these materials have a standard penetration resistance of l00 blows per foot or greater, they are referred to as partially weathered rock. The transition from soil to partially weathered rock is usually a gradual one, and may occur at a wide range of depths. Lenses or layers of partially weathered rock are not unusual in the soil profile.

Partially weathered rock represents the zone of transition between the soil and the indurated metamorphic rocks from which the soils are derived. The subsurface profile is, in fact, a history of the weathering process that the crystalline rock has undergone. The degree of weathering is most advanced at the ground surface, where fine-grained soil may be present. Conversely, the weathering process is in its early stages immediately above the surface of relatively sound rock, where partially weathered rock may be found.
The thickness of the zone of partially weathered rock and the depth to the rock surface have both been found to vary considerably over relatively short distances. The depth to the rock surface may frequently range from the ground surface to 80 feet or more. The thickness of partially weathered rock, which overlies the rock surface, may vary from only a few inches to as much as 40 feet or more.

TEST BORING SUMMARY

Boring K-1 initially encountered fill materials extending to a depth of about 3 feet. The fill materials were classified as sandy clay. A standard penetration resistance of 6 blows per foot was recorded in the fill materials.

Beneath the fill materials, the boring encountered residual soils typical of the Piedmont region. The residual soils were classified as clayey sand and silty sand with varying mica content. Standard penetration resistances in the residual soils ranged from 7 to 29 blows per foot.

Immediately after drilling completion, groundwater was encountered in boring K-1 at a depth of approximately 53 feet (approximate elevation 940). For safety reasons, the boring was backfilled with soil cuttings after the groundwater check. It should be noted that groundwater levels will fluctuate depending on yearly and seasonal rainfall variations and other factors, and may rise in the future.

For more detailed descriptions of subsurface conditions, please refer to the test boring record included in the Appendix.

EVALUATIONS AND RECOMMENDATIONS

The following evaluations are based on the information available on the proposed construction, the data obtained from the exploratory boring, and our experience with soils and subsurface conditions similar to those encountered at the site. Because the test boring represents a statistically small sampling of subsurface conditions, it is possible that conditions encountered during construction may be different from those indicated by the boring. In such instances, adjustments to the design and construction may be necessary.

Geotechnical Considerations

The following geotechnical characteristics of the site should be considered for planning and design:

- The test boring indicates generally favorable excavation conditions. Existing fill materials and residual soils should be readily removable within anticipated excavation depths using conventional soil excavation equipment such as loaders and backhoes. Difficult excavation conditions are not expected to be a concern during excavation for the proposed lift station. However, it is important to note that the depth to rock or partially weathered rock may vary drastically over relatively short distances. It would not be unusual to encounter partially weathered rock, rock lenses, rock pinnacles, or boulders within the limits of the site.

- At the time of drilling, groundwater was encountered in boring K-1 at a depth of approximately 53 feet (approximate elevation 940). The excavation required to construct the lift station may extend to approximate elevation 949 to allow the installation of an open-graded aggregate course and construction (cast in place) or installation (precast) of the bottom mat for the lift station. It is important to note that the groundwater level will fluctuate over time depending on local rainfall amounts and other factors and may be encountered at higher elevations. Based on our understanding of the proposed construction, we do not expect groundwater to be a major hindrance for design or construction.

- Based on the results of the test boring, the wet well, valve vault, and miscellaneous equipment/structures at grade can be supported using conventional mat foundations and concrete floors/mats. Intermediate or deep foundations are not required as a matter of design.

- Based on the results of the test boring and following the calculation procedures outlined in the 2018 International Building Code (Chapter 20, ASCE 7-16), the *Site Class* for the site is *D.* The mapped and design spectral response accelerations are as follows: S_S=0.182, S_1=0.085, S_{DS}=0.194, S_{D1}=0.135. Based on the information obtained from the soil test boring, it is our opinion that the potential for liquefaction of the residual soils at the site due to earthquake activity is relatively low.

The following sections provide recommendations regarding these issues and other geotechnical aspects of the project.

Existing Fill Materials

Boring K-1 initially encountered fill materials extending to a depth of about 3 feet. There are several important facts that should be considered regarding fill materials and the limitations of subsurface exploration.

- The quality of existing fill materials can be highly variable and test borings are often not able to detect all of the zones or layers of poor quality fill materials.
- Layers of poor quality fill materials that are less than about 2.5 to 5 feet thick may often remain undetected by soil test borings due to the discrete-interval sampling method used in this exploration.
- The interface between existing fill materials and the original ground surface may include a layer of organic material that was not properly stripped off during the original grading. Depending on its relationship to foundation and floor slab bearing surfaces and pavement subgrade elevations, an organic layer might adversely affect support of footings and floor slabs. If such organic layers are encountered during construction, it may be necessary to "chase out" the organic layer by excavating the layer along with overlying soils.
- The construction budget should include funds for management of poor-quality existing fill materials.
- Subsurface exploration is simply not capable of disclosing all conditions that may require remediation.

General Site Preparation

Trees, topsoil, roots, underbrush, and other deleterious materials should be removed from the proposed construction area. All existing utilities should be excavated and removed unless they are to be incorporated into the new construction. Additionally, site grubbing, and stripping should be performed only during dry weather conditions. Operation of heavy equipment on the site during wet conditions could result in excessive subgrade degradation. All excavations resulting from rerouting of underground utilities and from demolition of below-grade structures such as foundations and basements should be backfilled in accordance with the Structural Fill section of this report.

We recommend that areas to receive structural fill be proofrolled prior to placement of structural fill. Areas of proposed excavation should be proofrolled after rough finished subgrade is achieved. Proofrolling should be performed with multiple passes in at least two directions using a fully loaded tandem axle dump truck weighing at least 18 tons. If low consistency soils are encountered that cannot be adequately densified in place, such soils should be removed and replaced with well compacted fill material placed in accordance with the Structural Fill section of this report. Proofrolling should be observed by Geo-Hydro to determine if remedial measures are necessary.

For budgeting purposes, we suggest considering that approximately 20 percent of the construction area (miscellaneous equipment/structures at grade areas) will require undercutting and recompaction or replacement extending to a depth of about 2 feet below current grades (fill areas) or below target subgrade elevation (cut areas). The suggested stabilization approach is intended only as a tool to estimate a cost associated with ground stabilization. The need for, extent of, location, and optimal method of ground stabilization should be determined by Geo-Hydro at the time of construction based on actual site conditions. The extent and cost of ground stabilization may exceed the suggested budgetary estimate.

During site preparation, burn pits or trash pits may be encountered. All too frequently such buried material occurs in isolated areas which are not detected by the soil test borings. Any buried debris or trash found during the construction operation should be thoroughly excavated and removed from the site.

Excavation Characteristics

The test boring indicates generally favorable excavation conditions. Existing fill materials and residual soils should be readily removable within anticipated excavation depths using conventional soil excavation equipment such as loaders and backhoes. Difficult excavation conditions are not expected to be a concern during excavation for the proposed lift station. However, it is important to note that the depth to rock or partially weathered rock may vary drastically over relatively short distances. It would not be unusual to encounter partially weathered rock, rock lenses, rock pinnacles, or boulders within the limits of the site.

Excavation of partially weathered rock typically requires large equipment capable of ripping. Excavation of partially weathered rock from trench excavations is often impractical due to the leverage required to pre-loosen the material. We expect impact hammers to be necessary to remove partially weathered rock from trench excavations if partially weathered rock it is encountered.

For construction bidding and field verification purposes it is common to provide a verifiable definition of rock in the project specifications. The following are typical definitions of mass rock and trench rock:

- Mass Rock: Material which cannot be excavated with a single-tooth ripper drawn by a crawler tractor having a minimum draw bar pull rated at 56,000 pounds (Caterpillar D-8K or equivalent), and occupying an original volume of at least one cubic yard.

- Trench Rock: Material occupying an original volume of at least one-half cubic yard which cannot be excavated with a hydraulic excavator having a minimum flywheel power rating of 123 kW (165 hp); such as a Caterpillar 322C L, John Deere 230C LC, or a Komatsu PC220LC-7; equipped with a short tip radius bucket not wider than 42 inches.

The foregoing definitions are based on large equipment typically utilized for mass grading. Excavations for foundation and underground utilities are often performed with smaller equipment such as rubber-tired backhoe/loaders or even mini-excavators. Small equipment may encounter difficult excavation conditions, and contractors will often request additional payment for mobilizing larger equipment than that which was anticipated during preparation of their construction bid. The amount of additional compensation, if any, and the minimum equipment size necessary to qualify for any additional compensation should be defined before the start of construction

Reuse of Excavated Materials

Fill materials containing deleterious materials, organics, or debris cannot be used as structural fill and should be removed from the site. Based on the results of the test boring, the residual soils and any debris-free fill materials on site appear suitable for reuse as structural fill. Depending on rainfall levels near the time of construction, fill materials and residual soils may have moisture contents above or below optimum as determined by the standard Proctor test (ASTM D698). Adding water or drying soil may be necessary to achieve proper compaction.

It is important to establish as part of the construction contract whether soils having elevated moisture content will be considered suitable for reuse. We often find this issue to be a point of contention and a source of delays and change orders. From a technical standpoint, soils with moisture contents wet of optimum as determined by the standard Proctor test (ASTM D698) can be reused provided that the moisture is properly adjusted to within the workable range (approximately +/- 3 percent of the optimum moisture content). From a practical standpoint, wet soils can be very difficult to dry in small or congested sites, particularly in the winter and spring, and such difficulties should be considered during planning and budgeting. A clear understanding by the general contractor and grading subcontractor regarding the reuse of excavated soils will be important to avoid delays and unexpected cost overruns.

Groundwater

At the time of drilling, groundwater was encountered in boring K-1 at a depth of approximately 53 feet (approximate elevation 940). The excavation required to construct the lift station may extend to approximate elevation 949 to allow the installation of an open-graded aggregate course and construction of the bottom mat for the lift station. It is important to note that the groundwater level will fluctuate over time depending on local rainfall amounts and other factors and may be encountered at higher elevations. Based on our understanding of the proposed construction, we do not expect groundwater to be a major hindrance for design or construction.

Typically, dewatering should be performed to maintain the groundwater level at least 2 to 3 feet below the lowest prevailing excavation depth. We recommend that the project specifications require the use of dewatering as necessary, and dictate the result of the dewatering operation (performance specification). The contractor may then implement a technique or combination of techniques appropriate for the actual field conditions encountered. Exhibit "A" as follows provides a minimum guide specification that may be used to develop a dewatering performance specification suitable for this project.

EXHIBIT "A"

Minimum Guide Specification for Dewatering

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NOTE: The following specifications are for use as a guide for development of actual specifications. The guide is not intended for direct use as a construction specification without modifications to reflect specific project conditions.

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Control of groundwater shall be accomplished in a manner that will preserve the strength of the foundation soils, will not cause instability of the excavation slopes, and will not result in damage to existing structures. Where necessary to these purposes, the water level shall be lowered in advance of excavation, utilizing trenches, sumps, wells, well points or similar methods. The water level, as measured in piezometers, shall be maintained a minimum of 2 feet below the prevailing excavation level. Open pumping from sumps and ditches, if it results in boils, loss of soil fines, softening of the ground or instability of slopes, will not be permitted. Wells and well points shall be installed with suitable screens and filters so that continuous pumping of soil fines does not occur. The discharge shall be arranged to facilitate collection of samples by the Engineer.

Adapted from Construction Dewatering - A Guide to Theory and Practice, John Wiley and Sons.

Earth Slopes

Temporary construction slopes should be designed in strict compliance with OSHA regulations. The exploratory boring indicates that most soils at the site are Type B as defined in 29 CFR 1926 Subpart P, which dictates that temporary construction slopes be no steeper than 1H:1V for excavation depths of 20 feet or less. Temporary construction slopes should be closely observed on a daily basis by the contractor's "competent person" for signs of mass movement: tension cracks near the crest, bulging at the toe of the slope, etc. The responsibility for excavation safety and stability of temporary slopes should lie solely with the contractor.

We recommend that extreme caution be observed in trench excavations. Several cases of loss of life due to trench collapses in Georgia point out the lack of attention given to excavation safety on some projects. We recommend that applicable local and federal regulations regarding temporary slopes, and shoring and bracing of trench excavations be closely followed.

Formal analysis of slope stability was beyond the scope of work for this project. Based on our experience, permanent cut or fill slopes should be no steeper than 2H:1V to maintain long term stability and to provide ease of maintenance. The crest or toe of cut or fill slopes should be no closer than 10 feet to any foundation.

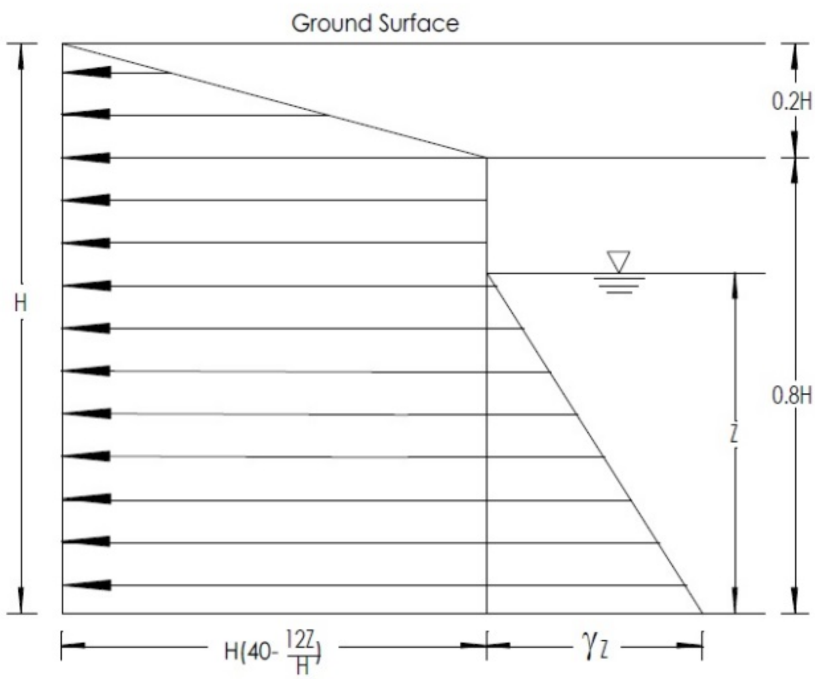
The crest or toe should be no closer than 5 feet to the edge of any pavements. Erosion protection of slopes during construction and during establishment of vegetation should be considered an essential part of construction.

Temporary Excavation Bracing

Because of the proximity of property boundaries and the overall project setting, it may not be feasible to construct temporary slopes at a gradient of 1H:1V. Temporary shoring systems such as internally-braced sheet piling or internally-braced soldier pile and wood lagging can be used. The primary disadvantage of internally braced excavation support systems is the obstruction caused by the bracing. The selection and design of a specific excavation bracing system should be left to the contractor. Geo-Hydro and R2T should review the proposed excavation bracing system.

Earth Pressure - Small Below-Grade Structures

Based on our experience, the lateral earth pressure distribution for the walls of relatively small and rigid below-grade structures, such as vaults, wets wells, etc., can be approximated by a braced wall configuration. The walls of these structures may be designed for the pressure distribution shown on the adjacent diagram:



Where: H = Total depth of wall below the ground surface, feet
Z = Height of groundwater level above the bottom of the wall, feet
γ = Unit weight of water (62.4 pcf)
pressure units are psf

Earth Pressure - Conventional Reinforced Concrete Retaining Walls

Three earth pressure conditions are generally considered for retaining wall design: "at rest", "active", and "passive" stress conditions. Retaining walls which are rigidly restrained at the top and will be essentially unable to rotate under the action of earth pressure should be designed for "at rest" conditions. Retaining walls which can move outward at the top as much as 0.5 percent of the wall height should be designed for "active" conditions. For the evaluation of the resistance of soil to lateral loads the "passive" earth pressure must be calculated. It should be noted that full development of passive pressure requires deflections toward the soil mass on the order of l.0 percent to 4.0 percent of total wall height.

Earth pressure may be evaluated using the following equation:

$$p_h = K\,(D_w Z + q_s) + W_w(Z\text{-}d)$$

where: p_h = horizontal earth pressure at any depth below the ground surface (Z).
W_w = unit weight of water
Z = depth to any point below the ground surface
d = depth to groundwater surface
D_w = wet unit weight of the soil backfill (depending on borrow sources). The wet unit weight of most residual soils may be expected to range from approximately 115 to 125 pcf. Below the groundwater level, D_w must be the buoyant weight.
q_s = uniform surcharge load (add equivalent uniform surcharge to account for construction equipment loads)
K = earth pressure coefficient as follows (*based on an angle of internal friction of* ***30*** *degrees*):

Earth Pressure Condition	Coefficient
At Rest (K_o)	0.5
Active (K_a)	0.33
Passive (K_p)	3.0

The groundwater term, $W_w(Z\text{-}d)$, should be used if no drainage system is incorporated behind retaining walls. If a drainage system is included which will not allow the development of any water pressure behind the wall, then the groundwater term may be omitted. The development of excessive water pressure is a common cause of retaining wall failures. Drainage systems should be carefully designed to ensure that long term permanent drainage is accomplished.

The above design recommendations are based on the following assumptions:

- Horizontal backfill
- 95 percent standard Proctor compactive effort on backfill (ASTM D698)
- No safety factor is included

For convenience, equivalent fluid densities are frequently used for the calculation of lateral earth pressures. For "at rest" stress conditions, an equivalent fluid density of 63 pcf may be used. For the "active" state of stress an equivalent fluid density of 42 pcf may be used. These equivalent fluid densities are based on the assumptions that drainage behind the retaining wall will allow *no* development of hydrostatic pressure; that native silty sands or sandy silts will be used as backfill; that the backfill soils will be compacted to 95 percent of standard Proctor maximum dry density; that backfill will be horizontal; and that no surcharge loads will be applied.

For analysis of sliding resistance of the base of a concrete cast-in-place retaining wall or foundation, the coefficient of friction may be taken as 0.4 for the residual soils at the project site. This is an ultimate value, and an adequate factor of safety should be used in design. The force which resists base sliding is calculated by multiplying the normal force on the base by the coefficient of friction. Full development of the frictional force could require deflection of the base of roughly 0.l to 0.3 inches.

Structural Fill

We provide the following recommendations for any structural fill that may be required at the site.

Materials selected for use as structural fill should be free of organic matter, waste construction debris, and other deleterious materials. The material should not contain rocks having a diameter over 4 inches. It is our opinion that the following soils represented by their USCS group symbols will typically be suitable for use as structural fill and are usually found in abundance in the Piedmont Region: (SM), (ML), and (CL). The following soil types are typically suitable but are not abundant in the Piedmont Region: (SW), (SP), (SC), (SP-SM), and (SP-SC). The following soil types are considered unsuitable: (MH), (CH), (OL), (OH), and (Pt).

Laboratory Proctor compaction tests and classification tests should be performed on representative samples obtained from the proposed borrow material to provide data necessary to determine acceptability and for quality control. Soils having a standard Proctor maximum dry density of less than 90 pcf should be considered unsuitable, unless laboratory evaluations of their stress-strain characteristics indicate that they will perform acceptably. The moisture content of suitable borrow soils should generally not be more than 3 percentage points above or below optimum at the time of compaction. Tighter moisture limits may be necessary with certain soils.

It is possible that highly micaceous soils could be utilized for structural fill material. The use of such materials will require very close attention to quality control of moisture content and density. Additionally, it is our experience that highly micaceous soils tend to rut under rubber-tired vehicle traffic. Continuous maintenance of areas subjected to construction traffic is typically required until construction is completed.

Suitable fill material should be placed in thin lifts. Lift thickness depends on type of compaction equipment, but in general, lifts of 8 inches loose measurement are recommended. The soil should be compacted by heavy compaction equipment such as a self-propelled sheepsfoot roller. Within small excavations such as in utility trenches or behind retaining walls, we recommend the use of "wacker packers" or "Rammax" compactors to achieve the specified compaction. Loose lift thicknesses of 4 to 6 inches are recommended in small area fills.

GEO HYDRO ENGINEERS

We recommend that structural fill be compacted to at least 95 percent of the standard Proctor maximum dry density (ASTM D698). The upper 12 inches of floor slab or mat foundation subgrade soils should be compacted to at least 98 percent of the standard Proctor maximum dry density (ASTM D698). The upper 12 inches of pavement subgrades should be compacted in accordance with Georgia DOT requirements to at least 100 percent of the standard Proctor maximum dry density (ASTM D698). Geo-Hydro should perform density tests during fill placement.

Foundation Design

Wet Well

The wet well structure will benefit from significant overburden relief. Once the lift station structure is constructed and in operation, there will be a net decrease in pressure applied at bearing elevation. A properly prepared bearing surface should result in little or no net settlement (less than 1 inch total and no more than ¼ inch planar tilt). We recommend a prepared subgrade consisting of a 12-inch thick course of open-graded stone such as #57 stone meeting Georgia DOT specifications for gradation. An allowable foundation bearing pressure of 3,500 psf can be used for design.

Remedial measures should be based on actual field conditions. Improving subgrade conditions in mat foundation excavations is generally limited to removing soft soils from the mat area and replacing the soft soils with crushed stone materials. The wet well excavation should be evaluated by Geo-Hydro prior to placement of any crushed stone.

Valve Vault

After general site preparation and site grading have been completed in accordance with the recommendations of this report, it is our opinion that the valve vault can be supported using conventional reinforced concrete mat foundations/floors. An allowable soil bearing pressure of 2,000 psf can be used for design.

The valve vault is likely to be completely or partially underlain by backfill materials that will be placed around the wet well structure. To meet the settlement tolerances required for the valve vault (½ inch total and ¼ inch differential), it is imperative that the backfill around the wet well comply with the recommendations presented in the Structural Fill section of this report.

Remedial measures should be based on actual field conditions. However, in most cases we expect the use of the stone replacement technique to be the primary remedial measure. Stone replacement involves the removal of soft or loose soils, and replacement with well-compacted graded aggregate base (GAB) meeting Georgia Department of Transportation specifications for gradation.

Miscellaneous Equipment/Structures at Grade

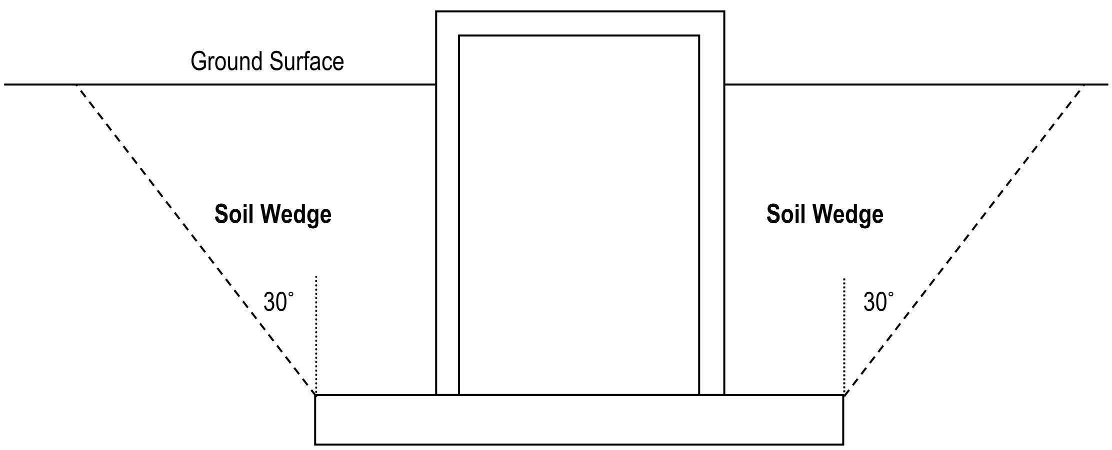
After general site preparation and site grading have been completed in accordance with the recommendations of this report, it is our opinion that miscellaneous equipment and small structures can be supported using conventional reinforced concrete mat foundations or conventional shallow foundations. An allowable soil bearing pressure of 2,000 psf can be used for design. Footings should bear at a minimum depth of 12 inches below the prevailing exterior ground surface elevation to help avoid potential problems due to frost heave.

It is likely that ancillary structures will be completely or partially underlain by backfill materials that will be placed around the wet well structure. To meet the settlement tolerances required (1 inch total and ½ inch differential), it is imperative that the backfill around the wet well comply with the recommendations presented in the Structural Fill section of this report

Remedial measures should be based on actual field conditions. However, in most cases we expect the use of the stone replacement technique to be the primary remedial measure. Stone replacement involves the removal of soft or loose soils, and replacement with well-compacted graded aggregate base (GAB) meeting Georgia Department of Transportation specifications for gradation.

Uplift Design

In general, structures which will have bottom elevations below current or future groundwater levels or are in flood-prone areas should be designed to resist potential buoyant uplift forces. In some instances, uplift forces may be resisted by the dead weight of the structure itself. Where necessary, the dead weight of the structure can be effectively increased by extending the reinforced concrete mat foundation beyond the walls of the structure. This mobilizes additional weight of soil to increase the effective dead weight of the structure. The effective weight of a "wedge" of soil backfill as depicted in the following sketch can be used to calculate additional uplift resistance from soil backfill.



If this approach to resist buoyant forces is not sufficient, it will be necessary to install anchors at the bottom of the structure to supplement uplift resistance. The scope of this exploration is insufficient to develop design recommendations for soil/rock anchors.

Seismic Design

Based on the results of the test boring and following the calculation procedures outlined in the 2018 International Building Code (Chapter 20, ASCE 7-16), the Site Class for the site is D. The mapped and design spectral response accelerations are as follows: S_S=0.182, S_1=0.085, S_{DS}=0.194, S_{D1}=0.135.

Based on the information obtained from the soil test boring, it is our opinion that the potential for liquefaction of the residual soils at the site due to earthquake activity is relatively low.

Pavement Design Parameters

Based on the soils expected at subgrade elevation and contingent on a properly prepared subgrade meeting GDOT requirements for pavement construction, we recommend using a California Bearing Ratio (CBR) of 5 and a modulus of subgrade reaction (k) of 120 pci for design of flexible and rigid pavements, respectively.

Flexible Pavement Design

Based on our experience with similar projects, assuming standard pavement design parameters, and contingent upon proper pavement subgrade preparation, we recommend the following pavement sections:

Truck Traffic Areas

Material	Thickness (inches)
Asphaltic Concrete 9.5mm Superpave	2
Asphaltic Concrete 19mm Superpave	2
Graded Aggregate Base (GAB) (Base Course)	8
Subgrade compacted to at least 100% standard Proctor maximum dry density (ASTM D698)	12

Automobile Parking and Automobile Traffic Only

Material	Thickness (inches)
Asphaltic Concrete 9.5mm Superpave	2
Graded Aggregate Base (GAB) (Base Course)	6
Subgrade compacted to at least 100% standard Proctor maximum dry density (ASTM D698)	12

A concrete thickness of 7 inches is recommended for designated truck turn-around areas. Please refer to the Concrete Pavement section of this report for concrete pavement recommendations.

The top 12 inches of pavement subgrade soils should be compacted to at least 100 percent of the standard Proctor maximum dry density (ASTM D698). Scarification and moisture adjustment will likely be required to achieve the recommended subgrade compaction level. Allowances for pavement subgrade preparation should be considered for budgeting and scheduling.

GAB must be compacted to at least 100 percent of the modified Proctor maximum dry density (ASTM D1557).

All pavement construction should be performed in general accordance with Georgia DOT specifications. Proper subgrade compaction, adherence to Georgia DOT specifications, and compliance with project plans and specifications, will be critical to the performance of the constructed pavement.

Concrete Pavement

We recommend concrete thicknesses of 5 inches in automobile parking areas and 6 inches in truck traffic areas. A concrete thickness of 7 inches is recommended for designated truck turn-around areas. A 600-psi flexural strength concrete mix (approximately 4,500 psi compressive strength) with 4 to 6 percent air entrainment should be used. The concrete pavement should be underlain by no less than 5 inches of compacted graded aggregate base (GAB). GAB should be compacted to at least 100 percent the modified Proctor maximum dry density (ASTM D1557). The top 12 inches of soil subgrade should be compacted to at least 100 percent of the standard Proctor maximum dry density (ASTM D698).

The concrete pavement may be designed as a "plain concrete pavement" with no reinforcing steel, or reinforcing steel may be used at joints. Construction joints and other design details should be in accordance with guidelines provided by the Portland Cement Association and the American Concrete Institute.

In general, all pavement construction should be in accordance with Georgia DOT specifications. Proper subgrade compaction, adherence to Georgia DOT specifications, and compliance with project plans and specifications will be critical to the performance of the constructed pavement.

Pavement Design Limitations

The pavement sections discussed above are based on our experience with similar type developments. After traffic information has been developed, we recommend that you allow us to review the traffic data and revise our recommendations as necessary.

Pavement Materials Testing

To aid in verifying that the pavement system is installed in general accordance with the design considerations, the following materials testing services are recommended:

- Density testing of subgrade materials.
- Proofrolling of pavement subgrade materials immediately prior to placement of graded aggregate base (GAB). This proofrolling should be performed the same day GAB is installed.
- Density testing of GAB and verification of GAB thickness. In-place density should be verified using the sand cone method (ASTM D1556).
- Coring of the pavement to verify thickness and density (asphalt pavement only).
- Preparation and testing of beams and cylinders for flexural and compressive strength testing (portland cement concrete only). The total number of beams and cylinders required will depend on the number of concrete placement events necessary to construct the pavement.

We appreciate the opportunity to work with you on this project, and are prepared to provide any additional services you may require. If you have any questions concerning this report or any of our services, please call us.

Sincerely,

GEO-HYDRO ENGINEERS, INC.

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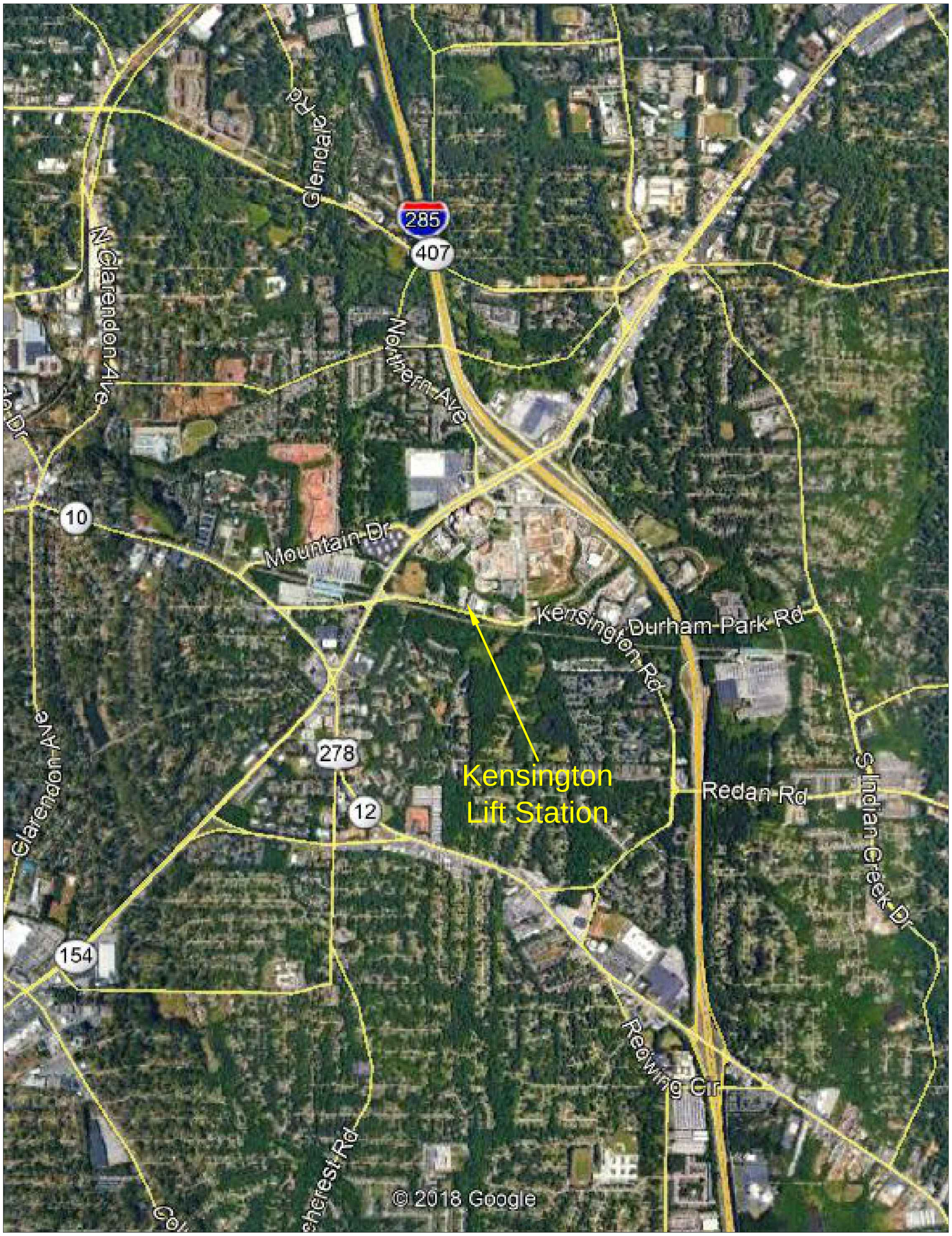
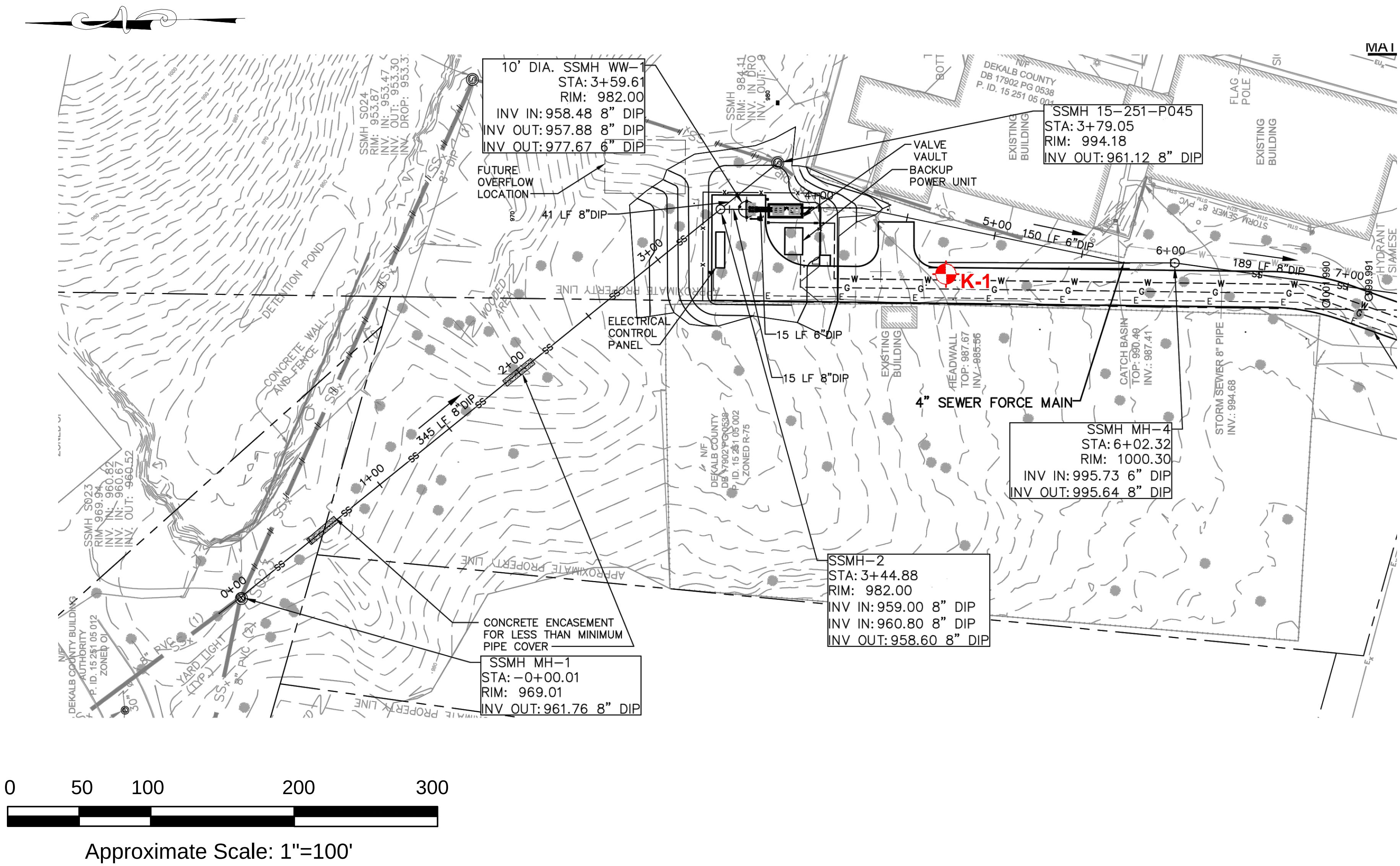
KDJ/LEB/191248.20 Kensington Lift Station leb

APPENDIX



Approximate Scale, Miles

Figure 1: Site Location Plan	Kensington Lift Station 3550 Kensington Road Dekalb County, Georgia Geo-Hydro Project Number 191248.20



LEGEND: Soil Test Boring

Figure 2: Boring Location Plan

Kensington Lift Station
3550 Kensington Road
Dekalb County, Georgia
Geo-Hydro Project Number 191248.20

Symbols and Nomenclature

Symbols

▌	Thin-walled tube (TWT) sample recovered
▯	Thin-walled tube (TWT) sample not recovered
●	Standard penetration resistance (ASTM D1586)
50/2”	Number of blows (50) to drive the split-spoon a number of inches (2)
65%	Percentage of rock core recovered
RQD	Rock quality designation - % of recovered core sample which is 4 or more inches long
GW	Groundwater
▼	Water level at least 24 hours after drilling
▽	Water level one hour or less after drilling
ALLUV	Alluvium
TOP	Topsoil
PM	Pavement Materials
CONC	Concrete
FILL	Fill Material
RES	Residual Soil
PWR	Partially Weathered Rock
SPT	Standard Penetration Testing

Penetration Resistance Results

	Number of Blows, N	Approximate Relative Density
Sands	0-4	very loose
	5-10	loose
	11-20	firm
	21-30	very firm
	31-50	dense
	Over 50	very dense

	Number of Blows, N	Approximate Consistency
Silts and Clays	0-1	very soft
	2-4	soft
	5-8	firm
	9-15	stiff
	16-30	very stiff
	31-50	hard
	Over 50	very hard

Drilling Procedures

Soil sampling and standard penetration testing performed in accordance with ASTM D 1586. The standard penetration resistance is the number of blows of a 140-pound hammer falling 30 inches to drive a 2-inch O.D., 1.4-inch I.D. split-spoon sampler one foot. Rock coring is performed in accordance with ASTM D 2113. Thin-walled tube sampling is performed in accordance with ASTM D 1587.

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K-1

Test Boring Record

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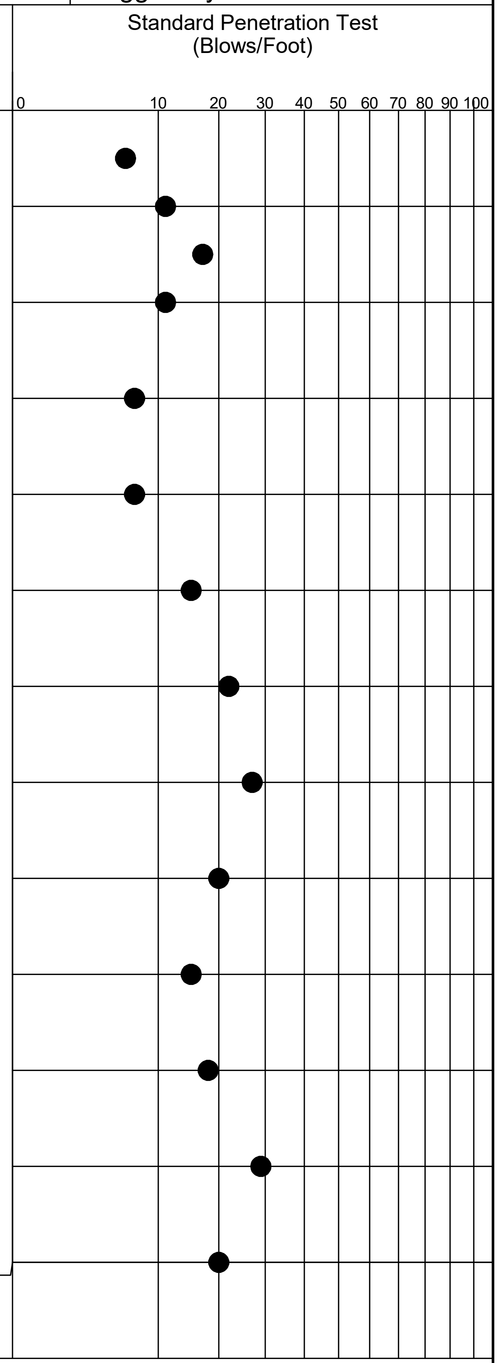
Project: **Kensington Lift Station**		Project No: **191248.20**
Location: **3550 Kensington Road - DeKalb County, Georgia**		Date: **12/23/19**
Method: **HSA- ASTM D1586**	GWT at Drilling: **53 feet**	G.S. Elev: **993**
Driller: **Premier Drilling (Autohammer)**	GWT at 24 hrs: **N/A: Boring Backfilled**	Logged By: **BD**

Elev. (Ft)	Depth (Ft)	GWT	Symbol	Description	N
				Firm brown slightly micaceous fine to medium sandy clay (CL) (FILL)	6
990	5			Firm red slightly micaceous clayey fine to medium sand (SC) (RESIDUUM)	11
985	10			Loose to firm brown and red micaceous silty fine to medium sand (SM)	17, 11, 7, 7, 15
980	15				
975	20				
970	25				
965	30			Firm to very firm gray and brown micaceous silty fine to medium sand (SM)	22, 27, 20, 15, 18, 29, 20
960	35				
955	40				
950	45				
945	50				
940	55	∇			
935	60				
930	65			Boring Terminated at 60 feet	

Standard Penetration Test (Blows/Foot)



Remarks: