Geotechnical Engineering Report

Apogee Townhomes
(Clay Street Development)
Montgomery County, Virginia

DAA Project Number: 18010224-020203

April 23, 2021

Prepared for
Blacksburg, LLC
510 Floyd Street
Blacksburg, Virginia 24060

Draper Aden Associates
Engineering • Surveying • Environmental Services

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April 23, 2021

Mr. Matt Chamberlain
Blacksburg, LLC
510 Floyd Street
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RE: Geotechnical Engineering Report
Apogee Townhomes (Clay Street Development)
Montgomery County, Virginia
Draper Aden Associates Project No. 18010224-020203

Dear Mr. Chamberlain:

In general accordance with our proposal, last revised April 1, 2020 and your subsequent authorization on April 3, 2020, Draper Aden Associates (DAA) has performed a geotechnical study for the proposed Apogee Townhomes residential development (previously known as the Clay Street Development), planned in the Blacksburg area of Montgomery County, Virginia. This study included a geotechnical field exploration, limited laboratory testing, and an engineering evaluation. The results of the field and laboratory testing, as well as our conclusions and recommendations for the proposed improvements, are presented in this report.

DAA will discard all soil samples associated with this project 60 days after the issue of this report unless other arrangements are made by the Client. DAA appreciates the opportunity to have been of assistance to you on this project. Should you have any questions or require additional information, please do not hesitate to contact our office.

Sincerely,
Draper Aden Associates
Geotechnical and Construction Services

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1.0 INTRODUCTION

1.1 Project Information

Blacksburg, LLC (the "Client") is planning the construction of the Apogee Townhomes residential development (formerly known as the Clay Street Development) in the Blacksburg area of Montgomery County, Virginia. The proposed development is planned to include 34 townhome units (five townhome buildings or "sticks") and associated site infrastructure, including roadways and parking areas, subsurface utilities, stormwater management facilities, and other improvements. Draper Aden Associates was retained to perform a geotechnical study for the proposed development. This study was performed in general accordance with our Proposal, dated April 1, 2020, and your subsequent authorization on April 3, 2020.

In conjunction with this study, DAA was provided with preliminary zoning plans for the proposed development, which show a schematic of the proposed site layout, as well as aerial imagery of the site and the existing site topography. These plans did not show a proposed site grading scheme or other specific details. At the time these preliminary zone plans were provided, DAA was advised to hold off on the exploration program until more detailed plans were available. Subsequently, in January of 2021, DAA was provided with selected sheets (Sheets C3 through C-7) of the site development plans, dated January 18, 2021, which show in greater detail the proposed site layout, as well as the proposed grading scheme. The plans also show the existing site topography and limited existing site features.

DAA has previously performed a geophysical study of the project site, which is summarized in our report dated September 20, 2018. The prior study included electrical resistivity imaging of the subsurface along eight lines, and was reviewed in conjunction with the preparation of this geotechnical report.

1.2 Objective and Scope of Work

The objective of this study was to provide information to generally characterize subsurface conditions and develop geotechnical engineering recommendations for design and construction of the proposed development.

Our scope of services included:

- A subsurface exploration program consisting of 17 Standard Penetration Test (SPT) borings (including four added at offset locations due to shallow refusal at the planned locations) and two auger probe borings (with no SPT or soil sampling) extended to depths ranging from approximately 1 to 26.5 feet below existing grades.
- Laboratory testing of representative split-spoon soil samples in order to develop pertinent data related to the on-site soils to support our design recommendations.
- Preparation of this geotechnical engineering report, which summarizes our geotechnical exploration program, laboratory testing, and geotechnical engineering recommendations. The relevant key findings of our geophysical study, performed prior to the geotechnical exploration program, were also incorporated into this report.

2.0 SITE CONDITIONS AND PROPOSED CONSTRUCTION

2.1 Existing Site Conditions

The project site is located along the north side of Clay Street, west of its intersection with Cherry Lane, in the Blacksburg area of Montgomery County, Virginia. Please refer to the Site Location Map, included as Figure 1 in Section 1 of the Appendices.

At the time of our field exploration, the site generally consisted of an open, grass-covered field with trees along the northern, southern, and western margins of the site. Several rock outcroppings were observed at the existing ground surface throughout the site. A pile of stumps, logs, and brush was present in the central portion of the site, and an area of stockpiled soil and rocks was present in the northwestern portion of the site. In addition, numerous small depressions, generally on the order of 1 foot deep and 3 to 5 feet wide, were observed throughout the site. Historic aerial imagery obtained from Google Earth show an apparent pattern or grid of small depressions across the site, suggestive of a possible prior test pit exploration program. Though this pattern is most visually apparent in imagery from 2007, 2009, and 2011, close inspection of imagery from as early as 2000 appears to reveal faint indications of the pattern. No documentation of a prior exploration program, or other prior earthwork or fill placement, was available at the time this report was prepared.

Photo 1: View of the project site from near its southwest corner, facing east.

Topographically, the site can be generally characterized as gently to moderately sloping downward to the northeast and southeast. Existing site grades range from a high on the order of 2,280 feet above Mean Sea Level (MSL) near the southwest corner of the site to a low of about 2,235 feet above MSL along its northern boundary. Surface water in the northern and western portions of the site is generally expected to flow toward the north while surface water in the southern portion of the site is generally expected to flow toward the south and east.

2.2 Proposed Construction

Based on a review of the above-referenced plans, the proposed development will include 34 townhome units (five townhome buildings or "sticks") with slabs-on-grade at the bottom level. Though specific details of the proposed townhome buildings were not provided, we have assumed that the proposed residential structures will be relatively lightly-loaded, timber-framed structures.

The proposed development will be accessed from an entrance road extending into the development from Clay Street, and two additional interior drive lanes. Although the townhomes will have individual driveways at each unit, additional parking is planned adjacent to one of the interior drive lanes. DAA has assumed that the proposed roadways will be privately maintained. A community recreation area is planned in the central portion of the site. Although not shown on the plans, we understand from information provided by the Client that a pool may be planned in the community recreation area. The proposed development will be served by public water and sewer utilities. In addition, a site storm drain system is planned to convey stormwater runoff to two below-grade stormwater retention facilities.

The proposed grading scheme and utility profiles shown on the available plans indicate that mass grading cuts and fills typically on the order of 5 feet or less, and locally approaching 10 feet, will be required to establish finished grades for the proposed development. In addition, the utility profiles indicate that excavations generally on the order of 5 to 10 feet below existing grades, and locally approaching 15 feet, are expected to be required for construction of the proposed utilities and stormwater management facilities. The plans also show multiple proposed slopes around the building pads, with some as steep as 2H:1V (horizontal to vertical).

3.0 SUBSURFACE EXPLORATION

3.1 Prior Geophysical Electrical Resistivity Imaging (ERI) Study

As noted above, DAA has previously performed a geophysical electrical resistivity imaging (ERI) study of the project site, summarized in our report dated September 20, 2018, which was reviewed in conjunction with this study. The prior ERI study was performed to provide cross-sectional imaging of the subsurface and allow for inference of some geologic conditions. The ERI field survey was performed using a dipole-dipole array to provide optimum vertical resolution across the project area. An Advanced Geosciences, Inc. (AGI) SuperSting™ R8 IP® multi-electrode system was utilized to collect the field data. Eight (8) resistivity lines were planned to provide representative coverage of the site as detailed site development or grading plans were not yet available.

The field resistivity data was processed utilizing inversion modeling techniques with AGI EarthImager 2D software, producing an Earth model which provides an approximation of subsurface conditions. The results of the geophysical ERI study are presented in our prior ERI study report, dated September 20, 2018.

3.2 Geotechnical Field Exploration Program

To characterize subsurface conditions at the site and supplement the prior geophysical study, DAA performed a geotechnical field exploration program on March 22 and 23, 2021. The exploration program included 17 Standard Penetration Test (SPT) borings (including four added to the exploration program due to shallow refusal at the planned locations) and two auger probe borings (with no SPT or soil sampling). The explorations were designated as Borings B-01 through B-13, B-05A, B-09A, B-10A, B-11A, B-12A, and B-13A. The exploration locations were selected by DAA in consultation with the Client to provide representative coverage of the proposed development areas and were field-located by DAA personnel using a hand-held global positioning system (GPS) unit. Please refer to the Exploration Location Plan, included as Figure 2 in Section 1 of the Appendices. The existing ground surface elevations at the exploration locations are based on the topography shown on the above-referenced provided plans and should be considered approximate.

The field exploration was performed using a subcontracted, track-mounted CME-55 drill rig, operated by Blue Ridge Drilling and equipped with hollow-stem augers and split-spoon sampler. At each SPT boring location, the hollow-stem augers were advanced to the top of the selected test interval, Standard Penetration Testing was performed, and split-spoon soil samples were collected by driving a 1⅜-inch-I.D. split spoon sampler, in accordance with ASTM D1586-11, *Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils*.

SPT tests were performed and soil samples taken at 2.5-foot intervals within the top 10 feet of each boring, and at 5-foot intervals thereafter. For each SPT test, the sampler was first seated 6 inches to penetrate loose cuttings at the bottom of the hollow-stem augers, and then driven an additional 12 inches with a 140-pound hammer free falling 30 inches. The standard penetration resistance, or N-value, designates the number of hammer blows required to drive the sampler through the second and third intervals. The N-value, reported in blows-per-foot (bpf), provides an indication of the relative density or comparative consistency of the subsurface soil, allowing for estimation of the approximate shear strength and other soil properties through empirical geotechnical correlations. The CME-55 drill rig utilizes an automatic hammer for the Standard Penetration Test. Automatic hammers typically produce approximately 30 percent more energy than traditional safety hammers. While the N-values reported on the boring logs represent the raw, uncorrected data, the increased energy produced by the automatic hammer has been accounted for in our analyses.

Groundwater levels and cave-in depths in the borings were recorded at the completion of drilling and one day later, except in the borings performed on the final day of drilling which were backfilled upon completion. After the final groundwater observations were made, the boreholes were backfilled with available soil cuttings. The soil samples recovered from the borings were delivered to DAA's laboratory for visual/manual classification and limited laboratory testing.

4.0 LABORATORY TEST RESULTS

The soil samples obtained during the geotechnical field exploration program were placed in labeled sample containers that were sealed to reduce moisture loss and delivered to DAA's U.S. Army Corps of Engineers (USACE) verified materials testing laboratory for further visual review and limited laboratory testing. Selected representative soil samples were subjected to the following laboratory tests:

Table 1: Laboratory Test Items and Related ASTM Standards

Test Items	Standard Name
Soil Natural Moisture Contents	ASTM D2216
Atterberg Limits	ASTM D4318
Gradation Analysis	ASTM D6913
Soil Classification	ASTM D2487

The natural (in-situ) soil moisture contents of the samples tested ranged from 25.5 to 37.1 percent. In addition to the soil moisture content testing, selected soil samples were subjected to index property testing, including grain-size analysis and plasticity testing. The following table summarizes the results of the soil index property testing.

Table 2: Summary of Classification Results

Sample ID	Sample Depth (ft)	Natural Moisture Content (%)	% Passing the No. 200 Sieve	Atterberg Limits			USCS Classification
				LL	PL	PI	
B-07	6.0-7.5	37.1	99.2	115	33	82	Fat CLAY (CH)
B-13	3.5-5.0	27.3	97.2	52	21	31	Fat CLAY (CH)

Notes: LL = Liquid Limit; PL = Plastic Limit; PI = Plasticity Index

For more detailed information, please refer to the individual laboratory test reports included in Section 3 of the Appendices.

5.0 SUBSURFACE CONDITIONS

5.1 Regional Geology

The Valley and Ridge Physiographic Province near the site is generally composed of long parallel ridges and valleys comprised of folded and faulted sedimentary rocks. The ridges are typically composed of resistant sandstones and conglomerates, with the valleys being composed of less-resistant carbonate rocks and shale. The age of the formations within the Valley and Ridge province ranges from Cambrian to Mississippian (approximately 570-300 million years).

5.2 Local Geology

According to the Blacksburg quadrangle geologic map (Bartholomew and Lowry, 1979) [1], the project site is underlain by the Knox Group, which is described as light to medium gray, massive, thick-bedded, fine- to medium-grained dolomite with 1- to 6-foot thick chert interbeds. Nearby strike-and-dip symbols on the referenced geologic map suggest that the bedrock strata beneath the site strike northwest-southeast, and likely dip gently toward the east.

Soils derived from the carbonate bedrock underlying the site are typically rich in clay and may have higher concentrations of residual chert rock fragments resulting from solutional weathering of carbonate rich bedrock over geologic time. In addition, it should be noted that carbonate rocks are generally soluble by acidic groundwater; thus, it is possible that “karst” features, such as sinkholes, caverns, or other soil- or air-filled voids, may be present within the carbonate rocks underlying the site.

5.3 Encountered Soil Conditions

6.3.1 General

The boring logs that reflect the subsurface conditions at the time of the exploration program are included in Section 2 of the Appendices. Soil strata inferences, discussed below and indicated on the boring logs, represent an estimate of the subsurface conditions based on the Unified Soil Classification System (USCS) in accordance with ASTM D2487 and D2488. It should be noted that the transitions between the soil strata are generally less distinct than shown on the boring logs and are interpolated between the boring locations. For more specific subsurface information, please refer to the Geotechnical Exploration Summary Table, as well as the individual exploration logs, included in Section 2 of the Appendices.

6.3.2 Subsurface Soils

From the existing ground surface, the borings encountered a topsoil layer which ranged from about 4 to 10 inches thick. The underlying soil strata encountered at the exploration locations are described in the following sections:

Possible Existing Fill: Below the surficial topsoil layer, materials identified as possible existing fills were encountered in Borings B-01, B-02, and B-13 to depths of approximately 3 to 5 feet below existing grades. These materials were classified as clayey sands and sandy lean clays (SC, CL) with gravel. The clayey sands were characterized as loose to dense based on SPT N-values of 9 to 34 bpf, and the sandy clays were classified as stiff based on SPT N-values of 9 to 13 bpf.

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[1] Bartholomew, M.J., and Lowry, W.D., 1979, Geology of the Blacksburg Quadrangle, Virginia: Virginia Division of Mineral Resources, Publication 14, scale:1:24,000.

Native Residual Soils: Underlying the topsoil layer and possible exiting fills, where encountered, the borings encountered native residual soils of the Valley and Ridge Physiographic Province. These materials consisted of generally high-plasticity clays, classified predominantly and fat clay (CH) and, to a lesser extent, lean clay (CL), with variable proportions of sand and gravel. SPT N-values within these materials were predominantly within the range of 5 to 15 bpf, indicating generally medium stiff to stiff conditions. Limited silty gravel (GM) with sand was encountered near the refusal depth in Boring B-10, which was characterized as dense based on an SPT N-value of 34 bpf. It should be noted that relatively soft conditions were observed around depths of approximately 13.5 to 15 feet below existing grade in Boring B-03 and around 23.5 to 25 feet below existing grade in Boring B-08. Additionally, the soil samples obtained from the soft zone in Boring B-08 were noted to be wet.

Partially Weathered Rock: Materials identified as Partially Weathered Rock (PWR) were encountered immediately above the refusal depths in Borings B-09, B-09A, B-11, and B-12. PWR is an intermediate "geomaterial" between soil and rock which is characterized as residual materials exhibiting SPT N-values greater than 50 blows per 6 inches of split-spoon penetration. The PWR materials were visually classified as silty/clayey sands and gravels (SM, GC). The thickness of the PWR layer (between the apparent top of PWR and auger refusal on underlying bedrock) was typically only a few inches, indicating a relatively sudden transition between the overburden soils and underlying bedrock that is typical of the local geology.

Refusal: Auger refusal was encountered in the borings (except Boring B-01) at depths ranging from about 1 to 26½ feet below existing surface grades.

5.4 Groundwater Observations

Groundwater was encountered during drilling in Boring B-08 at a depth of about 6½ feet below existing surface grades, as evidenced by water on the split-spoon soil sampler upon retrieval from the borehole. Upon the completion of drilling (and one day later in the borings not performed on the final day of drilling), the borings were observed to be dry to their cave-in depths, which ranged from approximately 1½ to 24 feet below existing surface grades. It should be noted that the shallow cave-in depths, where observed, generally correspond to borings in which shallow auger refusal was encountered and are generally not believed to represent a near-surface groundwater condition.

It should be noted that the observed groundwater levels reflect the conditions only at the times of our observations. Groundwater levels are expected to fluctuate with seasonal variations in precipitation and may be impacted by construction activity. In addition, "perched" water may be present where trapped within granular soils or existing fills which are underlain by less permeable, fine-grained soils or rock.

5.5 Geophysical Electrical Resistivity Imaging (ERI) Results

The results of the geophysical ERI study were evaluated for interpreted top of rock and anomalies indicative of possible karst formation, and are summarized below:

Top of Rock:

As is common in karstified carbonate bedrock and confirmed by the geotechnical borings, the results of the WRI study suggest an irregular bedrock profile, with the interpreted bedrock surface ranging from near the surface to depths of approximately 28 feet below existing surface grades along the resistivity lines. The interpreted bedrock depths correspond to elevations on the order of 2,212 to 2,266 feet above MSL. The contour models included in the ERI report illustrate the lateral variations in the interpreted bedrock surface elevation and depth to bedrock.

Karst Formation:

The ERI study also suggests a low to moderate degree of karstification of the bedrock beneath the site. A pair of isolated high-resistivity zones were observed at depths at least 30 feet below existing surface grades, which may represent air-filled voids within the bedrock zone. However, no discernible pathways were observed beneath electrodes 2-17 and 4-42 which would allow surface soils to ravel downward into the deeper possible air-filled voids. Additionally, a possible soil-filled void was observed beneath electrodes 1-26 and 1-27. On either side of this feature (beneath electrodes 1-23 and 1-29), possible solutionally-enlarged pathways were observed which may allow soils to ravel downward toward the possible soil-filled void. Similarly, possible solutionally-enlarged pathways into the bedrock are observed beneath electrodes 3-6 and 3-17, although no anomalies indicative of possible soil- or air-filled voids were observed beneath them.

For more detailed information regarding the results of the ERI study, please consult the above-referenced ERI study report, dated September 20, 2018.

6.0 CONCLUSIONS AND RECOMMENDATIONS

6.1 General

Based on our understanding of the site conditions, proposed improvements, and our experience with similar projects, it is our opinion that construction of the proposed residential development is feasible, provided that the geotechnical recommendations presented herein are followed and the professional standard of care is maintained during construction. Construction of the proposed development will be impacted by the presence of generally high-plasticity, fine-grained soils which are likely moisture- and disturbance-sensitive. In addition, the presence of relatively shallow bedrock pinnacles and "floating" boulders, as well as potential sinkholes and/or other karst features in localized portions of the site, will also likely impact some aspects of construction. A discussion of these issues, as well as other geotechnical design and construction considerations, is presented in the following sections.

The following conclusions and recommendations are based on the previously discussed project information, observations at the site, interpretations and analysis of the field and laboratory data, and our experience with similar subsurface conditions, using generally established correlations and methods commonly used by members of the geotechnical engineering profession. If the proposed project location or layout, grading scheme, loading conditions, or other pertinent information are changed, or differ from our assumptions, we should be contacted to review the updated project details and revise our recommendations as necessary. DAA should be provided with the final site plans, once available, to verify that the intent of the recommendations presented in this report is met.

6.2 Slope Stability Considerations

The proposed grades shown on the available site plan indicate that several relatively steep slopes, on the order of 2H:1V (horizontal to vertical), are planned throughout the site to facilitate grading of the proposed building pads and associated yards. The proposed slopes have heights typically on the order of 10 feet or less.

Slope stability factor of safety is calculated as the ratio of forces resisting slope failure to those driving slope failure. A factor of safety of 1.0 for slope stability theoretically represents imminent slope failure while a factor of safety within the range of 1.3 to 1.5 is generally considered acceptable for most applications, depending upon the level of risk that the project can tolerate and the level of confidence in the input parameters. Input parameters which impact slope stability include the finished slope surface geometry, subsurface conditions below and around the slope, soil type and strength, groundwater conditions, and other factors.

Some slopes in the proposed grading scheme will be unavoidable due to the naturally sloping site topography. Due to their potential for instability, slopes steeper than 3H:1V are generally not recommended unless specifically analyzed based on measured soil strengths, internal reinforcement, or other stabilizing measures. In addition, it should be noted that, depending upon the soil type/strength, slope geometry and height, and other factors, stabilizing measures may still be recommended or required, even for slopes of 3H:1V or flatter. The high-plasticity, clayey soils, which are predominant onsite, are expected to exhibit relatively low shear strengths, which may result in marginally low, possibly unacceptable, slope factors of safety. We recommend a slope stability study be performed to evaluate the stability of the proposed slope(s).

6.3 Karst Considerations

As previously noted, the referenced geologic map indicates that the project site is underlain by the Knox Group, which consists of dolomite with chert interbeds. Limestone and dolomite rocks are susceptible to karst formation because of the chemical reaction that takes place when carbonate rocks encounter slightly acidic rainwater or groundwater. This chemical reaction causes the carbonate rocks to dissolve more quickly than other minerals dissolve in water. The dissolution takes place primarily along bedding planes and joints as water percolates through those features. As the carbonates dissolve, the percolating water carries away the soluble components, leaving behind the insoluble clay minerals and silicates, enlarging the spaces through which the water flowed in the process. The remaining soils are often plastic, clayey soils, and may be soft and compressible.

The continued dissolution of carbonate rocks can sometimes result in open cavities in the rock. As these cavities grow, the overlying soils are susceptible to raveling into the underlying cavities, carried downward by the percolating water and the influence of gravity. As the surface soils ravel, the ground surface can subside and result in the gradual formation of closed depressions or sinkholes. This type of sinkhole is known as a cover-subsidence sinkhole and is usually characterized by imperceptible growth. As such, cover-subsidence sinkholes are often covered by vegetation in undeveloped areas.

Raveling at depth can also occur beneath surficial soils that bridge over the growing soil cavity. In this scenario, continued raveling enlarges the cavity until it eventually grows to the point where the surface soils cannot maintain the bridge, resulting in a sudden collapse of the surface soils. This type of sinkhole is known as a cover collapse sinkhole. These sinkholes tend to be less common than the cover subsidence type.

As discussed previously, the geophysical ERI study identified anomalous features, possibly indicative of solutionally-enlarged pathways into underlying soil-filled voids, beneath electrodes 1-23, 1-29, 3-6, and 3-17. Borings B-08, B-09, B-11, and B-13 were planned in the vicinity of these features to further characterize the subsurface conditions and potential for karst features in these areas. Boring B-08, performed near electrode 1-22 and near the northeast limit of proposed Townhome Unit 23, encountered wet soils at a depth of approximately 6½ feet below existing grade, followed by a soft, wet zone, exhibiting SPT N-values of 4 bpf, at a depth of approximately 23.5 to 25 feet below existing grade, just above the auger refusal depth of approximately 26½ feet. The conditions observed in this boring are characteristic of those expected in a solutionally-enlarged sinkhole throat. While the soft, wet zone is relatively deep and not expected to have a significant impact on the proposed construction, the potential exists that similar soft, wet conditions may be encountered at shallower depths in the vicinity of this boring where the apparent sinkhole throat has migrated toward the existing ground surface. Similarly, while not specifically identified as a possible sinkhole throat in the ERI report, the soft conditions encountered at approximately 13.5 to 15 feet in Boring B-03, just above the auger refusal depth of approximately 17 feet, may represent a solutionally-weathered zone of which similar conditions may exist, including at shallower depths, in the vicinity of this boring.

While inferences regarding the specific nature of the observed karst features can be made from geotechnical and geophysical data, the exact nature and extent of these features cannot be characterized with complete certainty. Additionally, the potential exists that additional karst features (soil- or air-filled voids, solutionally-enlarged pathways or sinkhole throats, near-surface “floating” boulders, shallow bedrock pinnacles, etc.) may be present throughout the site, including in unexplored areas and/or at shallower depths, which may impact some aspects of construction, particularly with respect to structural support and drainage. As such, the risk that karst features may impact construction cannot be eliminated. The project owner, developer, and contractors should understand that these risks exist and are inherent to development in karst-prone areas.

If karst-related problems develop during construction, they should be evaluated by the project geotechnical engineer. Remediation techniques, if required, will be developed based on the specific conditions encountered during construction. Additional geophysical and/or geotechnical testing may be required to further characterize the nature and extent of problem areas encountered during construction. The additional exploration and analysis, development of remediation techniques, preparation or modification of construction plans and specifications, if required, and implementation of the remedial measures may result in significant additional project costs and delays. The owner and developer should consider these potential impacts while establishing the project budget and schedule.

In our experience, karst-related challenges are most likely to develop where the site subsurface is significantly disturbed and/or exposed to new or different drainage conditions. Thus, the potential for karst-related challenges during construction can likely be reduced by minimizing the required extent and depth of mass grading cuts, by avoiding the use of blasting for rock removal, if possible, and by minimizing changes to the site drainage characteristics as well as the collection and introduction of water into the subsurface. We recommend that the project civil engineer should consider the potential karst-related implications when finalizing the site grading scheme and detailing the temporary and permanent stormwater management systems.

6.4 Site Preparation and Earthwork

Based on a review of the existing and proposed site grades, a combination of cuts and fills typically on the order of 5 feet or less, and locally approaching 10 feet, will be required to establish finished grades for the proposed development, with deeper excavations below existing grades required in localized areas for construction of some utilities. The provided grading plan indicates that the greatest mass grading cuts will be required in the western portion of the site.

Based on the results of the geophysical and geotechnical testing, the depth to bedrock is highly variable across the site. The ERI study indicated bedrock depths ranging from near the existing ground surface to depths on the order of 28 feet below existing grades while auger refusal depths in the borings ranged from approximately 1 to 26½ feet below existing grades. In addition, "floating" boulders may be present in some areas at variable depths, including near the existing ground surface. The overburden soils are generally expected to be feasible using standard excavation techniques. However, the underlying very dense PWR materials and bedrock are expected to require increased excavation effort, including rock excavation methods such as hoe-ramming, jacking, ripping, or, possibly, blasting. These materials, where encountered, will likely result in reduced excavation rates. We recommend that the project owner and developer include contingency funds for the cost of rock removal in the budget for this project, and consider the potential resulting delays while establishing the project schedule.

While not expected to be encountered on a widespread basis during construction, groundwater may be encountered in localized areas during construction, particularly in the lower-lying portions of the site, and where deeper cuts below existing grades are required. The potential for encountering groundwater during site grading activities will be greater during and after wet weather periods. Groundwater levels are expected to fluctuate with seasonal variations in precipitation and may be influenced by construction activities. In addition, perched water may be present in localized areas where more granular soils or existing fills overlie less permeable, fine-grained soils or rock. Earthwork contractors should provide adequate earth support systems and be prepared to dewater excavations as necessary. Conventional sump-and-pump techniques are generally expected to be adequate for groundwater control.

Prior to the placement of new mass grading fills, where required, the site should be stripped and grubbed to remove any existing trees, vegetation, topsoil, root mat, or other unsuitable materials. The measured thickness of the topsoil layer at the boring locations ranged from approximately 4 to 10 inches. The topsoil layer may be thicker in unexplored areas of the site. In addition, the topsoil may be mixed with the underlying soils to greater depths during stripping operations or if the site has historically been cultivated for agricultural use. The actual stripping thickness will be dependent upon topsoil development, soil moisture content, construction traffic disturbance, and contractor care.

Materials identified as possible existing fills were encountered in Borings B-01, B-02, and B-13 to depths of approximately 3 to 5 feet below existing surface grades. In addition, a grid pattern of numerous small depressions, suggestive of a possible prior test pit exploration program, was observed at the site and in available historic aerial imagery. No documentation of a prior exploration program, or other prior earthwork or fill placement, was available at the time this report was prepared. The borings do not suggest widespread existing fills over significant portions of the site, but the potential exists that undocumented existing fills may also be encountered in localized areas of the site. Existing undocumented fills should be explored and verified, or completely removed to native subgrades and replaced with controlled, compacted fill. Removal and replacement of undocumented existing fills should be performed during mass grading of the site, and should be observed and documented by the project geotechnical engineer.

Proposed fill supporting structures or infrastructure should consist of approved materials placed in a controlled manner. The fill placement should be observed and documented by the geotechnical engineer or their qualified representative. Fill materials obtained from on- or off-site sources should meet the requirements specified in the table below. When practical, requests to use soils that do not precisely meet these requirements may be evaluated by the project geotechnical engineer.

Table 3: Fill Material Requirements

Fill Material Use	Recommended USCS Material Classifications	Index Property Limitations
Under Structures, Foundations, and Paved Sections, or as Backfill	GW, GP, GC, GM, SW, SP, SC, SM, CL, & ML	Less than 75% passing the No. 200 sieve, LL < 50, & PI < 35
Below-Grade Wall Backfill	GW, GP, GM, SW, SP, SM	Internal Friction Angle ≥30°
General Site Grading	GW, GP, GC, GM, SW, SP, SC, SM, CL, ML, CH, & MH	None

The predominant onsite clayey soils are likely to be moisture- and disturbance-sensitive and may be excessively wet of their optimum moisture content to facilitate proper compaction. Significant moisture conditioning effort may be required if these materials are to be re-used as structural fill. Mass grading during the summer months is recommended to reduce the potential costs and delays associated with moisture conditioning of fine-grained soils. High-plasticity, fine-grained soils (CH and MH) are generally not considered suitable for use as fill in structural areas, including as undercut backfill, due to the difficulty in achieving proper compaction. If off-site fill is required, the contractor should identify borrow material and submit representative samples for engineering testing and review. Testing should consist of soils classification (ASTM D2487) and a Standard Proctor density test (ASTM D698) for each type of borrow soil.

Fills placed on slopes steeper than 5H:1V should be benched into the existing slope at maximum vertical intervals of 5 feet to limit the potential for instability of the constructed soil slope. As previously noted, slopes steeper than 3H:1V are generally not recommended, and may require engineering design, possibly including special material requirements and internal reinforcement or other stabilization measures.

Soil fill should be placed in maximum 8-inch thick loose lifts and compacted with a sheepsfoot-type roller. Soil fill below foundations and pavement areas should be compacted to minimum 95 percent of the maximum dry density based on the Standard Proctor compaction at a moisture content within ±2 percentage points of optimum. Fill placement should be observed and documented by the project geotechnical engineer or their qualified representative, and density testing should be performed on each lift to verify compactive effort.

6.5 Foundations

Foundations should be supported on firm, native soils or documented, controlled, compacted fill. As discussed above, materials identified as possible existing fills were encountered on a limited basis, and documentation of these possible existing fills was not available at the time of this study. While not expected to impact construction of the proposed development on a widespread basis, the potential exists that additional undocumented existing fills may be discovered in other, unexplored areas of the site during construction. Any undocumented existing fills encountered during construction in areas of proposed structural support should be evaluated and addressed during mass grading of the site as recommended by the project geotechnical engineer or their qualified representative.

Provided these recommendations are observed and that any additional required fills within proposed structural support areas are placed in a controlled manner, the proposed lightly loaded residential structures can be supported on shallow spread footings proportioned for a net allowable bearing pressure of 2,000 pounds per square foot (psf). Maximum estimated settlements of less than 1-inch total and ½ inch differential for the proposed structures are expected based on the recommended bearing pressure. These settlement tolerances are within the generally accepted range for residential structures. The recommended allowable bearing pressure and anticipated settlements assume that the foundation subgrade will be evaluated during construction, and that any soft/loose or unstable areas addressed prior to footing construction as recommended by the project geotechnical engineer or their qualified representative. Foundations should be designed for minimum widths of 18 inches and 24 inches for continuous wall and individual column footings, respectively. Although these dimensions may not fully utilize the recommended bearing pressure, they should be maintained to reduce the potential for local shear type bearing failures.

Based on the results of the field and laboratory testing, and our understanding of the local geology, it is our opinion that the onsite soils have a high potential for volume change (i.e. shrink/swell). Therefore, we recommend that exterior footings should be founded minimum 36 inches below exterior grades to provide protection from frost action and reduce the potential for shrink/swell of the foundation subgrade soils.

The depth to bedrock is widely variable, and the potential exists that bedrock may be encountered at the footing subgrade in some areas, particularly where the proposed building finished floor elevations are planned to be near existing grades or in cut areas. Where bedrock is encountered, the embedment depth for exterior footings can be reduced to 24 inches below final exterior grade. However, we recommend that foundations bearing directly on rock should incorporate a minimum 4-inch thick compacted soil "cushion" to limit the potential for differential settlement between the areas bearing on rock and those bearing on soils. The soil cushion should be compacted to minimum 95 percent of the Standard Proctor compaction at a moisture content within ±2 percentage points of optimum, and the use of compacted stone (VDOT 21A/B) may be required to prepare a level working surface for placement of the soil cushion if the underlying bedrock surface is irregular. Rock condition and suitability for foundation support should be evaluated by a professional geotechnical engineer.

Foundation excavations should be reviewed by the project geotechnical engineer or their qualified representative prior to concrete placement. Penetration testing should be performed on the exposed foundation subgrade to confirm the design allowable bearing capacity. Foundation concrete should be placed on the day the foundations are excavated to limit the potential for shrink/swell of the subgrade soils due to moisture or temperature changes, and the foundation subgrade soils should be protected from precipitation and frost prior to concrete placement.

6.6 Floor Slabs

Floor slabs for ground-level spaces within the proposed townhomes can be designed as concrete slabs-on-grade. DAA recommends that the concrete floor slabs should be minimum 4 inches thick, reinforced with welded wire mesh, and supported on a minimum 4-inch thick coarse granular layer. The granular base course material should consist of open-graded, imported washed gravel or crushed stone with less than 5 percent by weight passing the No. 200 sieve (i.e. VDOT No. 57 stone), and should be covered with a minimum 10-mil polyethylene vapor barrier to interrupt the rise if capillary moisture through the slab and joints. Natural and compacted fill subgrades for support of the floor slabs should be reviewed and evaluated by the project geotechnical engineer or their qualified representative prior to the placement of concrete. Any soft/loose or unstable layers should be removed from the slab subgrade and replaced as recommended by the geotechnical engineer or their qualified representative.

Floor slabs may bear on wall footing projections, but they should be jointed so that slight movements of the foundation walls will not adversely affect the floor slabs. Control joints should be provided to control shrinkage cracking of the concrete floor system. If floor slabs are to be placed upon uncompacted fills, the slabs should be reinforced to span the unsupported lengths. Structural slab and grade beam systems, if required, should be designed by a professional structural engineer.

6.7 Below-Grade Walls

Though not specifically shown on the available plans, below-grade walls or site retaining walls may be required in localized areas to facilitate grading. Such walls, if planned, will be subjected to unbalanced earth pressures, and must be designed to resist these pressures.

For below grade wall structural design, the lateral earth pressure is dependent upon the condition of wall restraint, construction and compaction methods, and the shear strength of the soil being retained. The two most common conditions of restraint used in retaining wall design are the active and at-rest conditions. The active condition generally applies to free-standing structures and walls where some movement and/or "relaxation" may occur in order to mobilize the shear strength of the soil. The at-rest condition applies to rigid walls, such as basement walls, where there is no movement to mobilize the shear strength of the soil. The passive state, which is typically ignored for below-grade wall design as it can be negatively influenced by freeze/thaw or potentially removed altogether, represents the maximum lateral earth pressure influencing a wall that is being pushed into a soil mass.

The following parameters are recommended for evaluating lateral earth pressures on below grade walls with non-sloping backfill:

Table 4: Lateral Earth Pressure Parameters

Backfill Type	Soil Unit Weight	Approx. Internal Friction Angle	Earth Pressure Coefficients	
			At Rest (Ko)	Active (Ka)
Imported Sandy Soil (SM or more granular)	120 pcf	30°	0.50	0.33
Imported Granular Fill (i.e. VDOT 21 A/B)	130 pcf	37°	0.40	0.25

The recommended values presented in the table above assume that the below-grade wall systems incorporate an adequate drainage system to prevent the accumulation of hydrostatic pressure behind the walls. At a minimum, the drains should utilize a 4-inch perforated pipe. The pipe should be surrounded by minimum 6 inches of VDOT No. 57 stone. The aggregate should be wrapped in a non-woven drainage geotextile.

A coefficient of sliding friction of 0.35 may be used for design for mass concrete on approved soil subgrade. A coefficient of sliding friction of 0.55 may be used for design for mass concrete on approved crushed stone such as VDOT 21. Additional pressures due to surcharge loads should be applied based on anticipated temporary construction or permanent loadings near the top of the wall. To prevent lateral earth pressures in significant excess of those listed above, we recommend that heavy equipment not operate within a distance behind (above) below-grade wall equal to their height.

We recommend that below-grade walls, as well as any site retaining walls planned for the project, should be backfilled with granular soils meeting USCS GW, GP, GM, SW, SP, or SM classification, with a minimum internal friction angle of 30 degrees. Materials classified as ML may be allowed with prior review and approval from the project geotechnical engineer, depending upon the specific soil properties. The soils encountered in the explorations consisted predominantly of high-plasticity clays which do not meet the recommended backfill materials requirements. Clayey soils are not recommended for below-grade wall or retaining wall backfill due to their poor drainage characteristics, lower shear strength, and shrink-swell potential which could impose excessive lateral loads on the walls. Therefore, it is likely that imported borrow from off-site sources will be required for backfill of such wall, if planned. The Client should consider this cost while establishing the project budget. During construction, the contractor should submit samples from the proposed borrow source to DAA for review prior to importing the material to the site.

Any site retaining wall planned for the proposed development should be designed by a professional engineer registered in the Commonwealth of Virginia. The civil engineer should consider the construction method(s) for any proposed site walls and should incorporate adequate space behind the wall for construction. DAA recommends that clear space of minimum 1.5 to 2 times the wall height incorporated behind any site walls to allow for construction. The construction sequence should also be considered where excavations for proposed retaining walls might undermine nearby building foundations, utilities, or other improvements. Construction of site retaining walls should be observed and documented by the project geotechnical engineer or their qualified representative, and compactive effort for retaining wall backfill should be verified by in-placed density testing.

6.8 Subsurface Utilities

The available plans indicate that public water and sewer utilities, as well as storm drain utilities, are planned to serve the proposed development. The utility profiles included on the referenced plans indicate that excavations generally on the order of 5 to 10 feet below existing grades, and locally approaching 15 feet below existing grades, are expected to be required for construction of the proposed utilities.

Utility excavations within the overburden soils are generally expected to be feasible using conventional techniques. However, as noted previously, the depth to bedrock is highly variable across the site, and “floating” boulders may be present in some areas near the existing ground surface. These floating boulders, as well as the very dense PWR materials and underlying bedrock encountered in the borings, are expected to require rock excavation techniques, including hoe-ramming, jacking, ripping, or, possibly, blasting. Reduced utility excavation rates can be expected where these materials are encountered. The project owner and developer are advised to consider the potential cost and schedule impacts associated with excavation through these materials.

While not expected to have a significant impact on the proposed development, groundwater may be encountered in localized areas of the site, particularly within the deeper utility excavations, and in the lower-lying portions of the site. It should also be noted that groundwater levels are expected to fluctuate with seasonal variations in precipitation and may be influenced by development activity. In addition, perched water may be encountered within utility excavations where trapped within granular soils or existing fills which overlie less permeable, fine-grained soils or rock. Utility contractors should be prepared to dewater excavations as necessary, particularly during or after wet weather, and should provide adequate trench support in accordance with the latest OSHA standards.

Any soft/loose soils encountered at the utility subgrade elevation should be over-excavated to a stable stratum and replaced with controlled, compacted fill or stone (VDOT 21 A/B) or lean concrete (“flowable fill”). If saturated conditions are encountered at the utility subgrade elevation, a 6-inch thick stone bedding layer is recommended to provide more uniform support for the utility pipe.

Relatively soft conditions, possibly indicative of sinkhole throats or other karst features, were observed at depths of approximately 13.5 to 15 feet below existing grades in Borings B-03 and around 23.5 to 25 feet in Boring B-08, and other possible karst anomalies were noted in the ERI data, including below electrode 3-6, located along the proposed sanitary sewer between manholes 9 and 10. The potential exists that other karst anomalies may be present in other, unexplored areas of the site. Any such features encountered during construction should be carefully evaluated and addressed as recommended by the project geotechnical engineer.

Utility pipe systems below pavements and other structural areas should be backfilled with controlled, compacted fill. The backfill should be placed in maximum 6-inch thick loose lifts and compacted with a sheepsfoot type roller to a minimum of 95 percent of the maximum dry density based on the Standard Proctor (ASTM D698) compaction within +/-2 percentage points of optimum. The use of high plasticity soils for utility trench backfill beneath areas of structural support should be avoided to the extent feasible. Lift thicknesses should be reduced to 4 inches when compacting with lightweight, walk-behind equipment. Special care is required when backfilling with fine-grained materials as improper practices will result in excessive trench settlement or collapse. If these materials are used as utility trench backfill, they must be very carefully controlled and compacted. The backfill placement should be observed and documented by the project geotechnical engineer or their qualified representative, and density testing should be performed to evaluate compactive effort.

The risk of utility trench settlement or failure due to improperly placed or under-compacted backfill, particularly within roadways or drive-lanes, is especially high when fine-grained, clayey soils (such as those predominantly encountered in the borings) are used as backfill. Furthermore, it is our experience that utility contractors are sometimes resistant to “buy-in” to the level of effort required for proper placement/compaction of fine-grained, clayey soils as utility trench backfill. Ideally, this risk can be significantly reduced by using only granular materials for utility trench backfill. However, DAA acknowledges that, due to the local geology, this may be impractical and/or cost-prohibitive. If, for economic or practical reasons, the onsite clayey soils are planned to be re-used as utility trench backfill, DAA recommends that the Client should incorporate contract language making the utility contractor responsible for the performance of the utility trench backfill.

6.9 Pavements

Based on the proposed grading scheme, a combination of cuts and fills on the order of 5 feet or less are expected to be required to establish the proposed roadway finished grades. Therefore, the proposed pavement subgrade will likely consist of a combination of the native variable-plasticity, fine-grained soils, or borrow soils placed as controlled, compacted fill.

DAA recommends that the pavement section for the proposed roadways should be designed in accordance with the latest edition of the Virginia Department of Transportation (VDOT) Pavement Design Guide for Subdivision and Secondary Roads in Virginia (VDOT Pavement Design Guide), which recommends that the following values should be used for preliminary pavement design, based on the site location:

Table 5: Recommended Preliminary Pavement Design Values (per VDOT Pavement Design Guide)

Pavement Design Parameter	Recommended Value for Preliminary Design
Resiliency Factor, RF	2
California Bearing Ratio, CBR	5
Soil Support Value, SSV	10

It should be noted that, while the VDOT Pavement Design Guide recommends that a CBR value of 5 be assumed for pavement design, it is our opinion that a CBR value on the order of 3 to 4 is likely more realistic, based on the results of our field exploration and our experience with other projects in similar geologic conditions.

DAA should be retained during construction of the project to provide construction observation and testing services, review the subgrade condition, and collect samples for testing, including CBR testing, to verify that assumed soil parameters for the pavement subgrade are met. It should be noted that the final pavement design may require revision based on the results of the construction-phase CBR testing of actual pavement subgrade samples.

Prior to construction of the proposed roadway pavement section, the pavement subgrade should be reviewed with a proof-roll to verify stability. The proof-roll should be performed with a loaded, tandem-axle dump-truck in the presence of the project geotechnical engineer or their qualified representative. Any unsuitable soils at the pavement subgrade identified during the proof-roll should be over-excavated to a stable stratum and replaced with controlled, compacted, granular fill. Construction traffic must be controlled to limit disturbance of previously approved subgrade, stone base course, or partially constructed asphalt pavement.

The pavement subgrade for the proposed residential driveways at the individual lots should be prepared in the same manner as described above. Once an approved subgrade has been achieved, the driveway pavement section should be constructed. DAA recommends a minimum driveway pavement section as follows:

Table 6: Recommended Minimum Residential Driveway Pavement Section

Pavement Course	Thickness & Material Notation
Surface Asphalt Layer	1.5 inches VDOT SM-9.5A (or approved equivalent)
Aggregate Base Layer	8.0 inches VDOT 21A/B (or approved equivalent)

It should be noted that this pavement section is intended for light vehicle traffic only and is not intended for heavy construction traffic. During construction of the proposed single-family homes, the driveway pavement should be protected from heavy construction traffic which may cause damage or premature failure.

6.10 Stormwater Management

The available plans indicate that underground retention facilities are planned in the eastern and southern portions of the site for stormwater management (SWM). Specific details of these SWM facilities were not available at the time this report was prepared. However, based on a review of the existing and proposed site grades and utility profiles shown on the available plans, we have assumed that excavations on the order of 5 to 10 feet below existing grades may be required for construction of these facilities.

Near-surface excavations within the existing overburden soils are expected to be feasible by conventional techniques. However, Boring B-13 and B-13A, performed in the vicinity of the eastern SWM facility, encountered auger refusal at depths of approximately 11 and 16 feet, respectively, below existing grades. As noted earlier, bedrock depth is variable across the site, and the potential exists that excavations for these SWM facilities may encounter PWR or bedrock above the proposed invert elevations. These materials, if encountered, are expected require the use of rock excavation techniques (i.e. hoe-ramming, jacking, ripping, or possibly blasting).

Groundwater is generally not expected to impact construction of the proposed SWM facilities. However, as noted previously, groundwater levels may fluctuate with seasonal variations in precipitation or as a result of construction activity, and perched water may be encountered locally where underlain by low-permeability soils or rock. Contractors should be prepared to provide earth support and dewatering systems as necessary for the proposed SWM excavations and should establish grades during construction to drain surface runoff effectively and maintain trafficability of the SWM areas.

It should be noted that the ERI study identified a potential karst anomaly below electrode 3-17, located in the vicinity of the eastern proposed SWM facility. While Borings B-13 and B-13A, performed near the suspected anomaly location, did not encounter conditions obviously indicative of a karst feature, the potential presence of a sinkhole throat or other karst feature at this location cannot be ruled out. DAA recommends that this area be carefully evaluated by the project geotechnical engineer during construction to identify and address potential karst risks in the proposed SWM area.

Due to the local geology and likelihood for karst conditions at the site, the potential exists that the repeated collection and introduction of water into the subsurface may, over time, promote the development of sinkholes or other problematic karst features. As such, DAA recommends that the use of infiltration techniques at proposed SWM facilities should be avoided, and that these facilities should be lined to limit the introduction of water into the subsurface at these areas.

7.0 LIMITATIONS

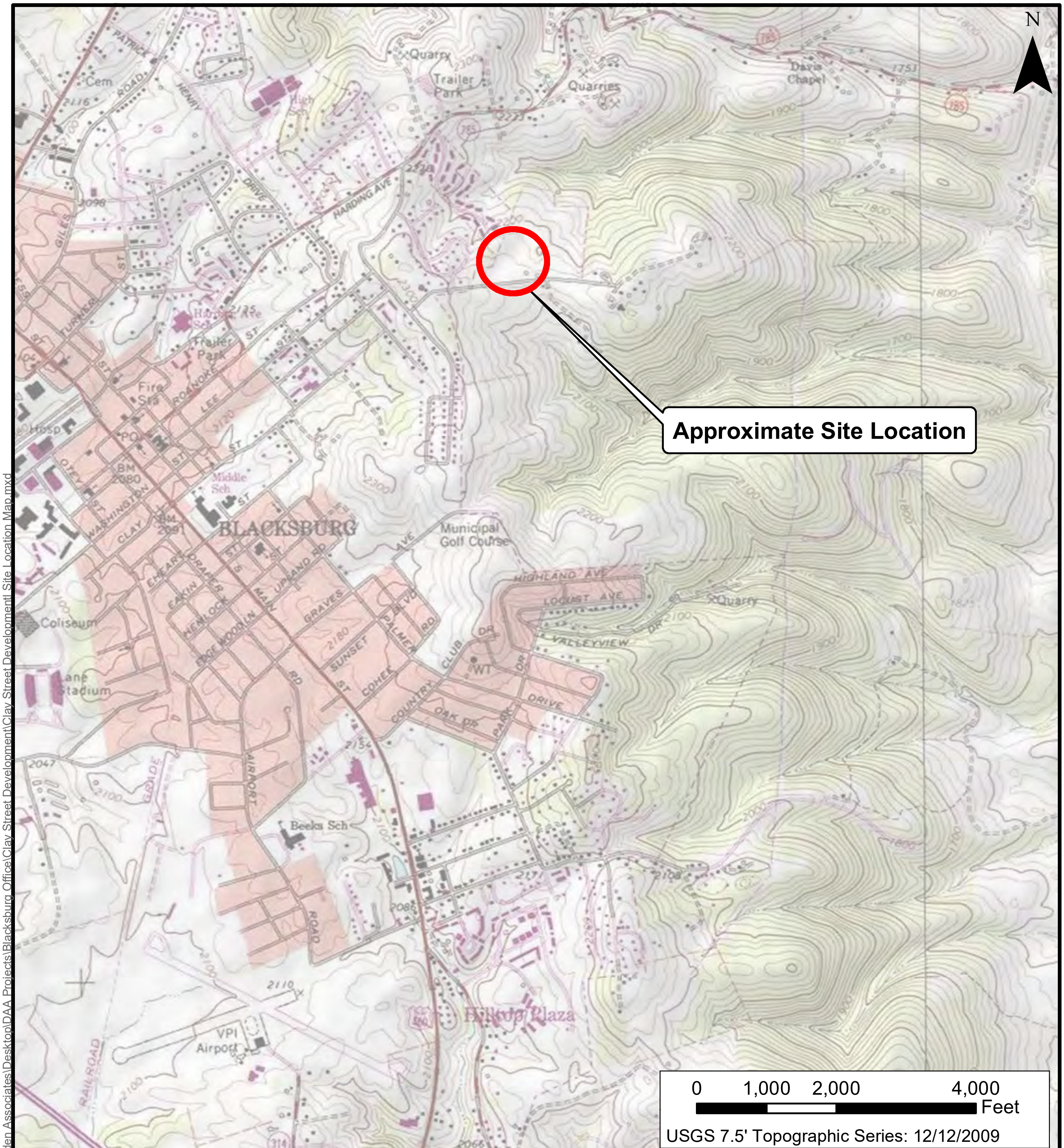
This report has been prepared for the exclusive use of Blacksburg, LLC and its designated representatives for the Apogee Townhomes (Clay Street Development) residential development project. Our conclusions and recommendations have been rendered in a manner consistent with the level and skill ordinarily exercised by members of the geotechnical engineering profession in the Commonwealth of Virginia at the time of our study. We make no other warranty, express or implied.

Our conclusions and recommendations are based on design information furnished to us and our experience. They do not necessarily reflect variations in the subsurface conditions, which have potential to exist intermediate of our borings and in unexplored areas of the site due to inherent variability of the subsurface conditions in this geologic region, as well as past land use. Should such variations become apparent during construction, it will be necessary for us to re-evaluate our conclusions and recommendations based upon on-site observations of the conditions.

If changes are made location, layout, or nature of the proposed improvements, then the recommendations presented in this report must not be considered valid unless the changes are reviewed by Draper Aden Associates, and our recommendations are modified or verified in writing. We request the opportunity to review the foundation plan, grading plan and applicable portions of the project specifications when the design is finalized. This review will allow us to check whether these documents are consistent with the intent of our recommendations. Draper Aden Associates is not responsible for the conclusions, opinions or recommendations of others based on the data in this report.

APPENDIX
Section 1

Site Vicinity Map
Exploration Location Plan



Site Location Map

Apogee Townhomes
Clay Street Deveopment
Blacksburg, VA 24060

SCALE: 1" = 2000'

DAA PN: 18010224-020203

Draper Aden Associates
Engineering ◆ Surveying ◆ Environmental Services

2206 South Main Street
Blacksburg, VA 24060
540-552-0444 Fax: 540-552-0291

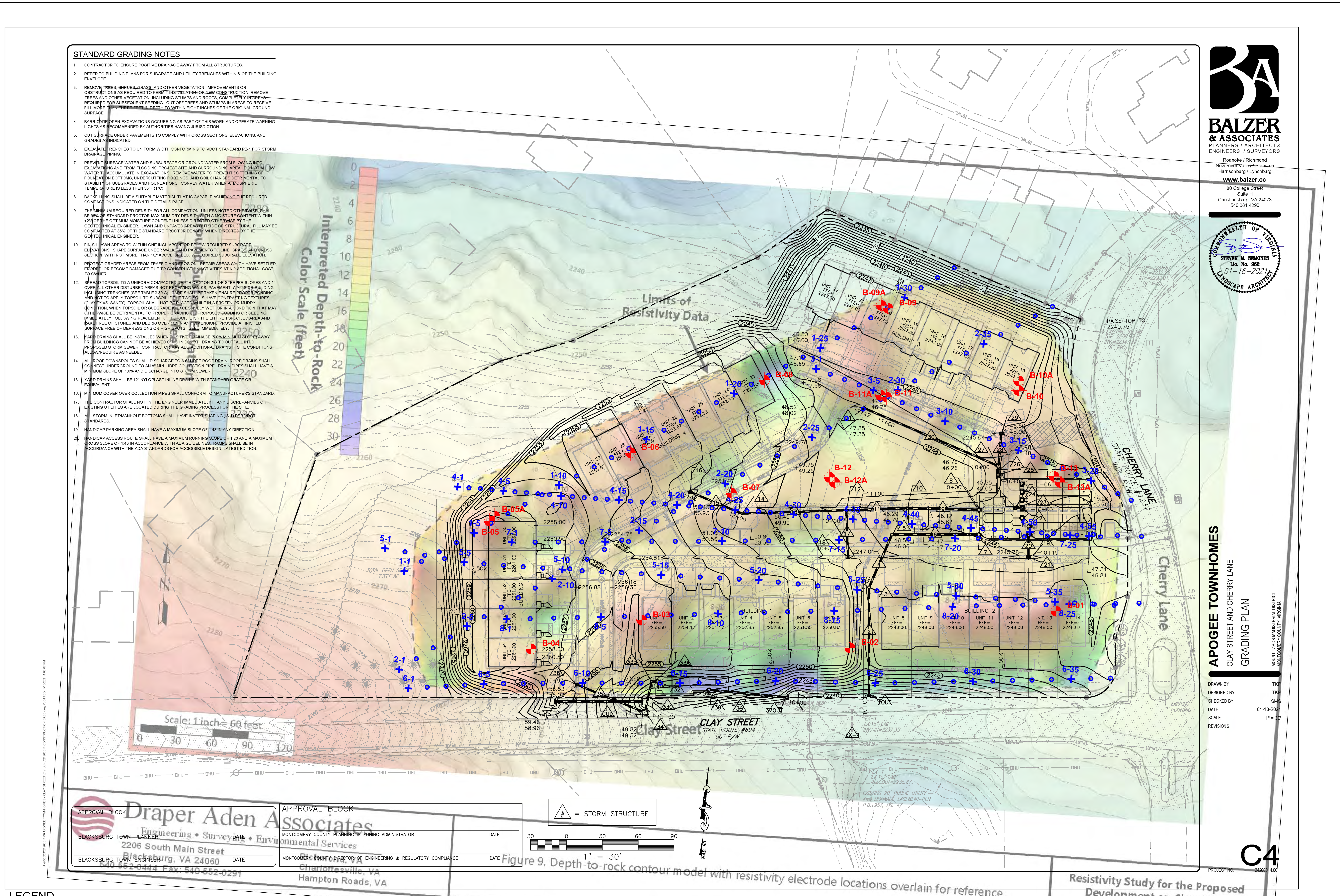
Richmond, VA
Charlottesville, VA
Hampton Roads, VA

Raleigh, NC
Fayetteville, NC
Northern Virginia
Virginia Beach, VA

DESIGNED	N/A
DRAWN	SAM
CHECKED	FDP
DATE	4/06/2021

FIGURE
1

Path: C:\Users\smagnuson\OneDrive - Draper Aden Associates\Desktop\DAA Projects\Blacksburg Office\Clay Street Development\Clay Street Development\ Site Location Map.mxd



LEGEND

B-01 IDENTIFICATION AND APPROXIMATE LOCATION OF STANDARD PENETRATION TEST (SPT) BORINGS PERFORMED BY DRAPER ADEN ASSOCIATES (DAA) IN MARCH OF 2021.

4-1 … 4-5 IDENTIFICATION AND APPROXIMATE LOCATION OF RESISTIVITY ELECTRODES FOR ELECTRICAL RESISTIVITY IMAGING (ERI) STUDY PERFORMED BY DAA IN SEPTEMBER OF 2018.

NOTES

1. BASE MAP ADAPTED FROM THE GRADING PLAN (SHEET C4) FOR APOGEE TOWNHOMES AT CLAY STREET AND CHERRY LANE, DATED JANUARY 18, 2021, PREPARED BY BALZER & ASSOCIATES (BALZER), AND FIGURE 9 FROM THE RESISTIVITY STUDY FOR THE PROPOSED DEVELOPMENT ON CLAY STREET, DATED SEPTEMBER 20, 2018, PREPARED BY DRAPER ADEN ASSOCIATES (DAA).

2. THE EXPLORATION LOCATIONS WERE SELECTED BY DAA, IN CONSULTATION WITH THE CLIENT, BASED ON THE ABOVE-REFERENCED PLANS AND PRIOR RESISTIVITY STUDY, AND WERE FIELD-LOCATED USING A HAND-HELD GLOBAL POSITIONING SYSTEM (GPS) UNIT. BORING B-04 WAS OFFSET FROM ITS ORIGINALLY MARKED LOCATION DUE TO ACCESS CONSTRAINTS WHILE BORINGS B-05A, B-09A, B-10A, B-11A, B-12A, AND B-13A WERE ADDED TO THE EXPLORATION PROGRAM AT OFFSET LOCATIONS DUE TO SHALLOW AUGER REFUSALS AT THE ORIGINALLY PLANNED LOCATIONS. THE LOCATIONS OF THE OFFSET BORINGS SHOWN ON THIS PLAN ARE BASED ON TAPE MEASUREMENTS FROM THE ORIGINALLY PLANNED BORING LOCATIONS AND ARE APPROXIMATE.

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EXPLORATION LOCATION PLAN
APOGEE TOWNHOMES (CLAY STREET DEVELOPMENT)
MONTGOMERY COUNTY, VIRGINIA

REVISIONS

DESIGNED BY:	N/A
DRAWN BY:	FDP
CHECKED BY:	JTH
SCALE:	1" = 30'
DATE:	04/22/2021
PROJECT NUMBER:	18010224-020203

FIGURE 1

APPENDIX
Section 2

Geotechnical Exploration Summary Table
Key to Boring Logs
SPT Boring Logs

Geotechnical Exploration Summary Table
Apogee Townhomes (Clay Street Development)
Montgomery County, Virginia
DAA Project Number: 18010224-020203

Draper Aden Associates
Engineering • Surveying • Environmental Services

Exploration Identification	Approximate Existing Surface Elevation[1]	Exploration Depth[2]	Approximate Exploration Termation Elevation	Approximate Proposed Grade[3]	Approximate Proposed Cut/Fill (-/+)[3]	Surface Layer		Possible Existing Fill[4]		Partially Weathered Rock[5]		Auger/Spoon Refusal[2]		Subsurface Water/Cave-In[6]									
						Material	Approximate Thickness	Approximate Depth to Bottom of Existing Fill	Approximate Bottom Elevation of Exising Fill	Approximate Depth to Partially Weathered Rock	Approximate Elevation of Partially Weathered Rock	Approximate Depth to Auger/Spoon Refusal	Approximate Elevation of Auger/Spoon Refusal	Subsurface Water Encountered During Drilling		Water Level Upon Completion of Drilling/Excavation		Cave Depth Upon Completion of Drilling/Excavation		Water Level One or More Days After Completion of Drilling/Excavation		Cave Depth One or More Days After Completion of Drilling/Excavation	
														Approximate Depth	Approximate Elevation	Approximate Depth	Approximate Elevation	Approximate Depth	Approximate Elevation	Approximate Depth	Approximate Elevation	Approximate Depth	Approximate Elevation
	(MSL)	(ft)	(MSL)	(MSL)	(ft)		(in)	(ft)	(MSL)	(ft)	(MSL)	(ft)	(MSL)	(ft)	(MSL)	(ft)	(MSL)	(ft)	(MSL)	(ft)	(MSL)	(ft)	(MSL)
B-01	2248	15	2233	2248	0	Topsoil	4	5	2243	NE	NE	NE	NE	NE	NE	NE	NE	2.0	2246	NA	NA	NA	NA
B-02	2243	9	2234	2250	7	Topsoil	6	5	2238	NE	NE	9	2234	NE	NE	NE	NE	8.3	2235	NE	NE	6.8	2236
B-03	2255	17	2238	2255	0	Topsoil	7	NE	NE	NE	NE	17	2238	NE	NE	NE	NE	16.2	2239	NE	NE	15.1	2240
B-04	2261	4.5	2257	2261	0	Topsoil	4	NE	NE	NE	NE	4.5	2257	NE	NE	NE	NE	3.8	2257	NE	NE	3.8	2257
B-05	2262	1	2261	2261	-1	Topsoil	NA	NE	NE	NA	NA	1	2261	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
B-05A	2262	2	2260	2261	-1	Topsoil	4	NE	NE	NE	NE	2	2260	NE	NE	NE	NE	1.3	2261	NE	NE	1.3	2261
B-06	2257	11	2246	2255	-2	Topsoil	6	NE	NE	NE	NE	11	2246	NE	NE	NE	NE	10.3	2247	NE	NE	9.5	2248
B-07	2255	16.5	2239	2251	-4	Topsoil	6	NE	NE	NE	NE	16.5	2239	NE	NE	NE	NE	14.5	2241	NE	NE	14.5	2241
B-08	2249	26.5	2223	2251	2	Topsoil	10	NE	NE	NE	NE	26.5	2223	7	2243	NE	NE	23.9	2225	NE	NE	23.8	2225
B-09	2242	6.1	2236	2247	5	Topsoil	5	NE	NE	6	2236	6.1	2236	NE	NE	NE	NE	5.2	2237	NA	NA	NA	NA
B-09A	2242	13.8	2228	2247	5	Topsoil	NA	NE	NE	13.5	2229	13.8	2228	NE	NE	NE	NE	8.3	2234	NA	NA	NA	NA
B-10	2243	3	2240	2247	4	Topsoil	5	NE	NE	NE	NE	3	2240	NE	NE	NE	NE	2.0	2241	NA	NA	NA	NA
B-10A	2243	6.3	2237	2247	4	Topsoil	NA	NE	NE	NE	NE	6.3	2237	NE	NE	NE	NE	5.3	2238	NA	NA	NA	NA
B-11	2247	8.5	2239	2246	-1	Topsoil	5	NE	NE	7	2240	8.5	2239	NA	NA	NA	NA	6.8	2240	NA	NA	NA	NA
B-11A	2247	7.5	2240	2246	-1	Topsoil	NA	NE	NE	NA	NA	7.5	2240	NE	NE	NE	NE	5.7	2241	NA	NA	NA	NA
B-12	2251	6.1	2245	2249	-2	Topsoil	8	NE	NE	6	2245	6.1	2245	NE	NE	NE	NE	5.5	2246	NA	NA	NA	NA
B-12A	2251	7.5	2244	2249	-2	Topsoil	NA	NE	NE	NE	NE	7.5	2244	NE	NE	NE	NE	5.8	2245	NE	NE	5.9	2245
B-13	2245	11	2234	2245	0	Topsoil	8	3	2242	NE	NE	11	2234	NE	NE	NE	NE	9.0	2236	NA	NA	NA	NA
B-13A	2245	16	2229	2245	0	Topsoil	NA	NE	NE	NE	NE	16	2229	NE	NE	NE	NE	7.8	2237	NA	NA	NA	NA

Notes:
1. Approximate existing surface elevations at the exploration locations were estimated from the existing site topography shown on the Grading Plan (Sheet C4) for Apogee Townhomes at Clay Street and Cherry Lane, dated January 18, 2021, prepared by Balzer & Associates (Balzer), the project civil engineer.
2. The borings (except Boring B-01) encountered auger refusal above their planned termination depths.
3. Approximate proposed grades at the exploration locations represent the proposed finished surface grades (or proposed finished floor elevations, where applicable) and are based on the proposed grading scheme shown on the above-referenced Grading Plan (Sheet C4) prepared by Balzer.
4. Borings B-01, B-02, and B-13 encountered materials identified as possible existing fills based on visual review of the samples recovered from the borings.
5. Partially Weathered Rock (PWR) is an intermediate geo-material characterized by SPT results of greater than 50 blows per 6 inches of split-spoon penetration.
6. Borings performed on the final day of drilling were backfilled upon completion for safety reasons, and subsequent groundwater level and cave-in depth observations are not available.

MSL = Mean Sea Level
NA = Not Available or Not Applicable
NE = Not Encountered

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UNIFIED SOIL CLASSIFICATION & SYMBOL CHART

COARSE-GRAINED SOILS (More than 50% of material is larger than No. 200 sieve)		
		Clean Gravels (Less than 5% fines)
GRAVELS More than 50% of coarse fraction retained on No. 4 seive	**GW**	Well-graded gravels, gravel-sand mixtures, little or no fines
	GP	Poorly-graded gravels, gravel-sand mixtures, little or no fines
		Gravels with more than 12% fines
	GM	Silty gravels, gravel-sand mixtures
	GC	Clayey gravels, gravel-sand-clay mixtures
		Clean Sands (Less than 5% fines)
SANDS More than 50% of coarse fraction passing No. 4 seive	**SW**	Well-graded sands, gravelly sands, little or no fines
	SP	Poorly-graded sands, gravelly sands, little or no fines
		Sands with more than 12% fines
	SM	Silty sands, sand-silt mixtures
	SC	Clayey sands, sand-clay mixtures
FINE-GRAINED SOILS (More than 50% of material is smaller than No. 200 sieve)		
SILTS & CLAYS Liquid limit less than 50%	**ML**	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
	CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
	OL	Organic silts and organic silty clays of low plasticity
SILTS & CLAYS Liquid limit 50% or greater	**MH**	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
	CH	Inorganic clays of high plasticity, fat clays
	OH	Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS	**PT**	Organic clays of medium to high plasticity, organic silts

SOIL DENSITY & CONSISTENCY DESCRIPTIONS

Coarse-Grained Soil		Fine-Grained Soil	
N-Value	Density	N-Value	Consistency
0 - 4	Very Loose	0 - 1	Very Soft
5 - 10	Loose	2 - 4	Soft
11 - 30	Medium Dense	5 - 8	Medium Stiff
31 - 50	Dense	9 - 15	Stiff
> 50	Very Dense	16 - 30	Very Stiff
		> 30	Hard

LOG SYMBOLS

- ▽ Subsurface water first encountered
- ▼ Subsurface water level upon completion
- Cave-in

LABORATORY CLASSIFICATION CRITERIA

GW: $C_u = D_{60}/D_{10}$, > or equal to 4 & $C_C = D_{30}/(D_{10}*D_{60})$ between 1 & 3

GP: Not meeting all gradation requirements for GW

GM: Atterburg limits below "A" line or PI less than 4	Limits plotting on or above "A" line with PI between 4 & 7 are borderline cases requiring dual symbols
GC: Atterburg limits above "A" line with PI greater than 7	

SW: $C_u = D_{60}/D_{10}$, > or equal to 6 & $C_C = D_{30}/(D_{10}*D_{60})$ between 1 & 3

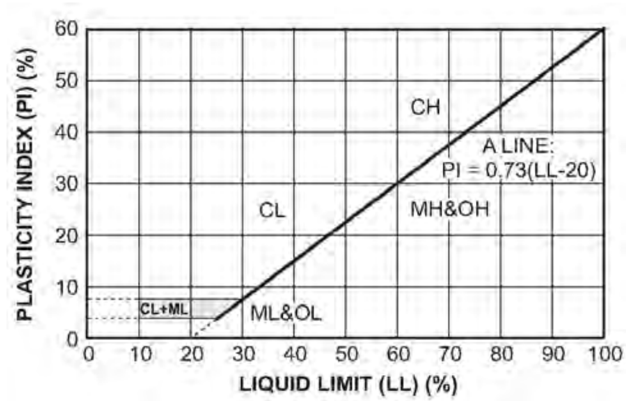
SP: Not meeting all gradation requirements for SW

SM: Atterburg limits below "A" line or PI less than 4	Limits plotting in shaded zone with PI between 4 & 7 are borderline cases requiring dual symbols
SC: Atterburg limits above "A" line with PI greater than 7	

Determine percentages of sand and gravel from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve), course-grained soils are classified as follows:

Less than 5%..GW, GP, SW, SP
More than 12%..GM, GC, SM, SC
5% to 12%..........................Borderline cases requiring dual symbols

PLASTICITY CHART



MOISTURE DESCRIPTIONS

DRY: No apparent moisture, dusty

DAMP: Apparent moisture, below Plastic Limit

MOIST: Significant moist, at or above the Plastic Limit, can be rolled into a 1/8" thread

WET: Appears saturated, free water in voids and pores

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Client: **Blacksburg, LLC**	Project No.: **18010224-020203**
Project: **Apogee Townhomes (Clay Street Development)**	Driller: **Blue Ridge Drilling**
Location: **Montgomery County, Virginia**	Method: **2-1/4" HSA w/SPT w/ Autohammer**
Total Depth **15.0'** | Elev GS **2248.0'** | Logged by: **FDP**	Completion Date: **March 22, 2021**

Elev.	Depth	DESCRIPTION (USCS)	Blow Counts / Recovery (%)	N Value	PP (tsf)	% Fines	% H_2O	LL	PI	REMARKS
		Topsoil ~ 4 inches								
		Possible Fill sampled as: Clayey SAND with Gravel (SC), Brown, Moist, Medium dense to dense	4-8-12 100	20						At completion: Dry to cave-in at approx. 2 feet. (Augers reversed during removal.) Borehole backfilled upon completion for safety reasons.
2245										
			12-18-16 100	34						
	5	Sandy Lean CLAY (CL), Brown to orange-brown, Moist, Stiff								
			6-6-6 100	12						
2240		Fat CLAY (CH), Orange-brown, Moist, Stiff								
			3-4-6 61	10						
	10									
2235										
			2-4-5 100	9						
	15	Bottom of borehole at 15.0 feet. Target Depth								

GEOTECH SPT APOGEE TOWNHOMES (CLAY STREET DEVELOPMENT) - FDP.GPJ DF DAA.GDT 4/19/21

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Client: **Blacksburg, LLC**	Project No.: **18010224-020203**
Project: **Apogee Townhomes (Clay Street Development)**	Driller: **Blue Ridge Drilling**
Location: **Montgomery County, Virginia**	Method: **2-1/4" HSA w/SPT w/ Autohammer**
Total Depth **9.0'** · Elev GS **2243.0'** · Logged by: **FDP**	Completion Date: **March 22, 2021**

Elev.	Depth	DESCRIPTION (USCS)	Blow Counts / Recovery (%)	N Value	PP (tsf)	% Fines	% H_2O	LL	PI	REMARKS
		Topsoil ~ 6 inches								
		Possible Fill sampled as: Sandy Lean CLAY with Gravel (CL), Brown, Moist, Stiff	2-2-7 67	9						
2240										
			5-6-7 17	13						
	5	Sandy Fat CLAY (CH), Orange-brown, Moist, Very stiff	3-7-11 83	18						
2235										At completion: Dry to cave-in at approx. 8.3 feet. 3/23/2021: Dry to cave-in at approx. 6.8 feet.
			50 100							
		Refusal at 9.0 feet. Bottom of borehole at 9.0 feet. Auger Refusal								

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Client: **Blacksburg, LLC**		Project No.: **18010224-020203**	
Project: **Apogee Townhomes (Clay Street Development)**		Driller: **Blue Ridge Drilling**	
Location: **Montgomery County, Virginia**		Method: **2-1/4" HSA w/SPT w/ Autohammer**	
Total Depth **17.0'**	Elev GS **2255.0'**	Logged by: **FDP**	Completion Date: **March 22, 2021**

Elev.	Depth	DESCRIPTION (USCS)	Blow Counts / Recovery (%)	N Value	PP (tsf)	% Fines	% H_2O	LL	PI	REMARKS
		Topsoil ~ 7 inches								
		Sandy Fat CLAY (CH), trace Gravel, Orange-brown, Moist, Stiff	1-3-6 94	9						
			3-6-8 100	14						
2250	5									
			3-4-6 100	10						
		Sandy Fat CLAY (CH), trace Gravel, Orange-brown, Moist, Very stiff	3-7-10 100	17						
2245	10									
		Fat CLAY (CH), Orange-brown to brown, Moist, Soft								
			3-2-2 89	4						
2240	15									At completion: Dry to cave-in at approx. 16.2 feet. 3/23/2021: Dry to cave-in at approx. 15.1 feet.
		Refusal at 17.0 feet. Bottom of borehole at 17.0 feet. Auger Refusal								

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Client: **Blacksburg, LLC**			Project No.: **18010224-020203**
Project: **Apogee Townhomes (Clay Street Development)**			Driller: **Blue Ridge Drilling**
Location: **Montgomery County, Virginia**			Method: **2-1/4" HSA w/SPT w/ Autohammer**
Total Depth **4.5'**	Elev GS **2261.0'**	Logged by: **FDP**	Completion Date: **March 22, 2021**

Elev.	Depth	DESCRIPTION (USCS)	Blow Counts / Recovery (%)	N Value	PP (tsf)	% Fines	% H_2O	LL	PI	REMARKS
		Topsoil ~ 4 inches								
2260		Sandy Fat CLAY (CH), trace Gravel, Orange-brown, Moist, Stiff	2-4-7 78	11						
			2-4-50 100	50/0"			25.5			At completion: Dry to cave-in at approx. 3.8 feet. 3/23/2021: Dry to cave-in at approx. 3.8 feet.
		Refusal at 4.5 feet. Bottom of borehole at 4.5 feet. Auger Refusal								

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Client: **Blacksburg, LLC**			Project No.: **18010224-020203**
Project: **Apogee Townhomes (Clay Street Development)**			Driller: **Blue Ridge Drilling**
Location: **Montgomery County, Virginia**			Method: **2-1/4" HSA w/SPT w/ Autohammer**
Total Depth **1.0'**	Elev GS **2262.0'**	Logged by: **FDP**	Completion Date: **March 22, 2021**

Elev.	Depth			DESCRIPTION (USCS)	Blow Counts / Recovery (%)	N Value	PP (tsf)	% Fines	% H_2O	LL	PI	REMARKS
				Auger probe to refusal at approx. 1 foot. No SPT or sampling.								
				Refusal at 1.0 feet. Bottom of borehole at 1.0 feet. Auger Refusal								

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Client: **Blacksburg, LLC**			Project No.: **18010224-020203**
Project: **Apogee Townhomes (Clay Street Development)**			Driller: **Blue Ridge Drilling**
Location: **Montgomery County, Virginia**			Method: **2-1/4" HSA w/SPT w/ Autohammer**
Total Depth **2.0'**	Elev GS **2262.0'**	Logged by: **FDP**	Completion Date: **March 22, 2021**

Elev.	Depth			DESCRIPTION (USCS)	Blow Counts / Recovery (%)	N Value	PP (tsf)	% Fines	% H_2O	LL	PI	REMARKS
				Topsoil ~ 4 inches								
2260				Sandy Fat CLAY with Gravel (CH), Orange-brown, Moist to Dry	4-17							At completion: Dry to cave-in at approx. 1.3 feet. 3/23/2021: Dry to cave-in at approx. 1.3 feet. SPT terminated due to laterally deflecting spoon.
				Refusal at 2.0 feet. Bottom of borehole at 2.0 feet. Auger Refusal								

Client: **Blacksburg, LLC**	Project No.: **18010224-020203**
Project: **Apogee Townhomes (Clay Street Development)**	Driller: **Blue Ridge Drilling**
Location: **Montgomery County, Virginia**	Method: **2-1/4" HSA w/SPT w/ Autohammer**
Total Depth **11.0'** / Elev GS **2257.0'** / Logged by: **FDP**	Completion Date: **March 22, 2021**

Elev.	Depth	DESCRIPTION (USCS)	Blow Counts / Recovery (%)	N Value	PP (tsf)	% Fines	% H_2O	LL	PI	REMARKS
		Topsoil ~ 6 inches								
2255		Fat CLAY (CH), Orange-brown, Moist, Stiff	2-3-6 100	9						
	5	Fat CLAY with Gravel (CH), Orange-brown to brown, Moist, Stiff to very stiff	3-7-9 78	16						
2250			3-7-10 78	17						
	10		3-8-8 100	16						At completion: Dry to cave-in at approx. 10.3 feet. 3/23/2021: Dry to cave-in at approx. 9.5 feet.
		Refusal at 11.0 feet. Bottom of borehole at 11.0 feet. Auger Refusal								

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Client: **Blacksburg, LLC**	Project No.: **18010224-020203**
Project: **Apogee Townhomes (Clay Street Development)**	Driller: **Blue Ridge Drilling**
Location: **Montgomery County, Virginia**	Method: **2-1/4" HSA w/SPT w/ Autohammer**
Total Depth **16.5'** / Elev GS **2255.0'** / Logged by: **FDP**	Completion Date: **March 22, 2021**

Elev.	Depth	DESCRIPTION (USCS)	Blow Counts / Recovery (%)	N Value	PP (tsf)	% Fines	% H_2O	LL	PI	REMARKS
	0	Topsoil ~ 6 inches								
		Fat CLAY (CH), Orange-brown, Moist, Stiff	2-3-7 100	10						
		Fat CLAY with Gravel (CH), Orange-brown, Moist, Stiff	2-5-9 100	14						
2250	5									
		Fat CLAY (CH), Orange-brown, Moist, Stiff	3-4-7 89	11		99.2	37.1	115	82	
			2-5-7 100	12						
2245	10									
		Fat CLAY with Gravel (CH), Orange-brown, Moist, Medium stiff								
2240	15		2-2-3 89	5						At completion: Dry to cave-in at approx. 14.5 feet. 3/23/2021: Dry to cave-in at approx. 14.5 feet. Augers grinding below approx. 16 feet.
		Refusal at 16.5 feet. Bottom of borehole at 16.5 feet. Auger Refusal								

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Client:	**Blacksburg, LLC**	Project No.:	**18010224-020203**
Project:	**Apogee Townhomes (Clay Street Development)**	Driller:	**Blue Ridge Drilling**
Location:	**Montgomery County, Virginia**	Method:	**2-1/4" HSA w/SPT w/ Autohammer**
Total Depth **26.5'**	Elev GS **2249.0'** Logged by: **FDP**	Completion Date:	**March 22, 2021**

Elev.	Depth	DESCRIPTION (USCS)	Blow Counts / Recovery (%)	N Value	PP (tsf)	% Fines	% H_2O	LL	PI	REMARKS
		Topsoil ~ 10 inches								
		Fat CLAY (CH), Orange-brown, Moist, Medium stiff	2-3-5 78	8						
2245	5		2-4-4 100	8						
		Fat CLAY (CH), Orange-brown, Moist to wet, Medium stiff	2-3-3 100	6						Spoon wet at approx. 6.5 feet.
2240	10	Fat CLAY (CH), Orange-brown, Moist, Medium stiff	2-3-3 67	6						
2235	15	Fat CLAY with Gravel (CH), Orange-brown with Black Mottles, Moist, Medium stiff	2-3-5 89	8						
2230	20		3-3-4 67	7						Augers chattering below approx. 22 feet.
2225	25	Sandy Fat CLAY with Gravel (CH), Brown, Wet, Soft	3-2-2 44	4						At completion: Dry to cave-in at approx. 23.9 feet. 3/23/2021: Dry to cave-in at approx. 23.8 feet.

Refusal at 26.5 feet.
Bottom of borehole at 26.5 feet.
Auger Refusal

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Client: **Blacksburg, LLC**			Project No.: **18010224-020203**
Project: **Apogee Townhomes (Clay Street Development)**			Driller: **Blue Ridge Drilling**
Location: **Montgomery County, Virginia**			Method: **2-1/4" HSA w/SPT w/ Autohammer**
Total Depth **6.1'**	Elev GS **2242.0'**	Logged by: **FDP**	Completion Date: **March 23, 2021**

Elev.	Depth	DESCRIPTION (USCS)	Blow Counts / Recovery (%)	N Value	PP (tsf)	% Fines	% H_2O	LL	PI	REMARKS
		Topsoil ~ 8 inches								
2240		Fat CLAY (CH), Brown to orange-brown, Moist, Stiff	2-4-7 83	11						
	5		4-5-9 100	14						At completion: Dry to cave-in at approx. 5.2 feet. Borehole backfilled upon completion for safety reasons.
		Partially Weathered Rock sampled as: Silty SAND with Gravel (SM), Light gray, Moist to dry, Very dense	50 100	50/1"						
		Refusal at 6.1 feet. Bottom of borehole at 6.1 feet. Auger Refusal								

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Client: **Blacksburg, LLC**			Project No.: **18010224-020203**
Project: **Apogee Townhomes (Clay Street Development)**			Driller: **Blue Ridge Drilling**
Location: **Montgomery County, Virginia**			Method: **2-1/4" HSA w/SPT w/ Autohammer**
Total Depth **13.8'**	Elev GS **2242.0'**	Logged by: **FDP**	Completion Date: **March 23, 2021**

Elev.	Depth	DESCRIPTION (USCS)	Blow Counts / Recovery (%)	N Value	PP (tsf)	% Fines	% H_2O	LL	PI	REMARKS
2242.0	0	Boring B-09A offset approx. 6 feet west of Boring B-09. Auger probe to 8.5 feet. No SPT or sampling.								
2240										
	5									
2235										
	8.5	Fat CLAY (CH), Orange-brown, Moist, Medium stiff	3-3-4 100	7						At completion: Dry to cave-in at approx. 8.3 feet. Borehole backfilled upon completion for safety reasons.
	10									
2230										
	13	Partially Weathered Rock sampled as: Silty SAND with Gravel (SM), Gray, Moist, Very dense	50 100	50/3"						

Refusal at 13.8 feet.
Bottom of borehole at 13.8 feet.
Auger Refusal

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Client: **Blacksburg, LLC**			Project No.: **18010224-020203**
Project: **Apogee Townhomes (Clay Street Development)**			Driller: **Blue Ridge Drilling**
Location: **Montgomery County, Virginia**			Method: **2-1/4" HSA w/SPT w/ Autohammer**
Total Depth **3.0'**	Elev GS **2243.0'**	Logged by: **FDP**	Completion Date: **March 23, 2021**

Elev.	Depth	DESCRIPTION (USCS)	Blow Counts / Recovery (%)	N Value	PP (tsf)	% Fines	% H_2O	LL	PI	REMARKS
		Topsoil ~ 6 inches								
		Fat CLAY (CH), Orange-brown, Moist, Soft	2-3-34 100	37						At completion: Dry to cave-in at approx. 2 feet. Borehole backfilled upon completion for safety reasons.
2240		Silty GRAVEL with Sand (GM), Light Gray, Dry, Dense								
		Refusal at 3.0 feet. Bottom of borehole at 3.0 feet. Auger Refusal								

Client:	**Blacksburg, LLC**	Project No.:	**18010224-020203**
Project:	**Apogee Townhomes (Clay Street Development)**	Driller:	**Blue Ridge Drilling**
Location:	**Montgomery County, Virginia**	Method:	**2-1/4" HSA w/SPT w/ Autohammer**
Total Depth	**6.3'** Elev GS **2243.0'** Logged by: **FDP**	Completion Date:	**March 23, 2021**

Elev.	Depth	DESCRIPTION (USCS)	Blow Counts / Recovery (%)	N Value	PP (tsf)	% Fines	% H_2O	LL	PI	REMARKS
		Boring B-10A offset approx. 8 feet north of Boring B-10. Auger probe to 3.5 feet. No SPT or sampling.								
2240										
	5	Fat CLAY (CH), Orange-brown, Moist, Stiff	3-6-7 33	13						At completion: Dry to cave-in at approx. 5.3 feet. Borehole backfilled upon completion for safety reasons.
			50 100	50/3"						
		Refusal at 6.3 feet. Bottom of borehole at 6.3 feet. Auger Refusal								

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Client:	**Blacksburg, LLC**	Project No.:	**18010224-020203**
Project:	**Apogee Townhomes (Clay Street Development)**	Driller:	**Blue Ridge Drilling**
Location:	**Montgomery County, Virginia**	Method:	**2-1/4" HSA w/SPT w/ Autohammer**
Total Depth **8.5'**	Elev GS **2247.0'** Logged by: **FDP**	Completion Date:	**March 23, 2021**

Elev.	Depth	DESCRIPTION (USCS)	Blow Counts / Recovery (%)	N Value	PP (tsf)	% Fines	% H_2O	LL	PI	REMARKS
		Topsoil ~ 8 inches								
2245		Lean CLAY with Sand (CL), Light Brown, Moist, Stiff	3-5-8 89	13						
			3-6-9 100	15						
	5	Fat CLAY (CH), Orange-brown, Moist, Medium stiff								
2240			2-3-50 88	50/4"			30.0			At completion: Dry to cave-in at approx. 6.8 feet. Borehole backfilled upon completion for safety reasons.
		Partially Weathered Rock sampled as: Silty SAND with Gravel (SM), Brown, Dry, Very dense	50	50/0"						
		Refusal at 8.5 feet. Bottom of borehole at 8.5 feet. Auger Refusal								

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Client: **Blacksburg, LLC**			Project No.: **18010224-020203**
Project: **Apogee Townhomes (Clay Street Development)**			Driller: **Blue Ridge Drilling**
Location: **Montgomery County, Virginia**			Method: **2-1/4" HSA w/SPT w/ Autohammer**
Total Depth **7.5'**	Elev GS **2247.0'**	Logged by: **FDP**	Completion Date: **March 23, 2021**

Elev.	Depth			DESCRIPTION (USCS)	Blow Counts / Recovery (%)	N Value	PP (tsf)	% Fines	% H_2O	LL	PI	REMARKS
				Boring B-11A offset approx. 5 feet west of Boring B-11. Auger probe to refusal at approx. 7.5 feet. No SPT or sampling.								
2245												
	5											At completion: Dry to cave-in at approx. 5.7 feet. Borehole backfilled upon completion for safety reasons.
2240												
				Refusal at 7.5 feet. Bottom of borehole at 7.5 feet. Auger Refusal								

GEOTECH SPT APOGEE TOWNHOMES (CLAY STREET DEVELOPMENT) - FDP.GPJ DF DAA.GDT 4/19/21

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Client: **Blacksburg, LLC**		Project No.: **18010224-020203**	
Project: **Apogee Townhomes (Clay Street Development)**		Driller: **Blue Ridge Drilling**	
Location: **Montgomery County, Virginia**		Method: **2-1/4" HSA w/SPT w/ Autohammer**	
Total Depth **6.1'**	Elev GS **2251.0'** Logged by: **FDP**	Completion Date: **March 22, 2021**	

Elev.	Depth	DESCRIPTION (USCS)	Blow Counts / Recovery (%)	N Value	PP (tsf)	% Fines	% H_2O	LL	PI	REMARKS
		Topsoil ~ 8 inches								
2250		Fat CLAY (CH), trace Gravel, Orange-brown, Moist, Stiff	2-4-7 100	11						
	5		3-5-4 100	9						At completion: Dry to cave-in at approx. 5.5 feet. Borehole backfilled upon completion for safety reasons.
2245		Partially Weathered Rock sampled as: Clayey GRAVEL (GC), Light gray, Dry, Very dense	50 100	50/1"						
		Refusal at 6.1 feet. Bottom of borehole at 6.1 feet. Auger Refusal								

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Client: Blacksburg, LLC	Project No.: 18010224-020203
Project: Apogee Townhomes (Clay Street Development)	Driller: Blue Ridge Drilling
Location: Montgomery County, Virginia	Method: 2-1/4" HSA w/SPT w/ Autohammer
Total Depth 7.5' / Elev GS 2251.0' / Logged by: FDP	Completion Date: March 22, 2021

Elev.	Depth	DESCRIPTION (USCS)	Blow Counts / Recovery (%)	N Value	PP (tsf)	% Fines	% H_2O	LL	PI	REMARKS
2250		Boring B-12A offset approx. 6 feet southeast of Boring B-12. Auger probe to 6 feet. No SPT or sampling.								
	5									
2245		Fat CLAY with Gravel (CH), Brown, Moist, Very stiff	3-4-22 100	26						At completion: Dry to cave-in at approx. 5.8 feet. 3/23/2021: Dry to cave-in at approx. 5.9 feet. Spoon deflecting laterally in third SPT intveral.
		Refusal at 7.5 feet. Bottom of borehole at 7.5 feet. Auger Refusal								

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Client:	**Blacksburg, LLC**	Project No.:	**18010224-020203**
Project:	**Apogee Townhomes (Clay Street Development)**	Driller:	**Blue Ridge Drilling**
Location:	**Montgomery County, Virginia**	Method:	**2-1/4" HSA w/SPT w/ Autohammer**
Total Depth **11.0'**	Elev GS **2245.0'** Logged by: **FDP**	Completion Date:	**March 23, 2021**

Elev.	Depth	DESCRIPTION (USCS)	Blow Counts / Recovery (%)	N Value	PP (tsf)	% Fines	% H_2O	LL	PI	REMARKS
		Topsoil ~ 9 inches								
		Possible Fill sampled as: Clayey SAND with Gravel (SC), Brown, Moist, Loose	10-6-3 61	9						
		Fat CLAY (CH), Brown to orange-brown, Moist, Medium stiff	2-2-5 67	7		97.2	27.3	52	31	
2240	5									
		Sandy Fat CLAY with Gravel (CH), Orange-brown, Moist, Medium stiff to stiff	3-4-6 33	10						
			2-4-3 67	7						At completion: Dry to cave-in at approx. 9 feet. Borehole backfilled upon completion for safety reasons.
2235	10									
		Refusal at 11.0 feet. Bottom of borehole at 11.0 feet. Auger Refusal								

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Client: **Blacksburg, LLC**	Project No.: **18010224-020203**
Project: **Apogee Townhomes (Clay Street Development)**	Driller: **Blue Ridge Drilling**
Location: **Montgomery County, Virginia**	Method: **2-1/4" HSA w/SPT w/ Autohammer**
Total Depth **16.0'** / Elev GS **2245.0'** / Logged by: **FDP**	Completion Date: **March 23, 2021**

Elev.	Depth	DESCRIPTION (USCS)	Blow Counts / Recovery (%)	N Value	PP (tsf)	% Fines	% H_2O	LL	PI	REMARKS
	0	Boring B-13A offset approx. 7 feet southeast of Boring B-13. Auger probe to 13.5 feet. No SPT or sampling.								
2240	5									
2235	10									At completion: Dry to cave-in at approx. 7.8 feet. Borehole backfilled upon completion for safety reasons.
2230	15	Fat CLAY (CH), Orange-brown, Moist, Medium stiff	2-3-4 100	7						
		Refusal at 16.0 feet. Bottom of borehole at 16.0 feet. Auger Refusal								

APPENDIX
Section 3

Laboratory Test Results

Natural Moisture Calculation

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Richmond, VA 23227
Army Corps of Engineers Validated Laboratory

Apogee Townhomes
DAA Project No: 18010224-020203
Prepared By: CBW

Sample ID	B-04
Sample Depth	3.5'-5'

Natural Moisture Content: ASTM D 2216

Pan ID	A1	
Pan Wt	6.68	grams
Pan + Soil (wet)	132.29	grams
Pan + Soil (dry)	106.80	grams
Natural Moisture Content	*25.5%*	

Sample Received:	4/12/2021
Date Test Performed:	4/12/2021

Soil Classification Calculations

Apogee Townhouse
DAA Project No: 18010224-020203
Prepared By: CBW

Draper Aden Associates
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Sample ID	B-07	Sample Recieved:	4/12/2021
Sample Depth	6'-7.5'	Date Tested:	4/12/2021
Visual Sample Description	Brown Fat CLAY		

Natural Moisture Content: ASTM D 2216

Pan ID	311	
Pan Wt	185.35	grams
Pan + Soil (wet)	291.10	grams
Pan + Soil (dry)	262.49	grams
Natural Moisture Content	*37.1%*	

Coarse or Fine Grained: ASTM D422 / D6913

Pan + Soil retained on No. 200 sieve (dry)	185.95	grams
Percent Passing No. 200 Sieve	99.2%	
Pan + Soil retained on No. 4 sieve (dry)	185.35	grams
Percent Passing No. 4 Sieve	100.0%	
Soil Classifies as	*Fine-Grained Soil*	

Atterberg Limits: ASTM D 4318

Date Tested: 4/13/2021

Liquid Limit

No of Blows	15	22	34
Pan ID	103	104	105
Pan Wt	26.29	23.91	25.05
Pan + Soil (wet)	43.90	41.83	42.75
Pan + Soil (dry)	34.17	32.17	33.53
Moisture Content	123.5%	116.9%	108.7%
Liquid Limit	116	115	113
Liquid Limit	*115*		

Plastic Limit

Pan ID	316	317
Pan Weight	9.06	8.08
Pan + Soil (wet)	20.32	18.80
Pan + Soil (dry)	17.55	16.14
Moisture Content	32.6%	33.0%
Plastic Limit	*33*	
Plastic Index	*82*	

USCS Classification: ASTM D 2487

Group Symbol	**CH**
Group Name	**Fat CLAY**

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Grain Size Distribution Calculations

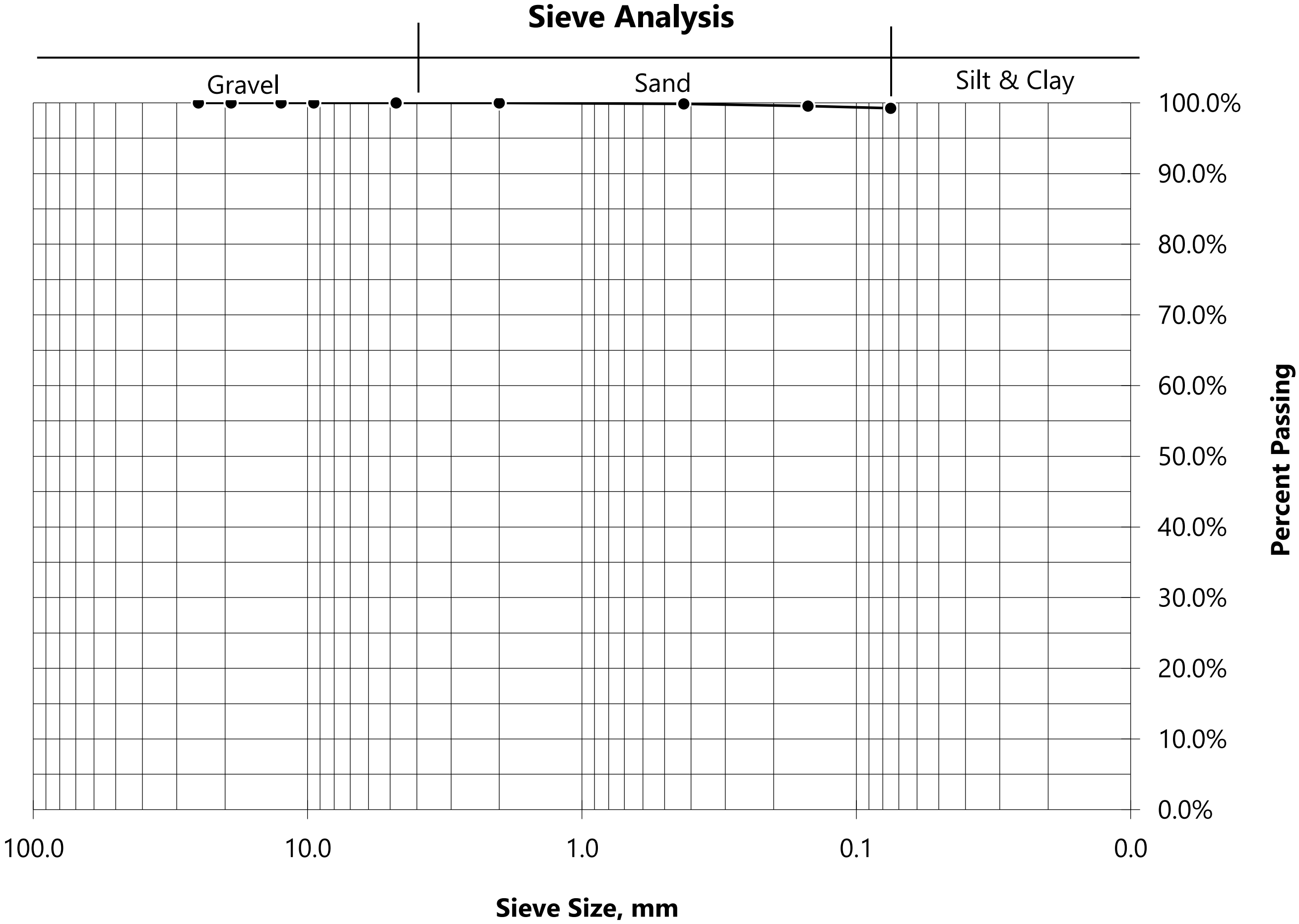
Apogee Townhouse
DAA Project No: 18010224-020203
Prepared By: CBW

Sample ID B-07
Sample Depth 6'-7.5'

Mechanical Sieve Analysis: ASTM D 422

Sieve Size	Weight Retained	Percent Retained	Sieve Size, mm	Percent Passing
1"	0.00	0.0%	25.0	100.0%
3/4"	0.00	0.0%	19.0	100.0%
1/2"	0.00	0.0%	12.5	100.0%
3/8"	0.00	0.0%	9.50	100.0%
No. 4	0.00	0.0%	4.75	100.0%
No. 10	0.01	0.0%	2.00	100.0%
No. 40	0.10	0.1%	0.425	99.9%
No. 100	0.22	0.3%	0.15	99.6%
No. 200	0.25	0.3%	0.075	99.2%
Pan	0.02	0.0%		
Total	0.60	0.8%		

Sieve Analysis



Natural Moisture Calculation

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Apogee Townhomes
DAA Project No: 18010224-020203
Prepared By: CBW

Sample ID B-11
Sample Depth 6'-7.5'

Natural Moisture Content: ASTM D 2216

Pan ID	I	
Pan Wt	6.72	grams
Pan + Soil (wet)	127.36	grams
Pan + Soil (dry)	99.53	grams
Natural Moisture Content	*30.0%*	

Sample Received: 4/12/2021
Date Test Performed: 4/12/2021

Soil Classification Calculations

Apogee Townhouse
DAA Project No: 18010224-020203
Prepared By: CBW

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Sample ID	B-13	Sample Recieved:	4/12/2021
Sample Depth	3.5'-5'	Date Tested:	4/12/2021
Visual Sample Description	Brown Fat CLAY		

Natural Moisture Content: ASTM D 2216

Pan ID	305	
Pan Wt	187.65	grams
Pan + Soil (wet)	309.33	grams
Pan + Soil (dry)	283.21	grams
Natural Moisture Content	*27.3%*	

Coarse or Fine Grained: ASTM D422 / D6913

Pan + Soil retained on No. 200 sieve (dry)	190.37	grams
Percent Passing No. 200 Sieve	97.2%	
Pan + Soil retained on No. 4 sieve (dry)	187.79	grams
Percent Passing No. 4 Sieve	99.9%	
Soil Classifies as	*Fine-Grained Soil*	

Atterberg Limits: ASTM D 4318

Date Tested: 4/13/2021

Liquid Limit

No of Blows	19	26	33
Pan ID	100	101	102
Pan Wt	27.43	24.04	25.75
Pan + Soil (wet)	43.68	41.30	46.28
Pan + Soil (dry)	37.86	35.40	39.59
Moisture Content	55.8%	51.9%	48.3%
Liquid Limit	54	52	50
Liquid Limit	*52*		

Plastic Limit

Pan ID	313	314
Pan Weight	9.14	9.12
Pan + Soil (wet)	20.51	21.27
Pan + Soil (dry)	18.53	19.15
Moisture Content	21.1%	21.1%
Plastic Limit	*21*	
Plastic Index	*31*	

USCS Classification: ASTM D 2487

Group Symbol	**CH**
Group Name	**Fat CLAY**

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Grain Size Distribution Calculations

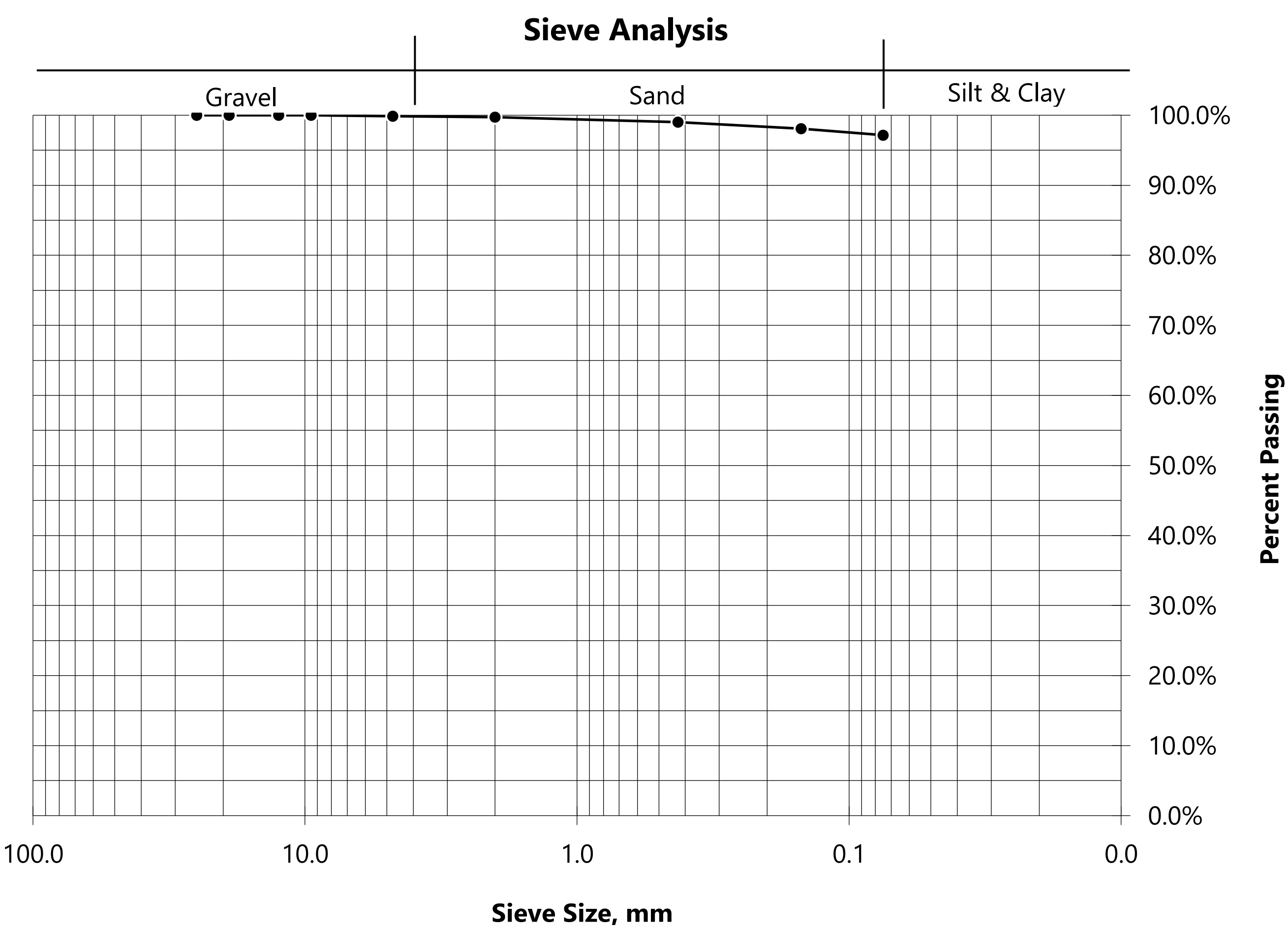
Apogee Townhouse
DAA Project No: 18010224-020203
Prepared By: CBW

Sample ID B-13

Sample Depth 3.5'-5'

Mechanical Sieve Analysis: ASTM D 422

Sieve Size	Weight Retained	Percent Retained	Sieve Size, mm	Percent Passing
1"	0.00	0.0%	25.0	100.0%
3/4"	0.00	0.0%	19.0	100.0%
1/2"	0.00	0.0%	12.5	100.0%
3/8"	0.00	0.0%	9.50	100.0%
No. 4	0.14	0.1%	4.75	99.9%
No. 10	0.13	0.1%	2.00	99.7%
No. 40	0.67	0.7%	0.425	99.0%
No. 100	0.88	0.9%	0.15	98.1%
No. 200	0.89	0.9%	0.075	97.2%
Pan	0.01	0.0%		
Total	2.72	2.8%		



APPENDIX
Section 4

Geotechnical Test Methods

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Blacksburg ◆ Richmond, Virginia
Engineering ◆ Surveying ◆ Environmental Services

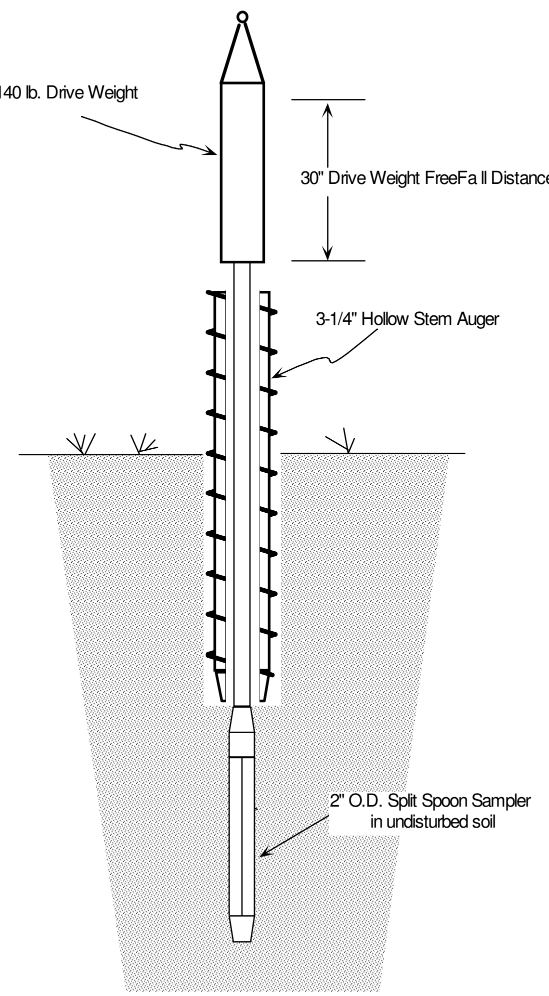
Standard Penetration Test

Split Spoon Sampling is an in-situ technique of obtaining samples of both cohesive and cohesionless soils. The sample is taken by actually driving the split spoon sampler into the “undisturbed” soil at the bottom of the bore hole. The bore hole is advanced using a hollow stem auger.

The Split Spoon Sampler is made up of a split steel barrel with a ball check valve in the head for venting and a hardened steel shoe for driving. A spring sample retainer is used between the shoe and the barrel to retain any loose or flowing materials. After the sampler is driven, the head and the shoe are removed and the barrel opens into two halves exposing the entire sample.

The use of a 140 lb. drive weight falling freely 30" to drive the 2" O.D. (1-$^{3}/_{8}$" I.D.) split spoon sampler a distance of one foot is known as the Standard Penetration Test. Once the sampler is lowered to the bottom of the borehole, the sampler is driven continuously for 18". The number of blows required by the 140 lb. weight to drive the sampler is recorded. Separate counts are made for the second 6" and the third 6" with the first 6" considered to be seating the sampler. An N-Value is obtained by adding the second and third 6" intervals and recorded. The N-Value correlation is shown below:

Standard Penetration Test Diagram



Soil Strength

Relative Density

Coarse Grained Soil, SAND

N-Value	Relative Density
0-4	Very Loose
5-10	Loose
11-30	Medium Dense
31-50	Dense
>50	Very Dense

Consistency

Fine Grained Soil, SILT or CLAY

N-Value	Relative Density
0-1	Very Soft
2-4	Soft
5-8	Medium Stiff
9-15	Stiff
16-29	Very Stiff
>29	Hard

SPT performed in accordance with ASTM D1586, Standard Method for Penetration Test and Split-Barrel Sampling of Soils.

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Moisture Content

Naturally occurring soils nearly always contain water as part of their structure. The moisture content of a soil is assumed to be the amount of water within the pore space between the soil grains which is removable by oven drying at 110^{o}C, expressed as a percentage of the mass of dry soil. By ‘dry’ is meant the result of oven drying at that temperature to constant mass, usually for a period of about 12-14 hours. In non-cohesive granular soils, this procedure removes all water present.

There are several ways in which water is held in cohesive soils, which contain clay minerals existing as plate-like particles of less than 2 μm across. The shape and very small size of these particles, and their chemical composition, enable them to combine with or hold on to water by several complex means as follows:

1) Adsorbed water is held on the surface of the particle by powerful forces of electrical attraction and virtually in a solid state. This water cannot be removed by oven drying at 110^{o}C, and may, therefore, be considered a part of the solid soil grain.

2) Water which is not so tightly held and can be removed by oven drying, but not by air drying.

3) Capillary water, held by surface tension, generally removable by air drying.

4) Gravitational water, which can move within the voids between soil grains, is removable by drainage.

5) Chemically combined water, in the form of water of hydration within the crystal structure. Except for gypsum, and some tropical clays, this water is not generally removable by oven drying.

Moisture content is usually expressed as a percentage, always on the basis of oven-*dry* mass of soil. The equation for the determination of moisture content is:

$$w(\%) = \frac{m_w}{m_d} x100$$

where ,

m_w = mass of water removed at 110^{o} C.

m_d = mass of dried soil

The following ASTM (American Society for Testing and Materials) apply to moisture content determinations:
ASTM D2216-90 Laboratory Determination of Water Content of Soil and Rock
ASTM D4959 -89 Determination of Water Content of Soil By Direct Heating Method
ASTM D4643-87 Determination of Water Content of Soil by the Microwave Oven Method
ASTM D3017-88 Water Content of Soil and Rock in Place by Nuclear Methods

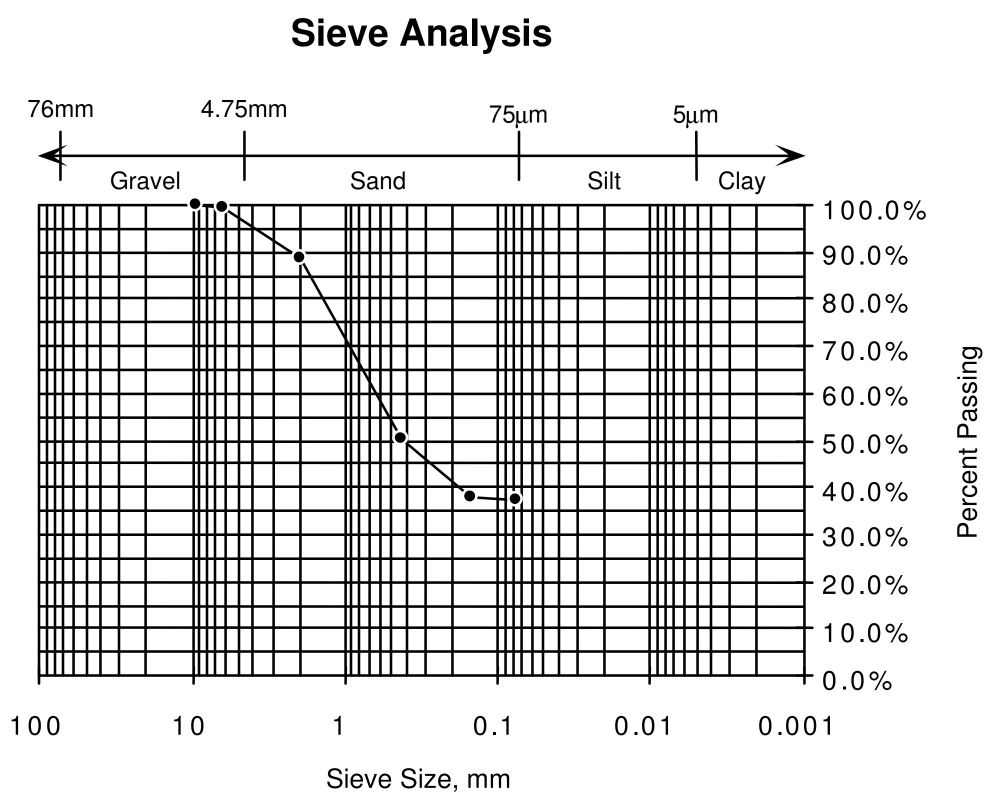
Particle Distribution

A soil consists of an assemblage of discrete particles of various shapes and sizes. The object of a particle size analysis is to group these particles into separate ranges of sizes, and so determine the relative proportions, by dry weight, of each size range.

Particle size analyses consist of two separate and quite different procedures in order to span the very wide range of particle sizes which are encountered. These are sieving and sedimentation procedures. Sieving is used for gravel and sand size (coarse) particles, which can be separated into different size ranges with a series of standard aperture openings. Sieving cannot be used for the very much smaller silt and clay size (fine) particles, so a sedimentation procedure is used instead. Measurements of the density of the suspension are made using a hydrometer.

For soils containing both coarse and fine particles, composite tests using both sieving and sedimentation methods may be used if a full particle size distribution analyses is required. Particle size testing can range from a simple sieving test on a ‘clean’ sand and gravel, to elaborate composite tests on clay-silt-sand-gravel mixtures.

Presentation of particle size distribution data may include a table showing the percentages, by dry weight, of particles finer than certain standard sizes and may include a graphical presentation of the percentages plotted against the particle size on a logarithmic scale. An example of the graphical presentation with respective particle sizes follows:



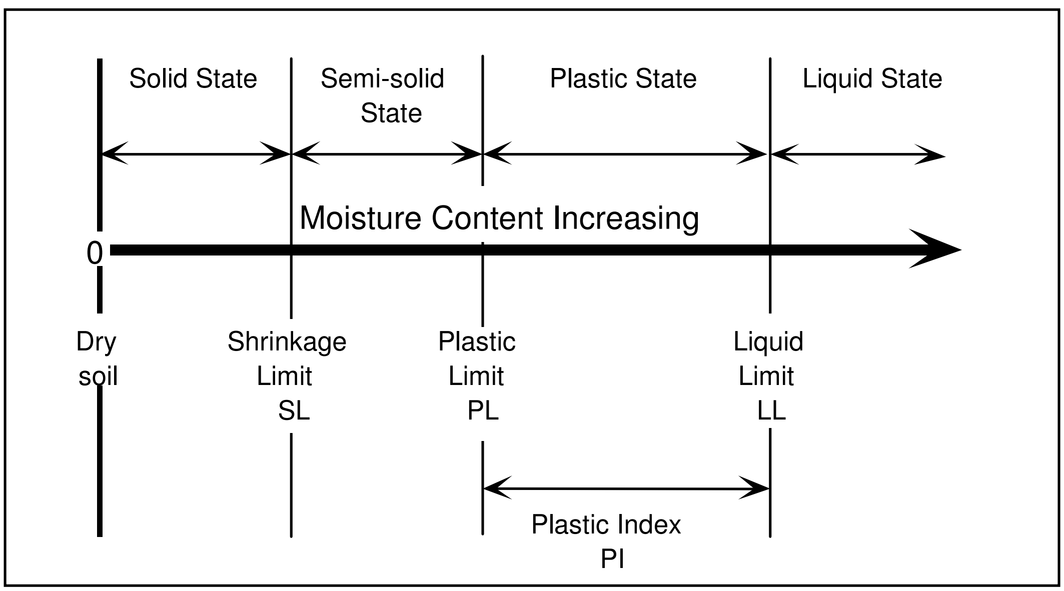
Particle size analyses are performed in accordance with ASTM D6913, Standard Test Method for Particle-Size Analysis of Soils or ASTM C136-84, Standard Method for Sieve Analysis of Fine and Coarse Aggregates.

The condition of a clay soil can be altered by changing the moisture content; the softening of clay by the addition of water is a well known example. For every clay soil there is a range of moisture contents within which the clay is of a plastic consistency, and the Atterberg limits provide a means of measuring and describing the plasticity range in numerical terms.

If sufficient water is mixed with a clay, it can be made into a slurry, which behaves as a viscous liquid. This is known as the 'liquid' state. If the moisture content is gradually reduced by allowing it to dry out slowly, the clay eventually begins to hold together and to offer some resistance to deformation; this is the 'plastic' state. With further loss of water the clay shrinks and the stiffness increases until there is little plasticity left, and the clay becomes brittle; this is the 'semi-solid' state. As drying continues, the clay continues to shrink in proportion to the amount of water lost, until it reaches the minimum volume attainable by this process. Beyond that point further drying results in no further decrease in volume, and this is called the 'solid' state.

These four states, or phases, are shown diagrammatically below. The change from one phase to the next is not observable as a precise boundary, but takes place as a gradual transition. Nevertheless three arbitrary but specific boundaries have been established empirically, as indicated below, and are universally recognized. The moisture contents at these boundaries are known as the Liquid Limit (LL), Plastic Limit (PL) and the Shrinkage Limit (SL).

The moisture content range between the PL and the LL is known as the Plastic Index (PI), and is a measure of the plasticity of the clay. Cohesionless soils have no plasticity phase, so their PI is zero.



Atterberg limits are performed in accordance with ASTM D4318-84, Standard Test Method for Liquid Limit, Plastic Limit and Plasticity Index of Soils.