

Yellow

Geotechnical Engineering Exploration and Analysis

**Proposed Public Storage Facility
424 Eagle Rock Avenue
West Orange, New Jersey**

Prepared for:

**Public Storage, Inc.
Mount Laurel, New Jersey**

**October 13, 2020
Project No. 9G-2008002**



GILES
ENGINEERING ASSOCIATES, INC.



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GEOTECHNICAL, ENVIRONMENTAL & CONSTRUCTION MATERIALS CONSULTANTS

- Atlanta, GA
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- Milwaukee, WI

October 13, 2020

Public Storage, Inc.
139-A Gaither Drive
Mount Laurel, NJ 08054

Attention: Mr. Frank Caccuro
Vice President of Construction – Eastern Region

Subject: Geotechnical Engineering Exploration and Analysis
Proposed Public Storage Facility
424 Eagle Rock Avenue
West Orange, New Jersey
Project No. 9G-2008002

Dear Mr. Caccuro:

As requested, Giles Engineering Associates, Inc. conducted a *Geotechnical Engineering Exploration and Analysis* for the proposed project. The accompanying report describes the services that were conducted, and it provides geotechnical-related findings, conclusions, and recommendations that were derived from those services.

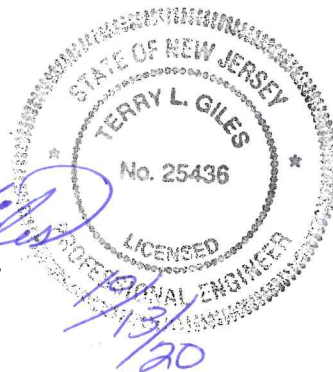
We sincerely appreciate the opportunity to provide geotechnical consulting services for the proposed project. Please contact the undersigned if there are questions about the report, or if we may be of further service.

Very truly yours,

GILES ENGINEERING ASSOCIATES, INC.

Darryl B. Keating
District Manager

Terry L. Giles, P.E.
President & CEO



Distribution: Public Storage, Inc.
Attn: Mr. Frank Caccuro (1 via email: fcaccuro@publicstorage.com)
Attn: Mr. Jarrod Yates (1 via email: jyates@publicstorage.com)

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424 EAGLE ROCK AVENUE
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YELLOW – This project has been given a Yellow designation, from a geotechnical standpoint due to the existing fill and possible fill throughout the site containing cobbles and/or boulders, with some loose areas recommended to be improved by means of surface compaction that may also require some over-excavation. A mat/slab is then recommended in conjunction with spread footings structurally tied to the mat slab as the foundation system. An 8-inch-thick aggregate layer is recommended to be below the mat, serving as a cushion layer. The footings should be over-excavated to a depth of at least 1 foot and replaced with suitable engineered fill, also serving as a cushion layer.



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GEOTECHNICAL ENGINEERING EXPLORATION AND ANALYSIS

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EXECUTIVE SUMMARY

This Executive Summary provides limited geotechnical information regarding the proposed project. Since this Executive Summary is exceedingly abbreviated, it must be read in complete context with the following report ("Report").

Surface and Subsurface Conditions

- Nine test borings (Test Borings 1 thru 9) were originally planned to explore subsurface conditions at the site, but as a result of frequent interference with buried cobbles and/or boulders within the fill an additional seven test borings (Test Borings 1A, 2A, 3A, 3B, 3C, 3D, and 9A) were performed; generally offset in the vicinity of the originally planned boring locations.
- Topsoil was encountered at the surface at one test boring. Asphalt pavement was at the surface at the remaining test borings. A crushed stone base course was observed below the asphalt pavement at eight of the test borings.
- Soil classified as fill and/or possible fill was below the surface materials at each of the test boring locations, extending to depths ranging between 0.5 inches and 14 feet. The fill predominantly consisted of silty sand containing variable amounts of gravel, and also containing cobbles and/or boulders, but also contained silt and clay containing variable amounts of sand and gravel, some also containing cobbles and/or boulders. Organics and/or wood fragments were observed in several samples collected from within the fill and possible fill. Based on the Atterberg limit test results, the tested samples exhibited slight to medium plasticity and low to medium expansive potential. The N-values were variable, but generally indicating a very loose to firm relative density, based on N-values from STSs. Many of the SPTs were elevated as a result of interference with cobbles and/or boulders, and as such are not indicative of relative density of the sampled fill materials.
- Native soil was below the fill and/or possible fill at five of the test borings, and extended to depths ranging between about 13.1 to 24 feet at three test borings, where auger and split spoon refusal were encountered. At two test borings the native soils extended to their respective termination depths. The native soil generally consisted of silty to clayey sand, and silt and lean clay. The native non-cohesive soils generally exhibited firm to very dense relative densities based on N-values from SPTs.
- Basalt bedrock was believed to have been encountered within Test Boring Nos. 1A and 3D (and expected to have been encountered at Test Boring 1), beginning at depths of about 1.0 and 9.8 feet respectively. However, it is possible that large boulders may have been encountered. Rock core samples were recovered from Test Borings 1A and 3D. The core samples of the bedrock were predominantly very slightly to moderately

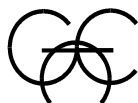


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fractured, partially to moderately healed, hard to very hard based on visual observation and rock classification.

- Water was encountered during drilling at depths ranging between 3 and 7.5 feet at several test borings but not encountered at the remaining test borings when the test borings were conducted. Considering the lack of water encountered and the moisture contents of the sampled soils at some of the test borings it is expected that the groundwater was deeper than the maximum exploration depth when the Geotechnical Subsurface Exploration Program was conducted, but that the encountered water conditions are a result of perched water associated with the adjacent lake to the west and by the granular soils predominant at the site. Encountered water was shallower along the west side of the site (nearest to the existing lake) and appears to be deeper (when encountered) with distance from the lake.

Construction Considerations

- Several of the samples from within the existing fill at the site contained an estimated trace amounts of organics and/or root matter. It is possible that unacceptable amounts of deleterious materials and/or construction debris may be buried within the fill at the site. If existing fill materials containing excessive amounts of deleterious materials are encountered within the proposed building area, it is recommended that these materials be removed and replaced with suitable engineered fill in accordance with the recommendations contained in this report.
- Surface compaction is recommended to improve the existing fill / possible fill for mat/slab support. Vibratory compaction equipment should be used with caution, especially within adjacent to existing structures such as buildings, pavements. In those areas, compaction should be performed using static (non-vibratory) equipment, as vibratory compactors could cause soil to become unstable.

Building Foundation

- The proposed building can be supported by a mat/slab foundation with spread footings, which is understood to be the planned foundation system. The mat slab foundation is recommended to be designed using a 65 pound per cubic inch (pci) *Modulus of Subgrade Reaction* (K_s).
- Before foundation construction and prior to placing additional fill within the building pad, the existing soils are recommended to be improved by surface compaction.
- The mat is recommended to be supported by an 8-inch-thick aggregate layer, also serving as a cushion layer for the mat.
- Spread footings are recommended to be designed using a 3,000 psf maximum, net, allowable soil bearing capacity for foundations bearing on suitable improved existing fill, native soils, or engineered fill. Further, it is recommended that the existing fill be over-excavated to a depth of at least 1 foot and replaced with engineered will (containing no oversized materials). Similarly, intact bedrock and/or large boulders (expected to be encountered at the south end of the proposed building) should also be removed to a depth of at least one foot below planned bottom of footings and replaced with engineered fill.



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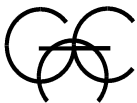
Pavement

- Asphalt-concrete pavement is considered suitable for the parking lot, with proper subgrade preparation as described in the Report. Portland cement concrete pavement is recommended in high-stress areas, such as at entrance/exit aprons, at a trash enclosure, and in areas where trucks will turn or will be parked.
- Depending on the actual pavement elevations, over-excavation and/or mechanical stabilization of unstable soil might be necessary to develop a stable subgrade for pavement support, especially considering the strength characteristics, sensitivity, and moisture conditions of the site soils.

Reuse of Site Soils

- Site soil that does not contain adverse organic content or other deleterious materials, as noted in the Guide Specifications, and that does not contain oversized materials could be used as engineered fill within the building pad and pavement areas. The majority of the on-site soils generally considered suitable for use as engineered fill; however, it is possible that pockets of the existing fill may contain excessive amounts of deleterious materials, and will require sorting through and removing oversized materials.

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GEOTECHNICAL ENGINEERING EXPLORATION AND ANALYSIS

PROPOSED PUBLIC STORAGE FACILITY
424 EAGLE ROCK AVENUE
WEST ORANGE, NEW JERSEY
PROJECT NO. 9G-2008002

1.0 SCOPE OF SERVICES

This report provides the results of the *Geotechnical Engineering Exploration and Analysis* that Giles Engineering Associates, Inc. ("Giles") conducted for the proposed development. The *Geotechnical Engineering Exploration and Analysis* included a Geotechnical Subsurface Exploration Program, Geotechnical Laboratory Services, and Geotechnical Engineering Services. The scope of each service area was narrow and limited, as directed by our client and based on our understanding and assumptions about the proposed project. Service areas are briefly described later.

Geotechnical-related recommendations for design and construction of the foundations for the proposed three-story structure are provided in this report. Geotechnical-related recommendations are also provided for new parking lot pavement. Recommendations regarding site preparation are given, but are also only preliminary as the means and methods of site preparation will depend on factors that were unknown when this report was prepared. Those factors include, but are not limited to, the weather before and during construction, subsurface conditions that are encountered during construction, and finalized details of the proposed development. Environmental consulting services were beyond Giles' authorized scope for this project.

2.0 SITE DESCRIPTION

The project site is located at 424 Eagle Rock Avenue in West Orange, New Jersey. The site is currently developed with a one story brick-faced structure occupied by a bowling alley (understood to be permanently closed); it is not known if the structure contains a basement, but it is expected to have some below-grade areas. The building is surrounded by an asphalt parking lot with parking stalls around the perimeter of the building and lot. Multi-story office spaces and the Eagle Rock Avenue borders the site to the north; a wooded area and then a lake borders the site to the east; a wooded area and then residential homes border the site to the south; and Crystal Lake borders the site to the west. Access to the site is from an access road running south from Eagle Rock Avenue to the west of the office buildings located directly north. The existing building was considered to be in fair condition, with no significant signs of distress or cracking noted, based on cursory observations. The existing pavements were observed to be in fair condition to poor condition with some cracking observed, based on cursory observations.

Based on a review of available historical aerial imagery it appears that the majority of the site (the portion generally extending to the east of the lake) had previously been lower in elevation, and at least partially an extension of this existing lake. As such, it is apparent that the site had been filled.



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The site slopes gently downward from the east to the west with ground surface elevations at the test borings (described later) ranging between about $\pm$ El. 98.9 and $\pm$ El. 101.1 in the vicinity of the existing building and parking lot; and downwards to El. 97.8 at the test boring positioned in the access road; the lake surface was estimated to be 2 to 4 feet below the pavement grades; these elevations are referenced to the assumed elevation of a temporary benchmark (top of a storm drain structure) shown on Figure 1A attached.

3.0 GEOLOGY AND SOILS MAPPING

The subject site is located in the Piedmont physiographic province. The *1998 Bedrock Geologic Map of Central and Southern New Jersey*, available through the United States Geologic Survey indicates that the site is underlain by Orange Mountain Basalt.

A review of the soil survey from the National Cooperative Soil Survey, prepared by the United States Department of Agriculture, Natural Resources Conservation Services indicates that the developed portion of the site is underlain by Urban Land, Bontoon Substratum. The southern edges of the site are mapped as being underlain by Holyoke Silt Loam, very rocky (map unit symbol HokCh) and Yalesville lom, very rocky (map unit symbol YamnBc). The parent material of the site soils mapped to the south are a coarse loamy till derived from basalt. Bedrock is typically very shallow. Urban Land areas are previously developed areas, and generally contain fill.

4.0 PROJECT DESCRIPTION

It is understood that the existing development is planned to be demolished, and a free-standing, three-story structure will be constructed in the area shown on the *Test Boring Location Plan*. Details of the proposed building were not provided, however, based on preliminary information provided by the client (*Preliminary Concept Plan "A", dated June 9, 2020*), also shown in Figure 1B, located in Appendix A, the building is planned to be a three-story structure of about 40,000 gross square feet. The proposed building generally overlies the footprint of the existing building. Based on our experience on similar Public Storage projects, it is assumed that the building will be a steel-frame structure with masonry, concrete, or metal exterior, and a steel bar-joist and metal-deck roof. Interior and perimeter columns will support the structure. The interior columns will be on a 10-foot grid. Furthermore, it is understood that a mat foundation (which will serve as the first floor) is preferred for the building. It is also understood that the planned foundation includes isolated-column footings at various locations within the building for cross-bracing and for other structural requirements. The maximum foundation load from isolated columns could be as high as 250,000 pounds due to internal bracing; however, the actual framing system for cross-bracing is unknown and the resulting loads may vary significantly. The foundation and ground floor system for the building is planned to consist of a mat with a resulting pressure at the bottom of the mat of about 1,200 psf. It is assumed that the proposed building will not have a basement, but will have elevator pits. The elevator pits are expected to be cast-in-place concrete structure that will be a maximum of 4 to 5 feet deep.



The proposed improvements will include new parking areas along the west side of the building, and drive aisles surrounding the building. New pavement is expected to consist of asphalt-concrete, except in high-stress areas, where Portland cement concrete (PCC) pavement is expected.

Proposed first floor building elevation was not available at the time this report was prepared. It is recommended that the finished floor elevation be kept as high as possible due to shallow water (discussed later) and the adjacent lake level. Considering the existing grading at the site, it is herein assumed that the first floor be at El. 101.0. This report also assumes that surface grades of new pavement will be between El. 99.5 and El. 101.0 in the parking lot and surrounding the building, and generally matching the surface elevations in the access road. These elevations are referenced to Giles' adopted benchmark shown in Figure 1A. Based on the assumed elevations and existing topography, cuts and fills of less than 2 feet are expected to be necessary for site development.

5.0 GEOTECHNICAL SUBSURFACE EXPLORATION PROGRAM

To explore subsurface conditions, sixteen geotechnical test borings were performed in the proposed development area using a truck mounted mechanical drill-rig. Nine test borings (Test Borings 1 thru 9) were originally planned, but as a result of frequent interference with buried cobbles and/or boulders in the fill (described later) an additional seven test borings (Test Borings 1A, 2A, 3A, 3B, 3C, 3D, and 9A) were performed; generally offset in the vicinity of the originally planned boring locations.

Test Borings 1, 1A, 2, 2A, 3, 3A thru 3D, 4, 5, and 6 were in the planned building area. Test Boring 1 was terminated at 0.5 feet due to interference with the basalt bedrock. Test Boring 1A advanced via hollow stem auger to 1 foot where auger refusal was encountered at the basalt bedrock. Test Boring 1A was further explored by advancing by rock coring methods to a depth of ± 10 feet below-ground. Test Borings 2, 3, 3A, 3B, 3C, 9, and 9A were terminated within the fill (described later), terminating due to auger refusal, expected to be a result of interference with buried cobbles and/or boulders within the fill. Test Boring 3D also terminated within the fill at a depth of 4.2 feet, due to interference with buried cobbles and/or boulders. Test Boring 3D was further advanced by means of rock coring methods to get through the buried obstructions (described later), and by means of rock coring advanced to a termination depth of 14 feet, terminating within the intact bedrock. Test Boring 2A was drilled to depth of 31 feet, generally as planned. Test Borings 4, 5, and 6 were terminated at depths ranging between 13.1 and 24 feet, where auger and split spoon refusal were encountered; expected to be a result of interference with the underlying intact basalt bedrock. Test Borings 7 and 8 were drilled to respective depths of 10 and 9.7 feet, generally as planned. Approximate locations of the test borings are shown on the *Test Boring Location Plan*. Test boring locations were positioned on-site relative to site features.



Samples were collected from each test boring, at certain depths, using the Standard Penetration Test (SPT), conducted with the drill rig. A brief description of the SPT is given in Appendix B, along with descriptions of other field procedures. Immediately after sampling, select portions of SPT samples were placed in containers that were labeled at the site for identification. A Standard Penetration Resistance value (N-value) was determined from each SPT. N-values are reported on the *Test Boring Logs* (in Appendix A), and were used to estimate the relative density of granular site soil. Rock core samples were collected by means of NQ (wireline) rock coring, with individual rock core runs of 4 to 5 feet. The collected rock core samples were retained in cardboard boxes. The retained samples were transported to Giles' geotechnical laboratory for classification by a geotechnical engineer, and subsequent laboratory testing.

At the completion of each boring, the boreholes were backfilled with cuttings from the drilling operations; except that Test Boring 2A was filled with a grout slurry mixture. Backfill materials will likely settle and/or heave, possibly creating a hazard that can lead to a threat of injury to people and animals. Borehole areas should, therefore, be carefully and routinely monitored by the property owner; settlement and/or heave of backfill materials should be repaired immediately. Giles will not monitor or repair boreholes or borehole areas after leaving the site.

Ground elevations at the test boring locations were estimated using differential leveling methods relative to Giles' adopted benchmark shown on the *Test Boring Location Plan*. The test boring elevations are noted on the *Test Boring Logs* and are considered accurate within about 1 foot.

6.0 GEOTECHNICAL LABORATORY SERVICES

The retained samples were classified using the descriptive terms and particle-size criteria shown on the *General Notes* in Appendix D, and by using the Unified Soil Classification System (ASTM D 2488-75) as a general guide. Classifications are shown on the *Test Boring Logs*, along with horizontal lines that show estimated depths of material change. Field-related information pertaining to the test borings is also on the *Test Boring Logs*. For simplicity and abbreviation, terms and symbols are used on the *Test Boring Logs*; the terms and symbols are defined on the *General Notes*.

Moisture content tests were performed on select SPT samples to evaluate their general engineering properties. Atterberg limits tests were conducted on select samples retained from Test Boring Nos. 2A, 4, 6 and 9 for classification purposes, to evaluate plasticity characteristics. Sieve analyses were conducted on select samples retained from Test Boring Nos. 2A, 3C, 4, and 8 to evaluate grain size characteristics and aide in soil classification. Laboratory test results are summarized on the *Test Boring Logs*, and results of the sieve analyses are shown on Figure 2 in Appendix A. Because SPT samples were used, results of the penetrometer tests shown on the *Test Boring Logs* are considered to be approximate, and were used as supplemental information.

Recovery (REC) percentages were recorded for each of the cored samples (including within the boulder fill), which is presented as a percentage of the total length of recovered sample to the



total length of the cored run. Rock quality designation (RQD) values were determined for each of the core samples. The RQD values are reported on the notes of the *Test Boring Logs*. RQD is expressed as a percentage and is the summation of the length of intact core segments longer than 4 inches divided by the total length of the core run. RQD is a rough measure of the degree of jointing or fracture in a rock mass. Results of the RQD values are provided on the *Test Boring Logs*.

As part of the Geotechnical Laboratory Services, SPT samples retained from each test boring were screened, in the laboratory, with a Photoionization Detector (PID) to check for Volatile Organic Compound (VOC) vapors, such as vapors associated with gasoline. The results of PID screening of other samples were *below detection limits* (BDL), and are reported as such on the *Test Boring Logs*. The PID results on the *Test Boring Logs* are lab-screened values and might differ from values determined in the field immediately after sample collection.

7.0 MATERIAL CONDITIONS

The subsurface conditions at each test boring were estimated based on the retained samples. Estimated subsurface conditions (at the test borings) are briefly discussed in this section, and are described in more detail on the *Test Boring Logs*. This report is based on the estimated conditions.

7.1. Surface Materials

Topsoil was encountered at the surface at Test Boring 1. Asphalt pavement between 1 and 12 inches thick was at the surface at the remaining test borings. A crushed stone base course was observed below the asphalt pavement at Test Borings 2, 3, 3A, 3C, 3D, 4, 8, and 9, and ranged in thickness between 1 and 5 inches.

7.2. Fill / Possible Fill Materials

Soil classified as fill and/or possible fill was below the surface materials at each of the test boring locations, extending to depths ranging between 0.5 inches and 14 feet. The fill predominantly consisted of silty sand containing variable amounts of gravel, and also containing cobbles and/or boulders, but also contained silt and clay containing variable amounts of sand and gravel, some also containing cobbles and/or boulders. Organics and/or wood fragments were observed in several samples collected from within the fill and possible fill.

Based on the Atterberg limit test results from Test Boring No. 2A (LL=19, PI=1), No. 4 (LL=19, PI=3), No. 6 (LL=33, PI=7) and 9 (LL=44, PI=23) the tested samples exhibited slight to medium plasticity and low to medium expansive potential. The N-values were variable, but generally indicating a very loose to firm relative density, based on N-values from STSS. Many of the SPTs were elevated as a result of interference with cobbles and/or boulders, and as such are not indicative of relative density of the sampled fill materials.



7.3. Native Soil

Native soil was below the fill and/or possible fill at Test Borings 2A, 4, 5, 6, and 7 and extended to depths ranging between about 13.1 to 24 feet at Test Borings 4, 5, and 6 where auger and split spoon refusal were encountered. At Test Borings 2A and 7 the native soils extended to their respective termination depths. The native soil generally consisted of silty to clayey sand with variable amounts of gravel, and silt and lean clay containing variable amounts of sand, and occasionally containing gravel. The native non-cohesive soils generally exhibited firm to very dense relative densities based on N-values from SPTs.

7.4. Bedrock

Basalt bedrock was believed to have been encountered within Test Boring Nos. 1A and 3D (and expected to have been encountered at Test Boring 1), beginning at depths of about 1.0 and 9.8 feet respectively. The bedrock extended to the maximum test boring depths of about 10.0 and 14.0 feet respectively at these test borings. Bedrock core samples were recovered from Test Borings 1A and 3D with individual 4 to 5-foot runs, with a total of 9 feet and 4.2 feet respectively. The core samples of the bedrock were predominantly very slightly to moderately fractured, partially to moderately healed, hard to very hard based on visual observation and rock classification. The percent of recovery for each individual rock core run were between 70% and 100%; rock quality designation (RQD) values were measured to range between 60 and 88%.

8.0 GROUNDWATER CONDITIONS

Groundwater was encountered during drilling at depths ranging between 3 and 7.5 feet at Test Borings 2A, 3C4, 5, 7, 8, 9, and 9a, but not encountered at the remaining test borings when the test borings were conducted. Considering the lack of water encountered and the moisture contents of the sampled soils at some of the test borings it is expected that the groundwater was deeper than the maximum exploration depth when the Geotechnical Subsurface Exploration Program was conducted, but that the encountered water conditions are a result of perched water associated with the adjacent lake to the west and the granular soils predominant at the site. Encountered water was shallower along the west side of the site (nearest to the existing lake) and appears to be deeper (when encountered) with distance from the lake, consistent with water seepage from the adjacent lake.

The groundwater conditions discussed above are only an estimate based on the conditions at the test borings. The actual conditions could differ and the water table might be higher than estimated. If a precise determination of the groundwater conditions at the site is needed, more permanent groundwater observation wells could be installed and monitored. Giles can install and monitor observation wells, if it is decided that a precise determination of the groundwater conditions is needed.



9.0 CONCLUSIONS AND RECOMMENDATIONS

YELLOW – This project has been given a Yellow designation, from a geotechnical standpoint due to the existing fill and possible fill throughout the site containing cobbles and/or boulders, with some loose areas recommended to be improved by means of surface compaction that may also require some over-excavation. A mat/slab is then recommended in conjunction with spread footings structurally tied to the mat slab as the foundation system. An 8-inch-thick aggregate layer is recommended to be below the mat, serving as a cushion layer. The footings should be over-excavated to a depth of at least 1 foot and replaced with suitable engineered fill, also serving as a cushion layer.

9.1. Seismic Design Considerations

A soil Site Class C is recommended for seismic design. By definition, Site Class is based on the average properties of subsurface materials to 100 feet below-ground. Since 100-foot borings were not requested or authorized, it was necessary for Giles to estimate Site Class based on the borings, presumed geology, prior projects in the area, and International Building Code.

9.2. Existing Fill Considerations

Several of the samples from within the existing fill at the site contained an estimated trace amounts of organics and/or root matter. It is possible that unacceptable amounts of deleterious materials and/or construction debris may be buried within the fill at the site. **If existing fill materials containing excessive amounts of deleterious materials are encountered within the proposed building area, it is recommended that these materials be removed and replaced with suitable engineered fill in accordance with the recommendations contained in this report.** Further, the existing fill contains cobbles and/or boulders. It is recommended that the foundations be over-excavated a minimum of one foot and replaced with compacted engineered fill so to provide a cushion layer between the bottom of footings and the buried cobbles/ boulders, but also reducing the potential for settlement at the foundation subgrade.

Following demolition (discussed later) and prior to the placement of additional fill in the building pad, it is recommended that the existing fill within the building pad be surface compacted using a minimum 10 ton (operating weight) vibratory roller operating at approximately the soil's resonant frequency, compacted in place with a minimum of 5 passes in each parallel and perpendicular direction, and until a minimum density of 95 percent Standard Proctor maximum dry density ((per ASTM D-698) has been achieved.



9.3. **Building Foundation Recommendations**

Design Recommendations

The foundation analysis was conducted with the assumption that the floor of the proposed building will be at El. 101.0. Based on that floor elevation, and assuming that the site will be prepared as recommended in this report, the proposed building can be supported by a mat/slab foundation with spread footings that are structurally connected to the mat/slab, which is understood to be the desired foundation system, as described earlier. It is estimated that the mat foundation will be 10 inches thick and that the bottom of the mat will be at about El. 100.2, and that a minimum 8-inch thick aggregate base will be beneath the mat. A somewhat increased aggregate base is recommended to act as a cushion layer for the mat. The mat/slab foundations is recommended to be supported by suitable-bearing *improved* existing soil, where the in-place density of the exiting fill and possible fill has been improved by surface compaction with vibratory compaction equipment as recommended in the following *Foundation Support Soil Requirements* Section; and/or engineered fill used to raise site grades. Any unsuitable existing fill materials should be removed and replaced with suitable bearing engineered fill. The mat/slab foundation is recommended to be designed using a maximum 65 pounds per square inch (psi/in) Modulus of Subgrade Reaction (K_s). A somewhat low modulus has been recommended considering the existing fill at the site. Spread footings are recommended to be supported on suitable native soil, improved existing fill / possible fill, or structural fill.

However, considering the cobbles and boulders contained within the fill, it is recommended that the existing fill should be over-excavated to a depth of at least one foot below the planned bottom of footings and replaced with controlled, compacted fill. Similarly, considering the shallow intact basalt bedrock that appears to have been encountered at Test Boring 1A, it is recommended that encountered bedrock at the southern end of the building be removed to a depth of at least 1 foot and replaced with controlled, compacted fill, creating a similar cushion layer between the bottom of footings and the intact bedrock. If exposed rock or large boulders is not removed from the foundation bearing elevations it will create a potential for excessive differential settlement.

It is recommended that spread footings bearing on suitable native soil, improved existing fill, possible fill, and/or structural fill be designed using a 3,000 psf maximum, net, allowable soil bearing capacity. A somewhat conservative bearing capacity has been recommended to limit over-excavation of existing fill / possible fill in conjunction with surface compaction improvement. The recommended subgrade modulus and bearing capacity could be increased by one third for short-term, transient loads, such as wind.

For geotechnical considerations, strip footing pads are recommended to be at least 18 inches wide and isolated column pads are recommended to be at least 24 inches wide, regardless of the calculated foundation-bearing stress. Foundation walls (if any) could be built of cast-in-place concrete or concrete masonry units. Spread footings should be structurally connected to



the mat/slab to control differential settlement. It is recommended and assumed that a structural engineer will provide specific foundation details including foundation dimensions, reinforcing, and other details.

It is understood that a minimum 42-inch foundation depth is required by the local building code. Therefore, perimeter footings and the perimeter of the mat slab are recommended to extend to a depth of at least 42 inches below the finished exterior surface grade.

Based on the minimum foundation embedment depth, assumed floor elevation, and considering the site soils, suitable bearing soils were encountered are considered to be at the foundation bearing depth after surface compaction improvement and presuming that the mat and footings are over-excavated to at least 8 and 12 inches respectively and replaced with compacted fill, as recommended and described in further detail in the Foundation Support Soil Requirements section of this report.

A frictional coefficient of 0.45 is recommended to determine the ultimate lateral resistance of foundations. The recommended coefficient is only for concrete cast directly on the granular site soil or engineered fill used to raise site grades that has been tested and approved by a geotechnical engineer or representative of the retained Testing and Inspections Agency. Lateral resistance (due to friction) should be determined based on dead load only. Also, the ultimate lateral resistance determined from the frictional coefficient is recommended to be factored to determine an allowable value. A factor of safety of 3 is recommended to determine the allowable lateral resistance.

A minimum 8-inch- base course is recommended to below the mat/slab foundation to serve as a capillary break, and also serving as a cushion layer for the mat. It is recommended that the base course consist of free-draining aggregate. On-site soils may be suitable for use as the base course. It is recommended that the geotechnical engineer test and approve base-course aggregate before it is placed. Depending on aggregate gradation, a geotextile might need to be below the base course to protect the vapor barrier. Where the vapor retarder is placed below the base, additional cover is not considered required such as where the foundation concrete is in contact with the soil.

A minimum 15-mil vapor retarder is recommended for moisture control. The vapor retarder should completely underlie the floor area, and extend to all perimeter grade beams. Abutting vapor-retarder sheets are recommended to be overlapped and taped. Vapor retarders are recommended to be in accordance with ASTM E 1745, entitled: *Standard Specification for Plastic Water Vapor Retarders Used in Contact with Soil or Granular Fill under Concrete Slabs*, or other relevant documents.

It is critical that contractors protect foundation support soil and foundation construction materials (concrete and reinforcing). In addition, engineered fill is recommended to be placed and compacted in benched excavations along foundations, immediately after the foundations are



capable of supporting lateral pressures from backfill, compaction, and compaction equipment. Footings constructed in granular soils will likely need to be formed due to instability of the excavation sidewalls.

Foundation excavations are recommended to be excavated with a smooth-edge backhoe bucket to develop a relatively undisturbed bearing grade. A toothed bucket will likely disturb foundation-bearing soil more than a smooth-edge bucket, thereby making soil at the excavation base more susceptible to saturation and instability, especially during adverse weather, and may require recompaction of the bearing grade.

Foundation Support Soil Requirements

The mat/slab foundations are recommended to be directly and entirely supported by a minimum 12-inch cushion layer (and 8 inch-thick cushion layer below the mat, as recommended earlier) supported by bearing improved existing fill, native soil, engineered fill used to raise site grades. However, immediately before foundation construction, any disturbed or loose soil within foundation areas and footing trenches (at a depth of at least 12 inches below the foundation-bearing grades, when over-excavating to provide the 12-inch cushion layer (8 inches for the mat) and over-excavation as recommended previously) is recommended to be removed or compacted in-place using an appropriate compactor approved by the geotechnical engineer. Where compaction is performed, soil to at least 12 inches below the foundation bearing grades (and 8 inches below the floor surface) is recommended to be compacted to at least 95 percent of the soil's maximum dry density determined by Standard Proctor (ASTM D-698). It is further recommended for mat slab foundations and spread footing foundations that granular soil at the foundation bearing grades and within at least 2 feet of the foundation bearing grade have an average N-value greater than or equal to 10 for the recommended soil bearing capacity and subgrade modulus (given above), unless Giles approves a lower N-value.

After the building area has been cut to planned subgrade and prior to placing on structural fill low areas, **the building pad is recommended to be improved in place by means of surface compacted with a 10-ton (operating weight) single, smooth-drum vibratory roller.** The entire proposed building area is recommended to be surface compacted, except that compaction equipment should be kept an appropriate distance away from adjacent pavement and other nearby facilities. In general, the compactor should make at least 5 passes over the building pad in a north-south direction, and at least 5 passes over the site in an east-west direction, perpendicular to the initial passes. Parallel passes should overlap at least 18 inches. Where feasible, surface compaction should also extend at least 10 feet beyond the building pad. Soils are recommended to be surface compacted to at least 95 percent of the maximum dry density determined by the Standard Proctor compaction test (ASTM D-698) as described in the attached *Guide Specifications*. Care must be taken during surface compaction, since it could induce settlement at neighboring properties. The geotechnical engineer should, therefore, monitor compaction operations using vibration-monitoring equipment. The compaction equipment should be operated approximately at the soil's resonant frequency; however, the amplitude and frequency should be carefully monitored to ensure the soils are not being



adversely affected considering the shallow water table. If improvement problems develop, it may be necessary to fill low areas prior to proceeding.

Because of the existing fill and possible fill that is recommended to be improved through surface compaction, it is recommended that the geotechnical engineer evaluate (and approve) foundation support-soil immediately before foundation construction. The purpose of the recommended evaluation is to confirm that the foundations will be properly supported; to determine over-excavation depths (if any); and to confirm that the support soil is similar to the conditions described herein and shown on the *Test Boring Logs*. If another firm (other than Giles) performs the recommended evaluation, they should use appropriate means and methods and Giles should be notified if the composition or strength characteristics of foundation support-soil differ from those shown on the *Test Boring Logs*, allowing us to revise this report, if needed. Soil that yields excessively or ruts during surface compaction, or shows other signs of instability, is recommended to be scarified to a sufficient depth (at least 6 to 12 inches), moisture-conditioned (uniformly moistened or dried), and compacted to the required in-place density. As an option to in-place improvement, unsuitable soil could be replaced with engineered fill. It is recommended that the geotechnical engineer provide specific soil improvement recommendations based on the conditions during construction. The geotechnical engineer should also observe and approve the compaction procedures, and determine when the existing soils have been sufficiently improved and have relatively uniform support characteristics. All footings and the mat/slab must be directly supported by improved soils. Based on the recommended 3,000 psf maximum, net, allowable soil bearing capacity, a maximum subgrade modulus (K_s) of 65 pci, the in-situ unconfined compressive strength of cohesive soil within foundation influence zones is recommended to be at least 1.5 tons per square foot (tsf) and granular soil within foundation influence zones is recommended to have an N-value (determined from SPTs and correlated from other in-situ tests) of at least 10. It is further recommended that the strength characteristics of soil within all foundation influence zones (determined by the geotechnical engineer during construction) meet or exceed the recommended values, unless Giles approves other values.

The geotechnical engineer should provide recommendations pertaining to soil over-excavation and replacement at the time of construction. As an option to soil replacement, spread footing pads could be stepped or thickened to extend through unsuitable bearing materials and isolated column pads could be uniformly thickened. It is recommended that a structural engineer provide the specific details of stepped or thickened footings.

Estimated Foundation Settlement

The post-construction total and differential settlements of the recommended foundation systems designed and constructed based on the recommendations in this report are estimated to be less than 1.0 inch and 0.5 inch, respectively. The post-construction angular distortion of spread



footings is estimated to be less than about 0.002 inch per inch across a distance of 20 feet or more. The mat/slab foundation differential settlement is estimated to be less than 0.5 inch over 40 feet.

9.4. Elevator Pit Recommendations

The proposed three-story building is assumed to include an elevator pit or pits located along the west side of the building. Additional details regarding the elevator pit were not provided; however, it is assumed that the pit will be a reinforced cast-in-place concrete structure. It is also assumed that elevator-pit floor will be 4 to 5 feet below the first floor. Giles must be notified if elevator-pit floor will be deeper; this report might need to be revised. Geotechnical-related recommendations for an elevator pit that is 4 to 5 feet deep are provided below. The elevator pit must be designed based on the appropriate codes.

This report assumes that floor of the elevator pit will be at or above El. 97.0 to 96.0. Groundwater was encountered at approximate elevation El. 95.7 and El. 96.2 at Test Borings 2A and 3C respectively (expected to be associated with perched conditions associated with the close proximity to the lake to the west). Therefore, it is assumed that the floor of the elevator pit will be above the water table, but may encounter perched water conditions associated with the close proximity to the lake to the west. The perched water is expected to be close to, but and generally about at the perched water elevation and will be supported by suitable bearing existing soil. The elevator pit foundation is recommended to be designed based on the foundation recommendations provided in the Foundation Recommendations Section of this report. It is assumed that elevator-pit walls will be cast near existing soil, and that engineered fill between the walls and surrounding soil will consist of properly compacted, imported or on-site granular material approved by the geotechnical engineer. Based on that assumption, elevator-pit walls are recommended to be designed for an equivalent "at-rest" fluid pressure of 50 psf/foot. Horizontal pressures caused by surface and subsurface surcharge loads (such as first-floor loads) must be added to the "at-rest" fluid pressure. Giles could provide supplemental recommendations regarding surface and subsurface surcharge loads on a case-by-case basis, but would require specific structural information.

9.5. Pavement Recommendations

Giles was not given information regarding future pavement traffic. Therefore, to provide pavement recommendations, it was necessary for Giles to use an arbitrarily-selected traffic volume. The recommended pavement section is for traffic consisting of only automobiles in parking stalls a maximum daily traffic volume of 1,000 and automobiles and five 18,000-pound equivalent single axle loads (ESALs) from trucks in the drive lanes.

It is recommended that the client, civil engineer and other design professionals involved with the project confirm that the arbitrarily-selected traffic volumes are appropriate. If requested, Giles can provide supplemental pavement recommendations based upon other traffic conditions if the



arbitrarily-selected traffic volumes are not appropriate. If the pavement section is subject to traffic greater than assumed, increased maintenance and premature failure could occur.

Hot-Mix Asphalt Pavement

A California Bearing Ratio (CBR) test is commonly used to determine the soil support parameters for pavement design. Since a CBR test was not authorized for this project, it was necessary for Giles to assume the CBR design value used to give pavement recommendations. The following pavement sections are based on a clayey sand fill subgrade and an assumed CBR design value of 6. Engineered fill that is placed in proposed pavement areas is recommended to be placed and compacted per the recommendations of this report.

The following table shows the recommended thicknesses for pavement sections consisting of hot-mix-asphalt (HMA) over an aggregate base course, along with the appropriate state highway specifications so that the proper materials are specified and used. The pavement sections are based upon the traffic volumes presented in this report. If the pavement is subject to greater amounts of traffic, increased pavement maintenance and premature pavement failure should be expected. Local codes may require specific testing to determine soil support characteristics and/or minimum pavement section thicknesses.

Recommended Pavement Sections			
Materials	Section Thickness (Inches)		New Jersey DOT Standard Specifications
	Parking Stalls	Drive Lanes	
Hot Mix Asphalt Surface Course	1½	1½	(a) Section 404
Hot Mix Asphalt Binder Course	2	2½	(a) Section 404
Dense Graded Aggregate Base Course	6	6	(b) Section 301 Graded Aggregate Base
(a) Compaction to within 92% and 97% Blow Marshal Density			
(b) Compaction to at least 100% of the standard Proctor (ASTM D-698) Compaction test Maximum Density.			

A minimum 6-inch-thick Portland cement concrete (PCC) pavement with a minimum 4-inch-thick compacted aggregate base course is recommended for high-stress areas such as at the entrance/exit aprons, at the dumpster pad enclosure, and in areas where trucks will turn or will be parked. Concrete should have a minimum 28-day compressive strength of 4,000 psi with 4 to 7 percent air entrainment. Control joint spacing should be determined in accordance with the current ACI code. Expansion joints should be provided where pavement abuts fixed objects, such as the buildings and light poles. Materials and construction procedures for concrete pavement are recommended to be per New Jersey DOT Standard Specifications Section 405 for concrete and Section 301 for base course.

The pavement recommendations assume that pavement subgrade will be prepared per this report, the base course will be properly drained, and a qualified geotechnical engineer will



observe and test pavement construction. The pavement was designed based on AASHTO and/or PCA design parameters for a twenty-year design period. Pavement maintenance along with a major rehabilitation after about 8 to 10 years should be expected. Local codes may require specific testing to determine soil support characteristics and/or minimum pavement section thickness might be required.

9.6. Generalized Site Preparation Recommendations

This section deals with site preparation, including preparation of floor slab (surface compaction of the building area is discussed in Section 9.3, *Building Foundation Recommendations*), pavement, and engineered fill areas. The means and methods of site preparation will greatly depend on the weather conditions before and during construction, the subsurface conditions that are exposed during earthwork operations, and the finalized details of the proposed development. Therefore, only generalized site preparation recommendations are given.

In addition to being generalized, the following site preparation recommendations are abbreviated; the *Guide Specifications* in Appendix D give further recommendations. The *Guide Specifications* should be read along with this section. Also, the *Guide Specifications* are recommended to be used as an aid to develop the project specifications.

Demolition and Site Clearing

It is understood that the existing building is planned to be razed. Site preparation will require complete removal and proper disposal of the existing construction and any remnants of previous construction, including all foundations, floor slabs, and below-grade walls within the proposed building footprint and planned new pavement. Underground utilities that are within the proposed building and are not reused should also be removed or properly abandoned in-place per local requirements. All soils disturbed by the demolition operations should be removed to expose a competent subgrade. Material used to backfill any excavation is recommended to consist of material in accordance with that recommended in the Proofrolling and Fill Placement subsection below.

Disposal of debris should be in accordance with local, state and federal regulations for the material type. Considering the overlap of the existing building and the proposed building (as shown in Figure 1 located in Appendix A), the existing building is within the vicinity of the proposed building footprint. It is recommended that the foundations and floor slab should be removed in their entirety. All materials resulting from the demolition of the existing structure and infrastructure should be legally disposed off-site. It should be noted that any construction remnants left in-place may cause excavation difficulties for new utilities, landscape plantings and future construction. All excavations should be backfilled with engineered fill. It is also expected to be necessary to bench into the surrounding soils, as noted in the *Guide Specifications* enclosed in Appendix D.



The existing pavement should be removed and properly disposed off-site, or it could be pulverized into a well-graded material (maximum 3-inch size particles) and used on-site as fill, subbase course material, or subgrade stabilization material. Similarly, the concrete and/or brick from the demolition of the existing structure could also be pulverized into a similarly well-graded material (maximum 3-inch size particles) and used on site as fill within the building or pavement areas. Pulverized asphalt material should not, however, be used as fill beneath the planned building. The existing pavement is recommended to be left in-place as long as possible to help protect the subgrade from weather and construction traffic disturbance.

Where present, surface vegetation, trees and bushes (including root-balls), topsoil with adverse organic content, and otherwise unsuitable bearing materials are recommended to be removed from the proposed building footprint, pavement area, and other structural areas. Clearing, grubbing and stripping should extend at least several feet beyond proposed development areas, where feasible. Topsoil and soils containing root matter and organics may be deeper or shallower in the wooded areas than encountered at the test borings.

Proof-Rolling and Fill Placement

After the recommended demolition, clearing, grubbing, and stripping, and lowering of site grades, where necessary, and after the building area has been cut to planned subgrade and prior to placing on structural fill low areas, the building pad area is recommended to be surface compacted with a 10-ton (operating weight) single, smooth-drum vibratory roller. The entire building pad is recommended to be surface compacted, except that compaction equipment should be kept an appropriate distance away from adjacent pavement and other nearby facilities. In general, the compactor should make at least 5 passes over the site in a north-south direction, and at least 5 passes over the site in an east-west direction, perpendicular to the initial passes. Parallel passes should overlap at least 18 inches. Where feasible, surface compaction should also extend at least 5 feet beyond the development area of the site. Soils are recommended to be surface compacted to at least 95 percent of the maximum dry density determined by the Standard Proctor compaction test (ASTM D-698) as described in the attached *Guide Specifications*. Care must be taken during surface compaction, since it could induce settlement at neighboring properties. The geotechnical engineer should, therefore, monitor compaction operations using vibration-monitoring equipment. The amplitude and frequency should be carefully monitored to ensure the soils are not being adversely affected considering the shallow water table. If improvement problems develop, it may be necessary to fill low areas prior to proceeding.

Soil that yields excessively or ruts during surface compaction, or shows other signs of instability, is recommended to be scarified to a sufficient depth (at least 6 to 12 inches), moisture-conditioned (uniformly moistened or dried), and compacted to the required in-place density. As an option to in-place improvement, unsuitable soil could be replaced with engineered fill. It is recommended that the geotechnical engineer provide specific soil improvement recommendations based on the conditions during construction.



The site is recommended to be raised, where necessary, to the planned finished grade with engineered fill immediately after a geotechnical engineer confirms that the subgrade has been sufficiently improved by surface compaction and is suitable to support the proposed improvements. Engineered fill is recommended to be placed in uniform, relatively thin layers. Each layer is recommended to be compacted to at least 95 percent of the fill material's maximum dry density determined by the Standard Proctor compaction test (ASTM D-698). As an exception, the in-place dry density of engineered fill within one foot of the pavement subgrade is recommended to be compacted to at least 98 percent of the fill's maximum dry density. Item Nos. 4 and 5 of the *Guide Specifications* give more specific information pertaining to selection and compaction of engineered fill.

The water content of fill material is recommended to be uniform and within a narrow range of the optimum moisture content, as described in Item No. 5 of the *Guide Specifications*. The optimum moisture content is to be determined by the Standard Proctor Compaction test.

Engineered fill that does not meet the density and water content requirements is recommended to be replaced or scarified to a sufficient depth (likely 6 to 12 inches, or more), moisture-conditioned, and compacted to the required density. A subsequent lift of fill should only be placed after a qualified geotechnical engineer (and/or their representative) or qualified representative of the retained Testing and Inspections Agency confirm that the previous lift was properly placed and compacted. Subgrade soil may need to be recompacted immediately before construction since equipment traffic and adverse weather may reduce soil stability.

Use of Site Soil as Engineered Fill

Site soil that does not contain adverse organic content or other deleterious materials, as noted in the *Guide Specifications*, could be used as engineered fill within the building pad and pavement areas. However, sorting through and removing oversized materials (larger than 3 inches in nominal size) will be required. The majority of the on-site soils generally considered suitable for use as engineered fill; however, it is possible that pockets of the existing fill may contain excessive amounts of deleterious materials. However, some of the existing site soils may still need to be moisture conditioned (dried) considering the elevated water contents of some of the existing fill soils, and if construction is during fall or winter months. Drying will likely also be needed if construction is during or after wet weather. Drying may not be feasible, depending on the weather conditions and construction schedule, and aggregate fill (or other fill material with a low water-sensitivity) may need to be imported to the site. Additional recommendations regarding fill selection, placement and compaction are given in the *Guide Specifications*.



9.7. Generalized Construction Considerations

Import Soil Considerations

Material proposed for use as engineered fill should be classified as CL or better (based on the USCS soil classification system), and have a Liquid Limit and Plasticity Index less than 40 and 15, respectively. Low expansive material may be compacted at a moisture content that is within ± 2 percent of the optimum moisture content determined by Standard Proctor Compaction test as noted in Item No. 5 of the enclosed *Guide Specifications*. The suitability of import engineered fill materials is recommended to be evaluated by Giles prior to placement.

Adverse Weather

Site soil is extremely sensitive to moisture, and will become unstable when exposed to adverse weather such as rain, snow, and freezing temperatures. Therefore, it might be necessary to remove or stabilize the upper 6 to 12 inches (or more) of soil due to adverse weather, which commonly occurs during late fall, winter, and early spring. At least some over-excavation and/or stabilization of unstable soil should be expected if construction is during or after adverse weather. Because site preparation depends on weather, bids for site preparation, and other earthwork activities, should consider the time of year that construction will be conducted.

In an effort to protect soil from adverse weather, the site surface is recommended to be smoothly graded and contoured during construction to divert surface water away from construction areas. Also, contoured subgrades are recommended to be rolled with a smooth-drum compactor, before precipitation, to "seal" the surface. Furthermore, construction traffic should be restricted to certain aggregate-covered areas in an effort to reduce traffic-related soil disturbance. Foundation, floor slab, and pavement construction should begin immediately after suitable support is confirmed.

Dewatering

The majority of excavations for the proposed project are expected to be above the groundwater table; however, dewatering might be necessary due to precipitation or if perched groundwater is encountered. As indicated previously, perched water was at about El. 96 (referenced to Giles' temporary benchmark shown in Figure 1A) along the west side of the site (nearest to the existing lake) and appears to be lower with distance from the lake. Water that accumulates in construction areas is recommended to be removed along with unstable soil as soon as possible. Filtered sump pumps, drawing water from sump pits excavated in the bottom of construction trenches, will likely be adequate to remove water that collects in shallow excavations. Excavated sump pits should be fully-lined with a geotextile and filled with open-graded, free-draining aggregate.



Excavation Stability

Excavations are recommended to be made in accordance with current OSHA excavation and trench safety standards, and other applicable requirements. Sides of excavations will likely need to be sloped, benched, and/or braced to maintain or develop a safe work environment, especially considering the granular nature of the site soils. Temporary shoring must be designed according to applicable regulatory requirements. Contractors are responsible for excavation safety.

Existing Utilities

All existing utilities should be identified and located, and any planned to be maintained should be relocated outside the proposed building area. Utilities that are not reused should be capped and removed or properly abandoned in-place in accordance with local codes and ordinances. Excavations for removal of utilities are recommended to be backfilled with engineered fill. Underground utilities that are to be reused or abandoned in-place should be evaluated by a structural engineer and/or plumbing contractor, and utility backfill should be evaluated by a geotechnical engineer. Grading operations must be done carefully so that existing utilities are not damaged or disturbed. Utility invert elevations, depths and sizes should be checked relative to the planned foundation elevations.

9.8. Recommended Construction Materials Testing

This report was prepared assuming that the geotechnical engineer (and/or their representative) will perform Construction Materials Testing ("CMT") services during construction of the proposed development. In general, CMT services are recommended (and expected) to at least include observation and testing of: surface compaction, foundation, mat/slab and pavement support soil; soil, aggregate, concrete, asphalt, and other construction materials. Further, during the construction phase, and following completion of the recommended surface compaction, it is recommended that Giles be retained to perform additional test borings (as recommended in this report) to confirm that sufficient ground improvement has been achieved. It might be necessary for the geotechnical engineer to provide supplemental geotechnical engineering recommendations based on the results of CMT services and provided specific details of the project not known at this time.

10.0 BASIS OF REPORT

This report is strictly based on the project description given earlier in this report. Giles should be notified if any parts of the project description or our assumptions are not accurate so that this report can be amended, if needed. This report is based on the assumption that the facility will be designed and constructed according to the codes that govern construction at the site.



The conclusions and recommendations in this report are based on estimated subsurface conditions as shown on the *Test Boring Logs*. Giles should be notified if the subsurface conditions that are encountered during construction of the proposed development differ from those shown on the *Test Boring Logs* because this report will likely need to be revised. General comments and limitations of this report are given in the appendix.

The conclusions and recommendations presented in this report have been promulgated in accordance with generally accepted professional engineering practices in the field of geotechnical engineering. No other warranty is either expressed or implied.

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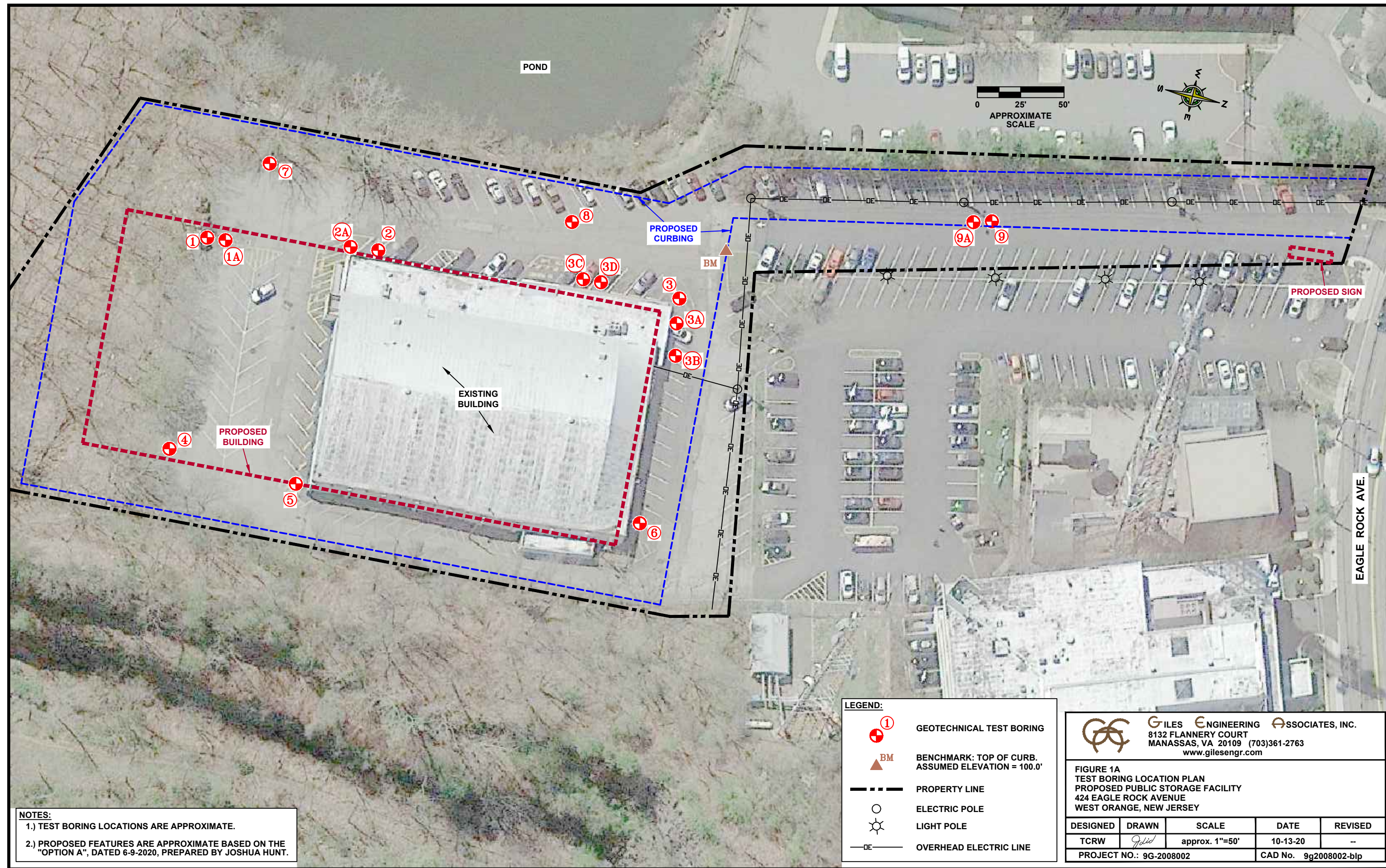


APPENDIX A

FIGURES AND TEST BORING LOGS

The Test Boring Location Plan contained herein was prepared based upon information supplied by *Giles'* client, or others, along with *Giles'* field measurements and observations. The diagram is presented for conceptual purposes only and is intended to assist the reader in report interpretation.

The Test Boring Logs and related information enclosed herein depict the subsurface (soil and water) conditions encountered at the specific boring locations on the date that the exploration was performed. Subsurface conditions may differ between boring locations and within areas of the site that were not explored with test borings. The subsurface conditions may also change at the boring locations over the passage of time.



NOTES:
1.) TEST BORING LOCATIONS ARE APPROXIMATE.
2.) PROPOSED FEATURES ARE APPROXIMATE BASED ON THE "OPTION A", DATED 6-9-2020, PREPARED BY JOSHUA HUNT.

LEGEND:

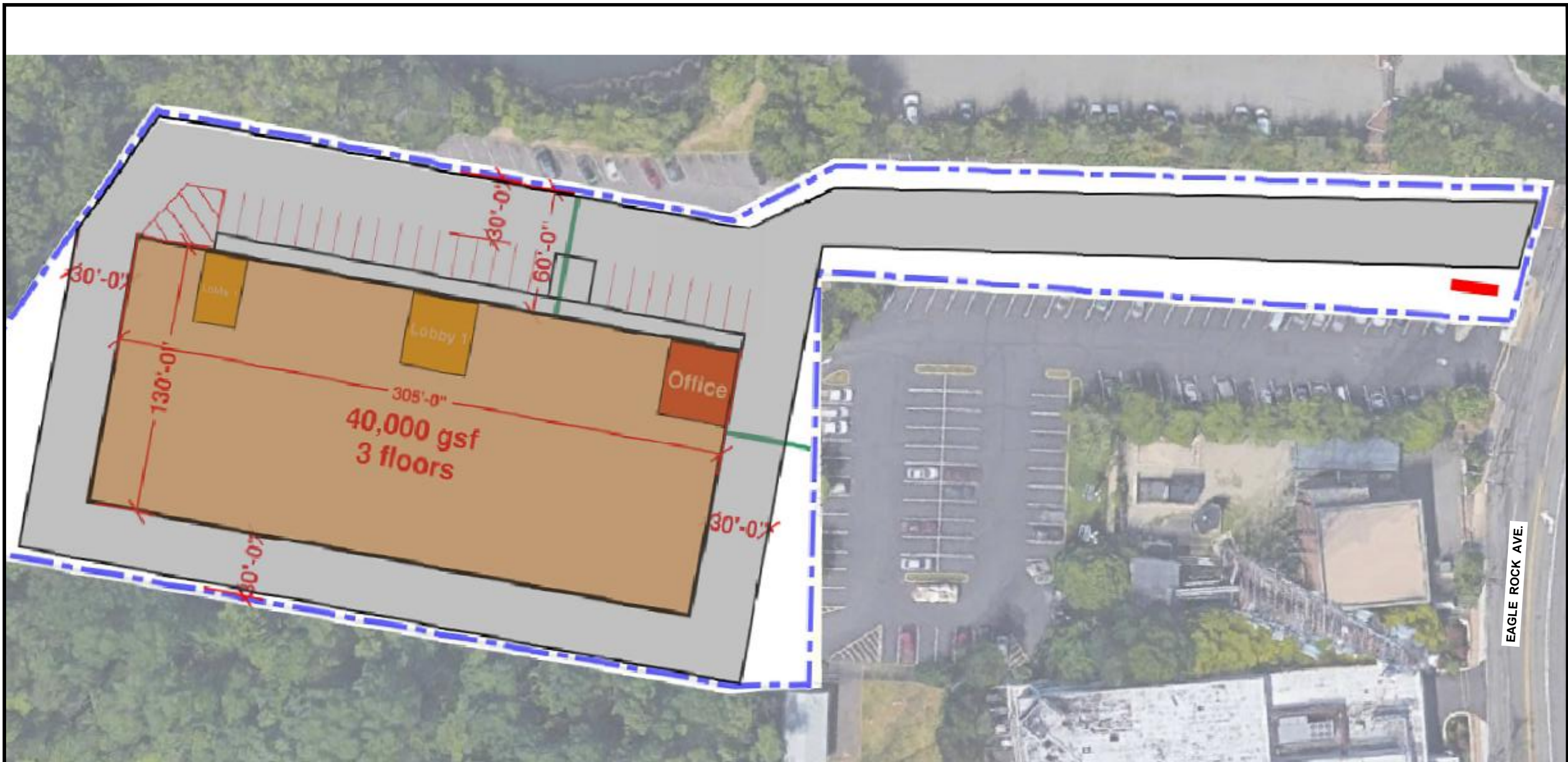
- GEOTECHNICAL TEST BORING
- BENCHMARK: TOP OF CURB. ASSUMED ELEVATION = 100.0'
- PROPERTY LINE
- ELECTRIC POLE
- LIGHT POLE
- OVERHEAD ELECTRIC LINE



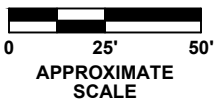
GILES ENGINEERING ASSOCIATES, INC.
8132 FLANNERY COURT
MANASSAS, VA 20109 (703)361-2763
www.gilesengr.com

FIGURE 1A
TEST BORING LOCATION PLAN
PROPOSED PUBLIC STORAGE FACILITY
424 EAGLE ROCK AVENUE
WEST ORANGE, NEW JERSEY

DESIGNED	DRAWN	SCALE	DATE	REVISED
TCRW	<i>Jed</i>	approx. 1"=50'	10-13-20	--
PROJECT NO.: 9G-2008002			CAD No. 9g2008002-blp	



NOTES:
1.) BASE MAP DEVELOPED FROM THE "OPTION A",
DATED 6-9-2020, PREPARED BY JOSHUA HUNT.



 GILES ENGINEERING ASSOCIATES, INC. 8132 FLANNERY COURT MANASSAS, VA 20109 (703)361-2763 www.gilesengr.com				
FIGURE 1B SITE PLAN PROPOSED PUBLIC STORAGE FACILITY 424 EAGLE ROCK AVENUE WEST ORANGE, NEW JERSEY				
DESIGNED	DRAWN	SCALE	DATE	REVISED
TCRW	<i>Jed</i>	approx. 1"=50'	10-13-20	--
PROJECT NO.: 9G-2008002			CAD No. 9g2008002-blp	

Giles Engineering Associates, Inc.
Sieve Analysis

MADE FOR:	Public Storage	DATE:	24-Sep-20
SOURCE:	See Legend	DATE RECEIVED:	
PROJECT:	Proposed Public Storage Facility, West Orange, NJ	PROJECT NO.:	9G-2008002

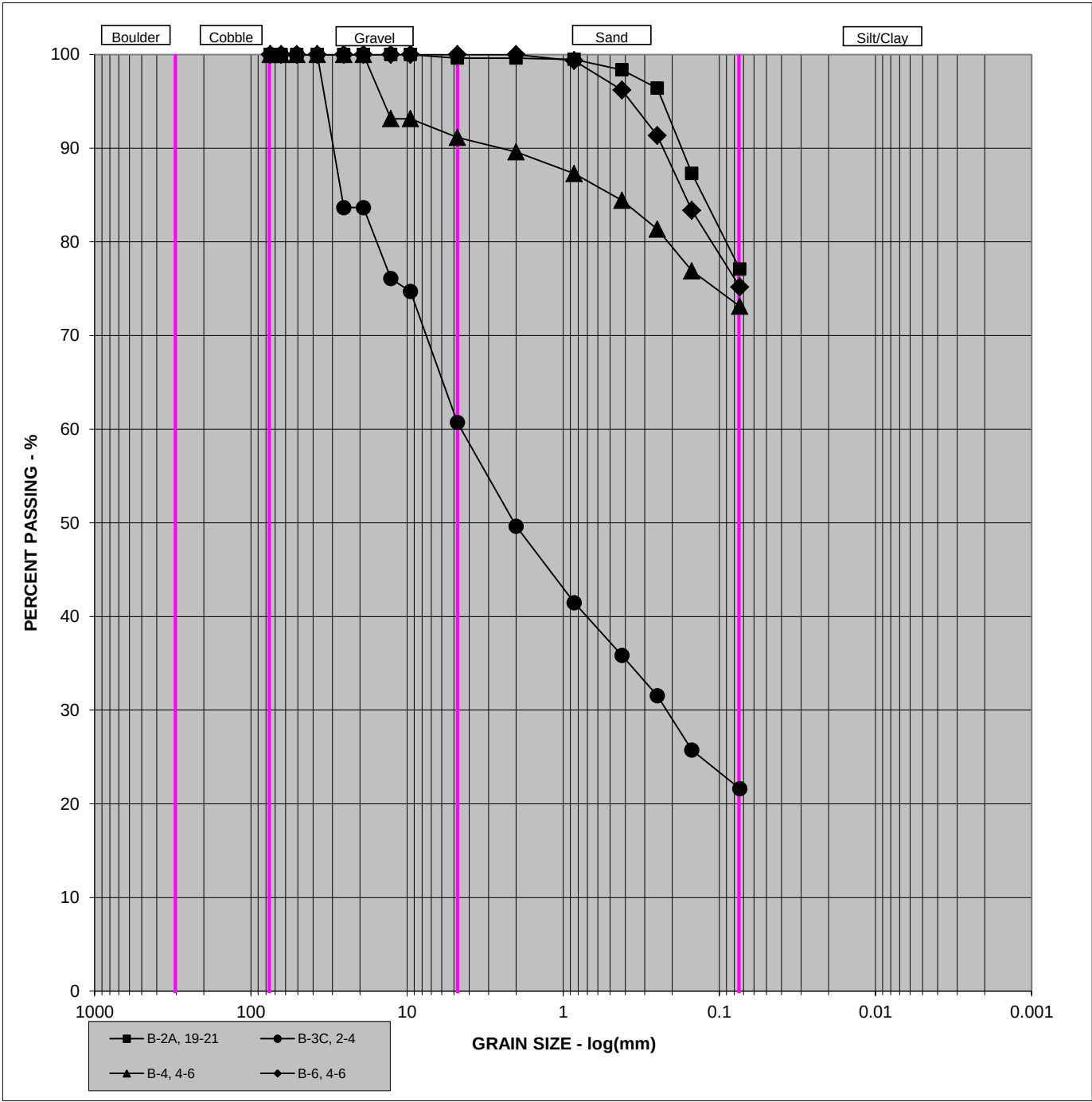


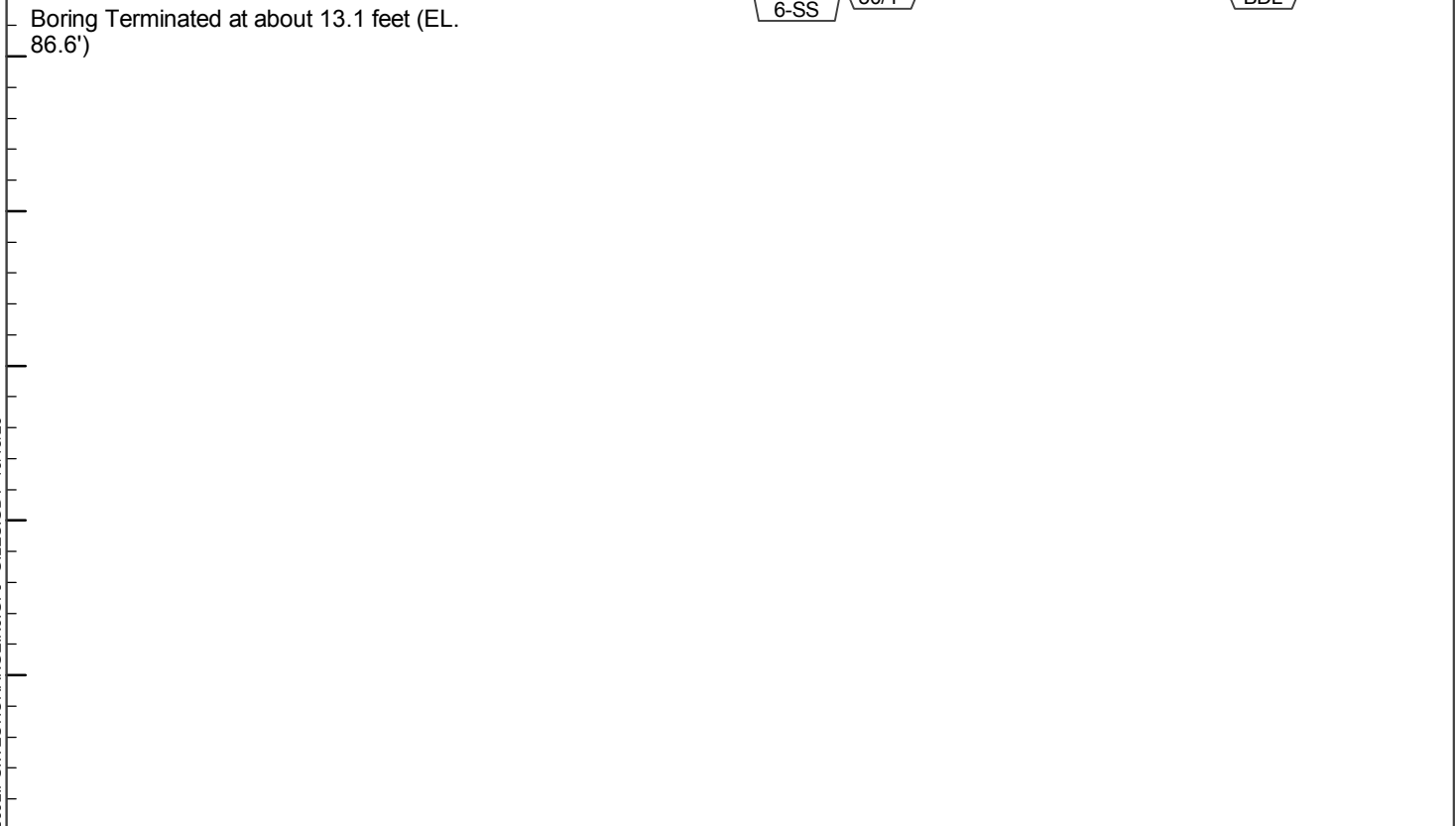
Figure 2

BORING NO. & LOCATION: 3	TEST BORING LOG	 GILES ENGINEERING ASSOCIATES, INC.	
SURFACE ELEVATION: 100.5 feet			PUBLIC STORAGE 424 EAGLE ROCK AVENUE WEST ORANGE, NEW JERSEY PROJECT NO: 9G-2008002
COMPLETION DATE: 09/01/20			
FIELD REP: TYLER WELCH			

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q _u (tsf)	Q _p (tsf)	Q _s (tsf)	W (%)	PID	NOTES
4" Asphalt, 5" Crushed Stone Base		100								
FILL: Black and Brown Silty fine to coarse Sand, some Clay, little Gravel, possible Cobbles, trace Wood Fragments - Very Moist			1-SS	22				14	BDL	
			2-SS	50/1*					BDL	
Boring Terminated at about 2.1 feet (EL. 98.4')										

BORING NO. & LOCATION: 4	TEST BORING LOG PUBLIC STORAGE 424 EAGLE ROCK AVENUE WEST ORANGE, NEW JERSEY PROJECT NO: 9G-2008002	 GILES ENGINEERING ASSOCIATES, INC.
SURFACE ELEVATION: 99.7 feet		
COMPLETION DATE: 09/01/20		
FIELD REP: TYLER WELCH		

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q _u (tsf)	Q _p (tsf)	Q _s (tsf)	W (%)	PID	NOTES
6" Asphalt, 1" Crushed Stone Base										
FILL: Brown Silty fine to coarse Sand, some Clay and Gravel, possible Cobbles and Boulders - Moist			1-SS	17				9	BDL	P200=73.1% (See Figure 2) LL=19, PI=3
			2-SS	10					BDL	
FILL: Greyish Brown Lean Clay, some fine to coarse Sand, little Gravel, trace Organics, possible Cobbles and Boulders - Moist	5	95	3-SS	22**				38	BDL	
			4-SS	29				14	BDL	
POSSIBLE FILL: Brown Silt, some Clay and fine to coarse Sand - Very Moist			5-SS	59				39	BDL	
Grey and Red Brown Silty fine to coarse Sand, little Clay and Gravel - Very Moist	10	90								
Brownish Red Clayey fine to coarse Sand, some Silt - Wet										
Boring Terminated at about 13.1 feet (EL. 86.6')			6-SS	50/1*					BDL	



Water Observation Data		Remarks:
	Water Encountered During Drilling: 7.5 ft.	Auger and split spoon refusal at 13.1 feet * N-value elevated due to interference with cobbles and/or boulders ** N-value likely elevated due to interference with cobbles and/or boulders
	Water Level At End of Drilling: NONE	
	Cave Depth At End of Drilling: 5 ft.	
	Water Level After Drilling:	
	Cave Depth After Drilling:	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION: 5	TEST BORING LOG	 GILES ENGINEERING ASSOCIATES, INC.	
SURFACE ELEVATION: 101.8 feet			PUBLIC STORAGE 424 EAGLE ROCK AVENUE WEST ORANGE, NEW JERSEY PROJECT NO: 9G-2008002
COMPLETION DATE: 09/02/20			
FIELD REP: TYLER WELCH			

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q _u (tsf)	Q _p (tsf)	Q _s (tsf)	W (%)	PID	NOTES
4" Asphalt			1-SS	50/4*				10	BDL	
FILL: Brown and Red Brown Clayey fine to coarse Sand and Gravel, contains Cobbles and Boulders - Moist		100	2-SS	50/0*				10	BDL	
FILL: Grey and Brown Silty fine to coarse Sand and Gravel, contains Cobbles and Boulders - Moist	5		3-SS	28**					BDL	
Greyish Brown Silt, some Clay, little fine to coarse Sand - Wet		95	4-SS	11					BDL	
Red Brown Silt and fine Sand, little Clay - Wet	10		5-SS	13				19	BDL	
	15		6-SS	28				21	BDL	
Red Brown Silty fine to medium Sand, trace Gravel - Wet	20		7-SS	17				10	BDL	
	80		8-SS	50/0*					BDL	

Boring Terminated at about 24 feet (EL. 77.8')

Water Observation Data		Remarks:
	Water Encountered During Drilling: 7 ft.	Auger and split spoon refusal at 24 feet * N-value elevated due to interfere with cobbles and/or boulders ** N-value likely elevated due to interfere with cobbles and/or boulders
	Water Level At End of Drilling: NONE	
	Cave Depth At End of Drilling: 4 ft.	
	Water Level After Drilling:	
	Cave Depth After Drilling:	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

GILES LOG REPORT 9G-2008002.PS.WEST ORANGE,NJ.GPJ GILES GDT 10/13/20

BORING NO. & LOCATION: 7	TEST BORING LOG	 GILES ENGINEERING ASSOCIATES, INC.	
SURFACE ELEVATION: 98.9 feet			PUBLIC STORAGE 424 EAGLE ROCK AVENUE WEST ORANGE, NEW JERSEY PROJECT NO: 9G-2008002
COMPLETION DATE: 09/03/20			
FIELD REP: TYLER WELCH			

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q _u (tsf)	Q _p (tsf)	Q _s (tsf)	W (%)	PID	NOTES
3" Asphalt										
FILL: Greyish Brown Clayey fine to coarse Sand, some Silt, little Gravel - Moist			1-SS	18				11	BDL	
FILL: Brown and Black Silty fine to coarse Sand, little Clay and Gravel, possible Cobbles - Very Moist		95	2-SS	22**				14	BDL	
	5		3-SS	23				20	BDL	
FILL: Brown and Black Silty fine to coarse Sand, some Gravel, little Clay, trace Organics, contains Cobbles and Boulders - Wet			4-SS	13				38	BDL	
Greyish Brown Lean Clay, some Silt, little fine to coarse Sand - Very Moist		90	5-SS	21				29	BDL	
	10									

Boring Terminated at about 10 feet (EL. 88.9')

Water Observation Data		Remarks:
	Water Encountered During Drilling: 4 ft.	** N-value likely elevated due to interferene with cobbles and/or boulders
	Water Level At End of Drilling: 4 ft.	
	Cave Depth At End of Drilling: 6 ft.	
	Water Level After Drilling:	
	Cave Depth After Drilling:	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION: 8	TEST BORING LOG	 GILES ENGINEERING ASSOCIATES, INC.	
SURFACE ELEVATION: 100.2 feet			PUBLIC STORAGE 424 EAGLE ROCK AVENUE WEST ORANGE, NEW JERSEY PROJECT NO: 9G-2008002
COMPLETION DATE: 09/02/20			
FIELD REP: TYLER WELCH			

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q _u (tsf)	Q _p (tsf)	Q _s (tsf)	W (%)	PID	NOTES
3" Asphalt, 2" Crushed Stone Base										
FILL: Brown and Greyish Brown Silty fine to coase Sand and Gravel, contains Cobbles and Boulders - Moist POSSIBLE FILL: Brown Silt, some Clay and fine to coarse Sand, trace Gravel and Organics - Very Moist			1-SS	21-50/0*					BDL	
			2-SS	50/1*						
			3-SS	7-50/5*						
			4-SS	21				3		
			5-SS	6						
			6-SS	50/2*						
Boring Terminated at about 9.7 feet (EL. 90.5')										

Water Observation Data	Remarks:
 Water Encountered During Drilling: 5.5 ft.  Water Level At End of Drilling: 4.5 ft.  Cave Depth At End of Drilling: 5 ft.  Water Level After Drilling:  Cave Depth After Drilling:	* N-value elevated due to interferene with cobbles and/or boulders

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

GILES LOG REPORT 9G-2008002.PS.WEST ORANGE,NJ.GPJ GILES GDT 10/13/20

BORING NO. & LOCATION: 1A	TEST BORING LOG	 GILES ENGINEERING ASSOCIATES, INC.	
SURFACE ELEVATION: 99.7 feet			PUBLIC STORAGE 424 EAGLE ROCK AVENUE WEST ORANGE, NEW JERSEY
COMPLETION DATE: 09/03/20			
FIELD REP: TYLER WELCH			

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q _u (tsf)	Q _p (tsf)	Q _s (tsf)	W (%)	PID	NOTES
3" Asphalt										
FILL: Brown Silty fine to coarse Sand, some Gravel, possible Cobbles - Moist			1-RC							REC=70% RQD=60%
BASALT BEDROCK, Dark Grey, very slightly to slightly fractured, Partially Healed, hard to very hard Rock	5	95								
BASALT BEDROCK, Dark Grey very slightly to slightly fractured, moderately healed, hard to very hard Rock			2-RC							REC=88% RQD=88%
	10	90								

Boring Terminated at about 10 feet (EL. 89.7')

Water Observation Data		Remarks:
	Water Encountered During Drilling:	
	Water Level At End of Drilling:	
	Cave Depth At End of Drilling:	
	Water Level After Drilling:	
	Cave Depth After Drilling:	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION: 2A	TEST BORING LOG	 GILES ENGINEERING ASSOCIATES, INC.	
SURFACE ELEVATION: 100.7 feet			PUBLIC STORAGE 424 EAGLE ROCK AVENUE WEST ORANGE, NEW JERSEY
COMPLETION DATE: 09/01/20			
FIELD REP: TYLER WELCH			

MATERIAL DESCRIPTION		Depth (ft)	Elevation	Sample No. & Type	N	Q _u (tsf)	Q _p (tsf)	Q _s (tsf)	W (%)	PID	NOTES
Drilled without sampling to 4.0 feet			100								
FILL: Dark Greyish Brown Silty fine to coarse Sand, some Gravel, contains Cobbles and Boulders - Moist		5	95	3-SS	14				7	BDL	LL=19, PI=1
				4-SS	WH					BDL	
POSSIBLE FILL: Greyish Brown Silty fine to coarse Sand, little Clay and Organics - Very Moist		10	90	5-SS	10				18	BDL	
Brown Silty fine to medium Sand - Wet		15	85	6-SS	21				20	BDL	P200=77.1% (See Figure 2)
Brownish Red Silt, some fine to medium Sand - Wet		20	80	7-SS	19				17	BDL	
		25	75	8-SS	11				22	BDL	
Brownish Red Silty fine to coarse Sand, trace Gravel - Wet		30	70	9-SS	28				8	BDL	

Boring Terminated at about 31 feet (EL. 69.7')

	Water Observation Data	Remarks:
	Water Encountered During Drilling: 6 ft.	
	Water Level At End of Drilling: 5 ft.	
	Cave Depth At End of Drilling: 10.3 ft.	
	Water Level After Drilling:	
	Cave Depth After Drilling:	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

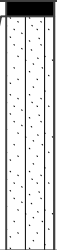
GILES LOG REPORT 9G-2008002.PS.WEST ORANGE,NJ.GPJ GILES GDT 10/13/20

BORING NO. & LOCATION: 3B	TEST BORING LOG PUBLIC STORAGE 424 EAGLE ROCK AVENUE WEST ORANGE, NEW JERSEY PROJECT NO: 9G-2008002	 GILES ENGINEERING ASSOCIATES, INC.
SURFACE ELEVATION: 100.8 feet		
COMPLETION DATE: 09/02/20		
FIELD REP: TYLER WELCH		

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q _u (tsf)	Q _p (tsf)	Q _s (tsf)	W (%)	PID	NOTES
1" Asphalt		100	1-SS	23				11	BDL	
FILL: Brown Silty Gravel and fine to coarse Sand, contains Cobbles and Boulders - Moist			2-SS	50/1*					BDL	
			2-SS	50/0*					BDL	

	Water Observation Data	Remarks:
	Water Encountered During Drilling: NONE	Auger and split spoon refusal at 3.0 feet * N-value elevated due to interferene with cobbles and/or boulders
	Water Level At End of Drilling: NONE	
	Cave Depth At End of Drilling: 2 ft.	
	Water Level After Drilling:	
	Cave Depth After Drilling:	

BORING NO. & LOCATION: 3C	TEST BORING LOG	 GILES ENGINEERING ASSOCIATES, INC.
SURFACE ELEVATION: 100.2 feet		
COMPLETION DATE: 09/02/20		
FIELD REP: TYLER WELCH		
PUBLIC STORAGE 424 EAGLE ROCK AVENUE WEST ORANGE, NEW JERSEY PROJECT NO: 9G-2008002		

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q _u (tsf)	Q _p (tsf)	Q _s (tsf)	W (%)	PID	NOTES
4" Asphalt, 2" Crushed Stone Base FILL: Black and Brown Silty fine to coarse Sand and Gravel, contains Cobbles and Boulders - Moist to Very Moist		95	1-SS	50/4*				26	BDL	P200=21.6% (See Figure 2)
			2-SS	29**				11	BDL	
			3-SS	39**				17	BDL	
			4-SS	10				10	BDL	

Boring Terminated at about 8 feet (EL. 92.2')

Water Observation Data			Remarks:
	Water Encountered During Drilling: 5.5 ft.		Auger refusal at 6 feet. Split spoon sampler advanced to 8 feet. * N-value elevated due to interferene with cobbles and/or boulders ** N-value likely elevated due to interferene with cobbles and/or boulders
	Water Level At End of Drilling: 4 ft.		
	Cave Depth At End of Drilling: 5 ft.		
	Water Level After Drilling:		
	Cave Depth After Drilling:		

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION: 3D	TEST BORING LOG	 GILES ENGINEERING ASSOCIATES, INC.
SURFACE ELEVATION: 100.7 feet		
COMPLETION DATE: 09/03/20		
FIELD REP: TYLER WELCH		
PUBLIC STORAGE 424 EAGLE ROCK AVENUE WEST ORANGE, NEW JERSEY PROJECT NO: 9G-2008002		

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q _u (tsf)	Q _p (tsf)	Q _s (tsf)	W (%)	PID	NOTES
3" Asphalt, 2" Crushed Stone Base		100								
Silty Gravel and fine to coarse Sand, contains Cobbles and Boulders and Wood Fragments										
FILL: Silty Gravel and fine to coarse Sand, contains Cobbles and Boulders	5	95	1-RC							REC=48%
FILL: Dark Grey Lean Clay, some Silt, little fine to coarse Sand, trace Organics - Very Moist	10	90	1-SS	50/3*				38	BDL	REC=100% RQD=78%
BASALT BEDROCK, Dark Grey, moderately to slightly fractured, Partially to moderately healed, hard to very hard rock			2-RC							

Boring Terminated at about 14 feet (EL. 86.7')

Water Observation Data		Remarks:
	Water Encountered During Drilling:	* N-value elevated due to interferene with cobbles and/or boulders
	Water Level At End of Drilling:	
	Cave Depth At End of Drilling:	
	Water Level After Drilling:	
	Cave Depth After Drilling:	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

APPENDIX B

FIELD PROCEDURES

The field operations were conducted in general accordance with the procedures recommended by the American Society for Testing and Materials (ASTM) designation D

420 entitled "Standard Guide for Sampling Rock and Rock" and/or other relevant specifications. Soil samples were preserved and transported to *Giles'* laboratory in general accordance with the procedures recommended by ASTM designation D 4220 entitled "Standard Practice for Preserving and Transporting Soil Samples." Brief descriptions of the sampling, testing and field procedures commonly performed by *Giles* are provided herein.

GENERAL FIELD PROCEDURES

Test Boring Elevations

The ground surface elevations reported on the Test Boring Logs are referenced to the assumed benchmark shown on the Boring Location Plan (Figure 1). Unless otherwise noted, the elevations were determined with a conventional hand-level and are accurate to within about 1 foot.

Test Boring Locations

The test borings were located on-site based on the existing site features and/or apparent property lines. Dimensions illustrating the approximate boring locations are reported on the Boring Location Plan (Figure 1).

Water Level Measurement

The water levels reported on the Test Boring Logs represent the depth of “free” water encountered during drilling and/or after the drilling tools were removed from the borehole. Water levels measured within a granular (sand and gravel) soil profile are typically indicative of the water table elevation. It is usually not possible to accurately identify the water table elevation with cohesive (clayey) soils, since the rate of seepage is slow. The water table elevation within cohesive soils must therefore be determined over a period of time with groundwater observation wells.

It must be recognized that the water table may fluctuate seasonally and during periods of heavy precipitation. Depending on the subsurface conditions, water may also become perched above the water table, especially during wet periods.

Borehole Backfilling Procedures

Each borehole was backfilled upon completion of the field operations. If potential contamination was encountered, and/or if required by state or local regulations, boreholes were backfilled with an “impervious” material (such as bentonite slurry). Borings that penetrated pavements, sidewalks, etc. were “capped” with Portland Cement concrete, asphaltic concrete, or a similar surface material. It must, however, be recognized that the backfill material may settle, and the surface cap may subside, over a period of time. Further backfilling and/or re-surfacing by *Giles’* client or the property owner may be required.



FIELD SAMPLING AND TESTING PROCEDURES

Auger Sampling (AU)

Soil samples are removed from the auger flights as an auger is withdrawn above the ground surface. Such samples are used to determine general soil types and identify approximate soil stratifications. Auger samples are highly disturbed and are therefore not typically used for geotechnical strength testing.

Split-Barrel Sampling (SS) – (ASTM D-1586)

A split-barrel sampler with a 2-inch outside diameter is driven into the subsoil with a 140-pound hammer free-falling a vertical distance of 30 inches. The summation of hammer-blows required to drive the sampler the final 12-inches of an 18-inch sample interval is defined as the “Standard Penetration Resistance” or N-value is an index of the relative density of granular soils and the comparative consistency of cohesive soils. A soil sample is collected from each SPT interval.

Shelby Tube Sampling (ST) – (ASTM D-1587)

A relatively undisturbed soil sample is collected by hydraulically advancing a thin-walled Shelby Tube sampler into a soil mass. Shelby Tubes have a sharp cutting edge and are commonly 2 to 5 inches in diameter.

Bulk Sample (BS)

A relatively large volume of soils is collected with a shovel or other manually-operated tool. The sample is typically transported to *Giles’* materials laboratory in a sealed bag or bucket.

Dynamic Cone Penetration Test (DC) – (ASTM STP 399)

This test is conducted by driving a 1.5-inch-diameter cone into the subsoil using a 15-pound steel ring (hammer), free-falling a vertical distance of 20 inches. The number of hammer-blows required to drive the cone 1¾ inches is an indication of the soil strength and density, and is defined as “N”. The Dynamic Cone Penetration test is commonly conducted in hand auger borings, test pits and within excavated trenches.

- Continued -



GILES ENGINEERING ASSOCIATES, INC.

Ring-Lined Barrel Sampling – (ASTM D 3550)

In this procedure, a ring-lined barrel sampler is used to collect soil samples for classification and laboratory testing. This method provides samples that fit directly into laboratory test instruments without additional handling/disturbance.

Sampling and Testing Procedures

The field testing and sampling operations were conducted in general accordance with the procedures recommended by the American Society for Testing and Materials (ASTM) and/or other relevant specifications. Results of the field testing (i.e. N-values) are reported on the Test Boring Logs. Explanations of the terms and symbols shown on the logs are provided on the appendix enclosure entitled "General Notes".



APPENDIX C

LABORATORY TESTING AND CLASSIFICATION

The laboratory testing was conducted under the supervision of a geotechnical engineer in accordance with the procedures recommended by the American Society for Testing and Materials (ASTM) and/or other relevant specifications. Brief descriptions of laboratory tests commonly performed by *Giles* are provided herein.

LABORATORY TESTING AND CLASSIFICATION

Photoionization Detector (PID)

In this procedure, soil samples are “scanned” in *Giles’* analytical laboratory using a Photoionization Detector (PID). The instrument is equipped with an 11.7 eV lamp calibrated to a Benzene Standard and is capable of detecting a minute concentration of **certain** Volatile Organic Compound (VOC) vapors, such as those commonly associated with petroleum products and some solvents. Results of the PID analysis are expressed in HNu (manufacturer’s) units rather than actual concentration.

Moisture Content (w) (ASTM D 2216)

Moisture content is defined as the ratio of the weight of water contained within a soil sample to the weight of the dry solids within the sample. Moisture content is expressed as a percentage.

Unconfined Compressive Strength (qu) (ASTM D 2166)

An axial load is applied at a uniform rate to a cylindrical soil sample. The unconfined compressive strength is the maximum stress obtained or the stress when 15% axial strain is reached, whichever occurs first.

Calibrated Penetrometer Resistance (qp)

The small, cylindrical tip of a hand-held penetrometer is pressed into a soil sample to a prescribed depth to measure the soils capacity to resist penetration. This test is used to evaluate unconfined compressive strength.

Vane-Shear Strength (qs)

The blades of a vane are inserted into the flat surface of a soil sample and the vane is rotated until failure occurs. The maximum shear resistance measured immediately prior to failure is taken as the vane-shear strength.

Loss-on-Ignition (ASTM D 2974; Method C)

The Loss-on-Ignition (L.O.I.) test is used to determine the organic content of a soil sample. The procedure is conducted by heating a dry soil sample to 440°C in order to burn-off or “ash” organic matter present within the sample. The L.O.I. value is the ratio of the weight loss due to ignition compared to the initial weight of the dry sample. L.O.I. is expressed as a percentage.



Particle Size Distribution (ASTB D 421, D 422, and D 1140)

This test is performed to determine the distribution of specific particle sizes (diameters) within a soil sample. The distribution of coarse-grained soil particles (sand and gravel) is determined from a "sieve analysis," which is conducted by passing the sample through a series of nested sieves. The distribution of fine-grained soil particles (silt and clay) is determined from a "hydrometer analysis" which is based on the sedimentation of particles suspended in water.

Consolidation Test (ASTM D 2435)

In this procedure, a series of cumulative vertical loads are applied to a small, laterally confined soil sample. During each load increment, vertical compression (consolidation) of the sample is measured over a period of time. Results of this test are used to estimate settlement and time rate of settlement.

Classification of Samples

Each soil sample was visually-manually classified, based on texture and plasticity, in general accordance with the Unified Soil Classification System (ASTM D-2488-75). The classifications are reported on the Test Boring Logs.

Laboratory Testing

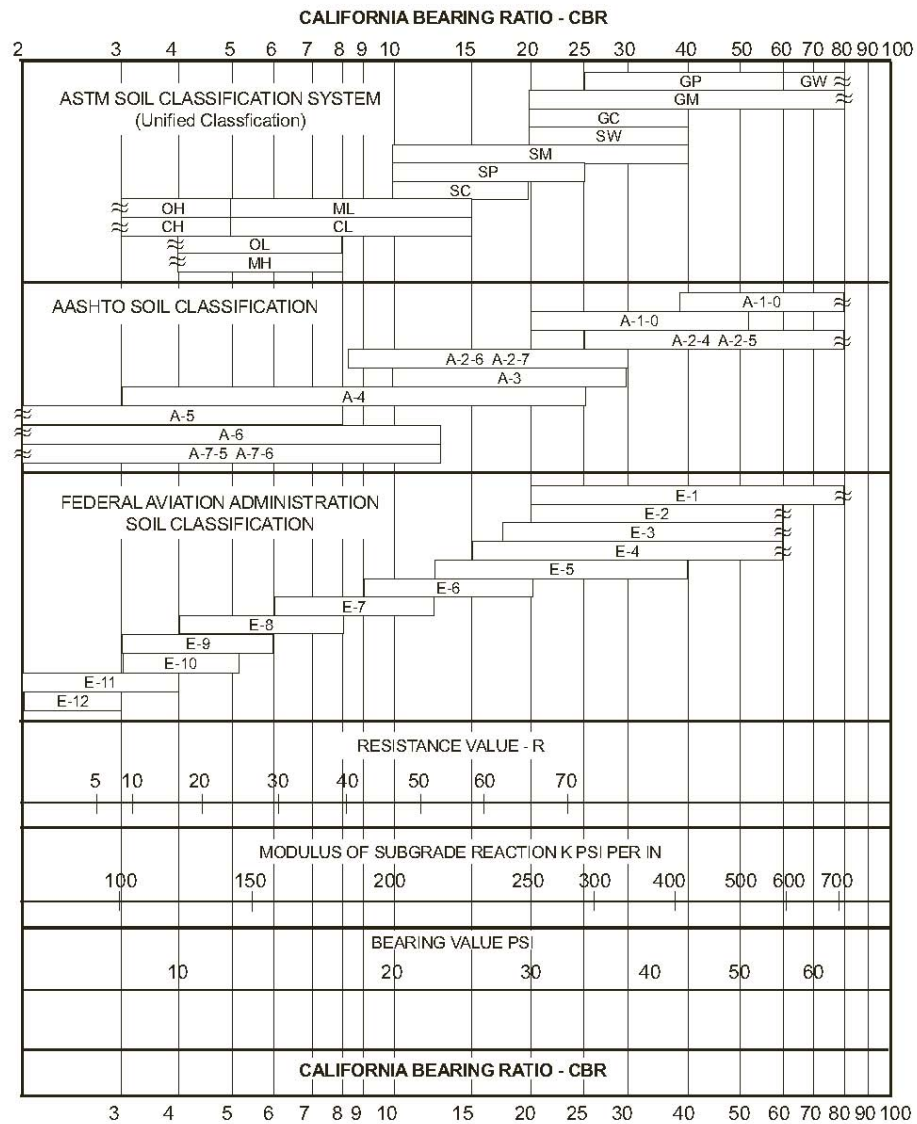
The laboratory testing operations were conducted in general accordance with the procedures recommended by the American Society for Testing and Materials (ASTM) and/or other relevant specifications. Results of the laboratory tests are provided on the Test Boring Logs or other appendix enclosures. Explanation of the terms and symbols used on the logs is provided on the appendix enclosure entitled "General Notes."



California Bearing Ratio (CBR) Test ASTM D-1833

The CBR test is used for evaluation of a soil subgrade for pavement design. The test consists of measuring the force required for a 3-square-inch cylindrical piston to penetrate 0.1 or 0.2 inch into a compacted soil sample. The result is expressed as a percent of force required to penetrate a standard compacted crushed stone.

Unless a CBR test has been specifically requested by the client, the CBR is estimated from published charts, based on soil classification and strength characteristics. A typical correlation chart is below.



APPENDIX D

GENERAL INFORMATION

AND

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL REPORT

GENERAL COMMENTS

The soil samples obtained during the subsurface exploration will be retained for a period of thirty days. If no instructions are received, they will be disposed of at that time.

This report has been prepared exclusively for the client in order to aid in the evaluation of this property and to assist the architects and engineers in the design and preparation of the project plans and specifications. Copies of this report may be provided to contractor(s), with contract documents, to disclose information relative to this project. The report, however, has not been prepared to serve as the plans and specifications for actual construction without the appropriate interpretation by the project architect, structural engineer, and/or civil engineer. Reproduction and distribution of this report must be authorized by the client and *Giles*.

This report has been based on assumed conditions/characteristics of the proposed development where specific information was not available. It is recommended that the architect, civil engineer and structural engineer along with any other design professionals involved in this project carefully review these assumptions to ensure they are consistent with the actual planned development. When discrepancies exist, they should be brought to our attention to ensure they do not affect the conclusions and recommendations provided herein. The project plans and specifications may also be submitted to *Giles* for review to ensure that the geotechnical related conclusions and recommendations provided herein have been correctly interpreted.

The analysis of this site was based on a subsoil profile interpolated from a limited subsurface exploration. If the actual conditions encountered during construction vary from those indicated by the borings, *Giles* must be contacted immediately to determine if the conditions alter the recommendations contained herein.

The conclusions and recommendations presented in this report have been promulgated in accordance with generally accepted professional engineering practices in the field of geotechnical engineering. No other warranty is either expressed or implied.



**GUIDE SPECIFICATIONS FOR SUBGRADE AND GRADE PREPARATION
FOR FILL, FOUNDATION, FLOOR SLAB AND PAVEMENT SUPPORT;
AND SELECTION, PLACEMENT AND COMPACTION OF FILL SOILS
USING STANDARD PROCTOR PROCEDURES**

1. Construction monitoring and testing of subgrades and grades for fill, foundation, floor slab and pavement; and fill selection, placement and compaction shall be performed by an experienced soils engineer and/or his representatives.
2. All compaction fill, subgrades and grades shall be (a) underlain by suitable bearing material; (b) free of all organic, frozen, or other deleterious material, and (c) observed, tested and approved by qualified engineering personnel representing an experienced soils engineer. Preparation of subgrades after stripping vegetation, organic or other unsuitable materials shall consist of (a) proof-rolling to detect soil, wet yielding soils or other unstable materials that must be undercut, (b) scarifying top 6 to 8 inches, (c) moisture conditioning the soils as required, and (d) recompaction to same minimum in-situ density required for similar materials indicated under Item 5. Note: compaction requirements for pavement subgrade are higher than other areas. Weather and construction equipment may damage compacted fill surface and reworking and retesting may be necessary to assure proper performance.
3. In overexcavation and fill areas, the compacted fill must extend (a) a minimum 1 foot lateral distance beyond the exterior edge of the foundation at bearing grade or pavement subgrade and down to compacted fill subgrade on a maximum 0.5(H):1(V) slope, (b) 1 foot above footing grade outside the building, and (c) to floor subgrade inside the building. Fill shall be placed and compacted on a 5(H):1(V) slope or must be stepped or benched as required to flatten if not specifically approved by qualified personnel under the direction of an experienced soil engineer.
4. The compacted fill materials shall be free of deleterious, organic, or frozen matter, shall contain no chemicals that may result in the material being classified as "contaminated", and shall be low-expansive with a maximum Liquid Limit (ASTM D-423) and Plasticity Index (ASTM D-424) of 30 and 15, respectively, unless specifically tested and found to have low expansive properties and approved by an experienced soils engineer. The top 12 inches of compacted fill should have a maximum 3-inch-particle diameter and all underlying compacted fill a maximum 6-inch-diameter unless specifically approved by an experienced soils engineer. All fill materials must be tested and approved under the direction of an experienced soils engineer prior to placement. If the fill is to provide non-frost susceptible characteristics, it must be classified as a clean GW, GP, SW or SP per the Unified Soil Classification System (ASTM D-2487).
5. For structural fill depths less than 20 feet, the density of the structural compacted fill and scarified subgrade and grades shall not be less than 95 percent of the maximum dry density as determined by Standard Proctor (ASTM-698) with the exception of the top 12 inches of pavement subgrade which shall have a minimum in-situ density of 100 percent of maximum dry density, or 5 percent higher than underlying fill materials. Where the structural fill depth is greater than 20 feet, the portions below 20 feet should have a minimum in-place density of 100 percent of its maximum dry density of 5 percent greater than the top 20 feet. The moisture content of cohesive soil shall not vary by more than -1 to +3 percent and granular soil ± 3 percent of the optimum when placed and compacted or recompacted, unless specifically recommended/approved by the soils engineer monitoring the placement and compaction. Cohesive soils with moderate to high expansion potentials ($PI > 15$) should, however, be placed, compacted and maintained prior to construction at a moisture content 3 ± 1 percent above optimum moisture content to limit further heave. The fill shall be placed in layers with a maximum loose thickness of 8 inches for foundations and 10 inches for floor slabs and pavement, unless specifically approved by the soils engineer taking into consideration the type of materials and compaction equipment being used. The compaction equipment should consist of suitable mechanical equipment specifically designed for soil compaction. Bulldozers or similar tracked vehicles are typically not suitable for compaction.
6. Excavation, filling, subgrade and grade preparation shall be performed in a manner and sequence that will provide drainage at all times and proper control of erosion. Precipitation, springs and seepage water encountered shall be pumped or drained to provide a suitable working platform. Springs or water seepage encountered during grading/foundation construction must be called to the soil engineer's attention immediately for possible construction procedure revision or inclusion of an underdrain system.
7. Non-structural fill adjacent to structural fill should typically be placed in unison to provide lateral support. Backfill along walls must be placed and compacted with care to ensure excessive unbalanced lateral pressures do not develop. The type of fill material placed adjacent to below-grade walls (i.e. basement walls and retaining walls) must be properly tested and approved by an experienced soils engineer with consideration for the lateral pressure used in the wall design.
8. Whenever, in the opinion of the soils engineer or the Owner's Representatives, an unstable condition is being created either by cutting or filling, the work shall not proceed into that area until an appropriate geotechnical exploration and analysis has been performed and the grading plan revised, if found necessary.



CHARACTERISTICS AND RATINGS OF UNIFIED SOIL SYSTEM CLASSES FOR SOIL CONSTRUCTION *									
Class	Compaction Characteristics	Max. Dry Density Standard Proctor (pcf)	Compressibility and Expansion	Drainage and Permeability	Value as an Embankment Material	Value as Subgrade When Not Subject to Frost	Value as Base Course	Value as Temporary Pavement	
								With Dust Palliative	With Bituminous Treatment
GW	Good: tractor, rubber-tired, steel wheel or vibratory roller	125-135	Almost none	Good drainage, pervious	Very stable	Excellent	Good	Fair to poor	Excellent
GP	Good: tractor, rubber-tired, steel wheel or vibratory roller	115-125	Almost none	Good drainage, pervious	Reasonably stable	Excellent to good	Poor to fair	Poor	
GM	Good: rubber-tired or light sheepsfoot roller	120-135	Slight	Poor drainage, semipervious	Reasonably stable	Excellent to good	Fair to poor	Poor	Poor to fair
GC	Good to fair: rubber-tired or sheepsfoot roller	115-130	Slight	Poor drainage, impervious	Reasonably stable	Good	Good to fair **	Excellent	Excellent
SW	Good: tractor, rubber-tired or vibratory roller	110-130	Almost none	Good drainage, pervious	Very stable	Good	Fair to poor	Fair to poor	Good
SP	Good: tractor, rubber-tired or vibratory roller	100-120	Almost none	Good drainage, pervious	Reasonably stable when dense	Good to fair	Poor	Poor	Poor to fair
SM	Good: rubber-tired or sheepsfoot roller	110-125	Slight	Poor drainage, impervious	Reasonably stable when dense	Good to fair	Poor	Poor	Poor to fair
SC	Good to fair: rubber-tired or sheepsfoot roller	105-125	Slight to medium	Poor drainage, impervious	Reasonably stable	Good to fair	Fair to poor	Excellent	Excellent
ML	Good to poor: rubber-tired or sheepsfoot roller	95-120	Slight to medium	Poor drainage, impervious	Poor stability, high density required	Fair to poor	Not suitable	Poor	Poor
CL	Good to fair: sheepsfoot or rubber-tired roller	95-120	Medium	No drainage, impervious	Good stability	Fair to poor	Not suitable	Poor	Poor
OL	Fair to poor: sheepsfoot or rubber-tired roller	80-100	Medium to high	Poor drainage, impervious	Unstable, should not be used	Poor	Not suitable	Not suitable	Not suitable
MH	Fair to poor: sheepsfoot or rubber-tired roller	70-95	High	Poor drainage, impervious	Poor stability, should not be used	Poor	Not suitable	Very poor	Not suitable
CH	Fair to poor: sheepsfoot roller	80-105	Very high	No drainage, impervious	Fair stability, may soften on expansion	Poor to very poor	Not suitable	Very poor	Not suitable
OH	Fair to poor: sheepsfoot roller	65-100	High	No drainage, impervious	Unstable, should not be used	Very poor	Not suitable	Not suitable	Not suitable
Pt	Not suitable		Very high	Fair to poor drainage	Should not be used	Not suitable	Not suitable	Not suitable	Not suitable

* "The Unified Classification: Appendix A - Characteristics of Soil, Groups Pertaining to Roads and Airfields, and Appendix B - Characteristics of Soil Groups Pertaining to Embankments and Foundations," Technical Memorandum 357, U.S. Waterways Experiment Station, Vicksburg, 1953.

** Not suitable if subject to frost.



UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487)

Major Divisions		Group Symbols	Typical Names	Laboratory Classification Criteria	
Coarse-grained soils (more than half of material is larger than No. 200 sieve size)	Gravels (More than half of coarse fraction is larger than No. 4 sieve size)	Clean gravels (little or no fines)	GW	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			GP	Not meeting all gradation requirements for GW	
		Gravels with fines (appreciable amount of fines)	GM ^a d	Atterberg limits below "A" line or P.I. less than 4	Limits plotting within shaded area, above "A" line with P.I. between 4 and 7 are <i>borderline</i> cases requiring use of dual symbols
			GM ^a u		
	Sands (More than half of coarse fraction is smaller than No. 4 sieve size)		GC	Atterberg limits above "A" line or P.I. greater than 7	
		Clean sands (little or no fines)	SW	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			SP	Not meeting all gradation requirements for SW	
		Sands with fines (Appreciable amount of fines)	SM ^a d	Atterberg limits below "A" line or P.I. less than 4	Limits plotting within shaded area, above "A" line with P.I. between 4 and 7 are <i>borderline</i> cases requiring use of dual symbols
			SM ^a u		
			SC	Atterberg limits above "A" line or P.I. greater than 7	
Fine-grained soils (More than half material is smaller than No. 200 sieve size)	Silts and clays (Liquid limit less than 50)		ML	Plasticity Chart	
			CL		
			OL		
	Silts and clays (Liquid limit greater than 50)		MH		
			CH		
			OH		
	Highly organic soils		Pt		

^a Division of GM and SM groups into subdivisions of d and u are for roads and airfields only. Subdivision is based on Atterberg limits, suffix d used when L.L. is 28 or less and the P.I. is 6 or less; the suffix u is used when L.L. is greater than 28.

^b Borderline classifications, used for soils possessing characteristics of two groups, are designated by combinations of group symbols. For example GW-GC, well-graded gravel-sand mixture with clay binder.

GENERAL NOTES

SAMPLE IDENTIFICATION

All samples are visually classified in general accordance with the Unified Soil Classification System (ASTM D-2487-75 or D-2488-75)

DESCRIPTIVE TERM (% BY DRY WEIGHT)

Trace:	1-10%
Little:	11-20%
Some:	21-35%
And/Adjective	36-50%

PARTICLE SIZE (DIAMETER)

Boulders:	8 inch and larger
Cobbles:	3 inch to 8 inch
Gravel:	coarse - ¾ to 3 inch fine - No. 4 (4.76 mm) to ¾ inch
Sand:	coarse - No. 4 (4.76 mm) to No. 10 (2.0 mm) medium - No. 10 (2.0 mm) to No. 40 (0.42 mm) fine - No. 40 (0.42 mm) to No. 200 (0.074 mm)
Silt:	No. 200 (0.074 mm) and smaller (non-plastic)
Clay:	No 200 (0.074 mm) and smaller (plastic)

SOIL PROPERTY SYMBOLS

Dd:	Dry Density (pcf)
LL:	Liquid Limit, percent
PL:	Plastic Limit, percent
PI:	Plasticity Index (LL-PL)
LOI:	Loss on Ignition, percent
Gs:	Specific Gravity
K:	Coefficient of Permeability
w:	Moisture content, percent
qp:	Calibrated Penetrometer Resistance, tsf
qs:	Vane-Shear Strength, tsf
qu:	Unconfined Compressive Strength, tsf
qc:	Static Cone Penetrometer Resistance (correlated to Unconfined Compressive Strength, tsf)
PID:	Results of vapor analysis conducted on representative samples utilizing a Photoionization Detector calibrated to a benzene standard. Results expressed in HNU-Units. (BDL=Below Detection Limit)

N:	Penetration Resistance per 12 inch interval, or fraction thereof, for a standard 2 inch O.D. (1½ inch I.D.) split spoon sampler driven with a 140 pound weight free-falling 30 inches. Performed in general accordance with Standard Penetration Test Specifications (ASTM D-1586). N in blows per foot equals sum of N-Values where plus sign (+) is shown.
Nc:	Penetration Resistance per 1¾ inches of Dynamic Cone Penetrometer. Approximately equivalent to Standard Penetration Test N-Value in blows per foot.
Nr:	Penetration Resistance per 12 inch interval, or fraction thereof, for California Ring Sampler driven with a 140 pound weight free-falling 30 inches per ASTM D-3550. Not equivalent to Standard Penetration Test N-Value.

DRILLING AND SAMPLING SYMBOLS

SS:	Split-Spoon
ST:	Shelby Tube - 3 inch O.D. (except where noted)
CS:	3 inch O.D. California Ring Sampler
DC:	Dynamic Cone Penetrometer per ASTM Special Technical Publication No. 399
AU:	Auger Sample
DB:	Diamond Bit
CB:	Carbide Bit
WS:	Wash Sample
RB:	Rock-Roller Bit
BS:	Bulk Sample
Note:	Depth intervals for sampling shown on Record of Subsurface Exploration are not indicative of sample recovery, but position where sampling initiated

SOIL STRENGTH CHARACTERISTICS

COHESIVE (CLAYEY) SOILS

COMPARATIVE CONSISTENCY	BLOWS PER FOOT (N)	UNCONFINED COMPRESSIVE STRENGTH (TSF)
Very Soft	0 - 2	0 - 0.25
Soft	3 - 4	0.25 - 0.50
Medium Stiff	5 - 8	0.50 - 1.00
Stiff	9 - 15	1.00 - 2.00
Very Stiff	16 - 30	2.00 - 4.00
Hard	31+	4.00+

NON-COHESIVE (GRANULAR) SOILS

RELATIVE DENSITY	BLOWS PER FOOT (N)
Very Loose	0 - 4
Loose	5 - 10
Firm	11 - 30
Dense	31 - 50
Very Dense	51+

DEGREE OF PLASTICITY	PI	DEGREE OF EXPANSIVE POTENTIAL	PI
None to Slight	0 - 4	Low	0 - 15
Slight	5 - 10	Medium	15 - 25
Medium	11 - 30	High	25+
High to Very High	31+		



Important Information About Your Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

The following information is provided to help you manage your risks.

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 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 your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you — should apply the 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.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of 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,

- elevation, configuration, location, orientation, or weight of the proposed structure,
- 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 study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the 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 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 construction recommendations included in your report. *Those 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 recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower 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. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing 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 Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors 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 contractors have sufficient time to perform additional study.* Only then might you be in a position to give contractors 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 contractors do not 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.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental 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 geoenvironmental 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, a number of 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 ASFE-Member Geotechnical Engineer for Additional Assistance

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with you ASFE-member geotechnical engineer for more information.



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