Report of Subsurface Exploration and Geotechnical Engineering Evaluation
Slope Evaluation Remediation Study
Ashborough Village Condominiums

Ashborough Road
Marietta, Cobb County, Georgia
PGC Project No. 109027

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PIEDMONT
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July 10, 2009

Ashborough Village Condominium Association
1810 Ashborough Circle
Marietta, Georgia 30067

Attention: Ms. Ginny Swancy, Manager

Subject: **Report of Subsurface Exploration and**
Geotechnical Engineering Evaluation
Slope Evaluation Remediation Study
Ashborough Village Condominiums
Ashborough Road
Marietta, Cobb County, Georgia
PGC Project No. 109027

Dear Ms. Swancy:

Piedmont Geotechnical Consultants, Inc. is pleased to provide this report of our subsurface exploration and geotechnical engineering evaluation for the referenced project. The field study and this report were accomplished in general accordance with PGC Proposal No. P9039, dated February 5, 2009.

The following report will present a brief summary of our pertinent findings and recommendations followed by our understanding of the proposed construction, methods of exploration employed, site and subsurface conditions encountered, and conclusions and recommendations regarding the geotechnical aspects of the project.

We sincerely appreciate the opportunity to provide our geotechnical engineering services to you on this project and look forward to our continued involvement during plan development and quality control testing during construction. Should you have any questions regarding items discussed in this report, please do not hesitate to contact the undersigned.

Sincerely,
Piedmont Geotechnical Consultants, Inc.

H. Craig Robinson, P.E.
Senior Registered Engineer
Registered Georgia 19121

cc: Addressee (3)
Mr. Mark Lee, P.E. – Paul Lee Consulting Engineering Associates (1)
Mr. Robert Kendall – Kendall and Associates (1)

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APPENDIX

1.0 SUMMARY

The following is a brief summary of our pertinent findings and recommendations. The reader is referred to the remaining text of this report for elaboration on these items.

1. Stream bank erosion along Rottenwood Creek initiated slope failures behind Buildings 1811 and 1189.

2. A geotechnical subsurface exploration was performed to collect subsurface data for design of slope remediation measures. General subsurface conditions consist of previously placed fill materials and alluvial materials underlain by residual soil, partially weathered rock and refusal materials with increasing depths. Previously placed fill materials ranged in depth from 10 to 13 feet below existing grades. Alluvial material was underlying the fill and ranged in depth from 8 to 13 feet below existing grades and extended to depths of 24 to 28 feet below existing grades. Residual soils were encountered at depths of 24 to 28 feet below existing grades and extended to depths of 32 to 40 feet below existing grades. Partially weathered rock (PWR) was encountered in borings B-1, B-4, and B-6. PWR in boring B-1 extended from 32 to 35 feet below exiting grade. Boring B-2 encountered PWR from 31 to 38 feet below existing grade and boring B-6 encountered PWR from 26 to 27 feet below existing grade. Groundwater was encountered at depths of 16 to 17 feet below existing grades.

3. Various slope remediation options have been considered. Due to physical site constraints and the need to provide positive protection to the shallow supported structures, a structural tied-back wall system consisting of either an H-pile/lagging and shotcrete face or steel sheet piles is recommended.

4. Final design and construction of the slope remediation measures will involve a specialty design-build contractor.

5. Several non-geotechnical issues (environmental/regulatory) remain to be settled that may ultimately impact the design-build efforts. It is our understanding that Paul Lee Consulting Engineering Associates (PLCEA) and Kendall and Associates are performing these tasks. As these issues are resolved, or limitations and directives given, some revision to the geotechnical recommendations may be required.

2.0 INTRODUCTION

2.1 Project Information

Our understanding of this project is based on information provided by Mr. Mark Lee, P.E. of Paul Lee Consulting Engineering Associates (PLCEA), various meetings and discussions with Mr. Lee, Mr. Robert Kendall, Kendall and Associates, representatives of the Ashborough Village

Condominiums, our site observations and performing this authorized subsurface study. Ashborough Village is a condominium complex located in Marietta, Cobb County, Georgia and is generally bounded by US Highway 41 to the west, Delk Road to the south, Franklin Road to the east and Rottenwood Creek to the north. Several two-story wood framed buildings border Rottenwood Creek. Rottenwood Creek is a deeply incised channel estimated to be about 10 feet deep below the general floodplain level and about 30 feet wide that at the site generally flows along the south side of a wide floodplain area. Several large diameter logs from fallen trees can be seen exposed in the stream banks. Considerable household/automobile debris is present in the stream channel. The creek has eroded and undermined portions of the existing fill/alluvium slope at the east end of Building 1189 and the west end of Building 1811. At both locations the stream channel makes abrupt turns directing flows against the near stream bank. Additional tree fall in the creek and storm water outfalls potentially have affected the stream flow, resulting in increased scour/erosion in these localized areas.

The current failed slope conditions behind Building 1189 have encroached within 20 feet of the northeast corner of the structure. It is our understanding the building is supported on shallow foundations. We understand some of the condo owners believe the ground may have settled a little behind Building 1189, but no building (structural) distress has been observed/reported. The chain link fence which borders the creek has been undermined by recent ground loss. The slope failure scarp behind Building 1189 has resulted in a 5 to 10 foot near vertical face beginning at the chain link fence line that then slumps into the creek channel. The available survey data suggests the natural slope was about 1(H):1(V). The slope is vegetated with low growth bushes and a few large trees. A mostly rotted tree stump, located about 5 to 10 feet into the stream, beyond the current toe of slope, likely marks a historical stream bank location. A sanitary sewer parallels the creek between the current stream channel and the buildings. The sewer invert is about 15 feet below the ground surface (approximately the flow line of the stream). Several shallow buried utilities are also present in the area between the building and the creek. A partially collapsed storm drain outfalls into the creek about 40 feet east of Building 1189. The stream channel slope is severely eroded in the area of the outfall. The headwall has dropped away from the corrugated metal pipe.

The failed slope condition behind Building 1811 has resulted in a semi-circular failure scarp about 30 to 40 feet along the creek and about 15 feet wide, which has dropped about 1 to 2 foot. The scarp is about 20 feet away from the nearest building element. The natural stream channel slope is oriented about 1(H):1(V), and is vegetated with medium size trees and small undergrowth. A sanitary sewer is located between the creek and building line. The sewer is about 14 to 15 feet deep. A storm drain pipe passes between Buildings 1811 and 1809 and outfalls to the creek. Several shallow buried utilities exist between the stream and the building. A portion of this area was previously stabilized using rock and chain-link fencing.

Based on our discussions with Mr. Lee, we understand upgrades to the storm drain system are currently being evaluated by his firm.

2.2 Purpose and Scope of Services

The primary purpose of this study was to obtain sufficient subsurface data in the areas identified so that appropriate slope remediation measures can be taken to recreate a stable stream channel and provide adequate support for the nearby structures. This report is intended to provide sufficient subsurface information and to address items of design and construction related to geotechnical engineering. This study and report is not intended to address any environmental or other regulatory limitations imposed; although we will attempt to incorporate any restrictions imposed by the regulatory community into our final recommendations. It is our understanding that Mr. Bob Kendall of Kendall and Associates, Inc. has been retained to provide environmental consulting services.

Our authorized scope of services included a site reconnaissance, soil test and hand auger borings, geotechnical engineering evaluation of the data obtained, preparation of this report and supplemental engineering consulting during plan and specification development.

3.0 METHODS OF EXPLORATION

3.1 Site Reconnaissance

Prior to and during our field exploration, detailed site reconnaissances were performed by engineers from our office to observe surface conditions along the areas under study. The observations and information obtained during these visits were used in planning and adjusting the field exploration, and relating site conditions to known geologic and subsurface conditions in the area.

3.2 Field Exploration

To evaluate the subsurface conditions in the area of the active slope failures, we drilled six mechanical soil test borings to depths ranging from 13 to 40 feet below the existing grade, supplemented by drilling 11 hand auger borings to depths ranging from 1.5 to 10 feet below the existing grades in areas generally not accessible to the mechanical drill rig. Hand auger borings HA-1 through HA-3 and HA-7 through HA-10 were located in the Rottenwood Creek channel. The borings were located in the field by the engineers by measuring distances and estimating directions from identifiable site features. The placement of the borings, especially the mechanical soil testing borings, was limited due to physical site limitations, and the presence of several underground utilities that pass between the buildings and creek. Therefore, their locations as shown on the Site and Boring Location Plans in the Appendix should be considered approximate.

The mechanical soil test borings were advanced by twisting continuous hollow stem auger flights into the ground. At selected intervals, Standard Penetration Resistance Testing (SPT) was performed in general accordance with ASTM standard D-1586, and soil samples were collected for visual classification.

The hand auger borings were advanced by manually twisting a bucket auger into the ground. At selected intervals, Dynamic Cone Penetration Testing (DCP) was performed, and soil samples were visually classified. Hand auger borings conducted in the creek channel (below the groundwater table) were cased with 4 inch PVC pipe to prevent stream flow from filling the borehole and to stabilize the open borehole. The casing was manually advanced as the hand auger boring was deepened. The DCP tests were performed just below the casing.

The results of the penetration tests (SPT and DCP), when properly evaluated, provide an indication of the relative consistency of the soil being sampled, the potential for difficult excavation, and the soil's ability to support loads. A more detailed description of the mechanical drilling and sampling process is included in the Appendix of this report.

Soil samples recovered during the drilling process were classified in the field by the geotechnical engineer/geologist in general accordance with the Unified Soil Classification System (USCS). Detailed descriptions of the materials encountered at each mechanical boring location, along with a graphical representation of the Standard Penetration Test results, are shown on the Soil Boring Records in the Appendix. Elevations on the Soil Boring Records were interpolated from the topographical contours on the plan provided to us and should be considered approximate. A Summary of Hand Auger Borings is provided which includes a description of the soils encountered along with the numerical DCP results.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Geology

The site is located in the Piedmont Physiographic Province of Georgia. The residual soils in the Piedmont are the result of the chemical and physical weathering of the underlying parent rock. The weathering profile usually results in fine grained clayey silts and silty clays near the surface, where weathering is more advanced. With depth, sandy silts and silty sands are found, often containing mica. Below the residual soils, partially weathered rock is often found as a transitional material above relatively unweathered rock. In local practice, partially weathered rock is arbitrarily defined as residual soils with Standard Penetration Resistances in excess of 100 blows per foot (50 blows per 6 inches), and which can be penetrated by a power auger. The natural weathered profile has been altered by the placement of man-made fill materials and water deposited alluvial sediments.

4.2 Subsurface Conditions

The following sections describe the subsurface conditions encountered during this study. The following description of subsurface conditions has been generalized and brief. We refer the reader to the attached Soil Boring Records, Summary of Hand Auger Borings and Figures 3, 4 and 5: Subsurface Profiles. Figures 3 through 5 graphically depict the generalized subsurface data.

Mechanical soil test borings performed near buildings 1811, 1809 and 1189 encountered subsurface conditions consisting of previously placed fill materials underlain by alluvial soils, transitioning to residual and partially weathered rock (PWR) materials and eventually auger refusal materials (presumed rock) with increasing depth. Hand auger borings performed in the creek channel encountered alluvial materials from the ground surface to termination/refusal depths. Zones of organic laden alluvial materials were encountered. Groundwater was encountered in most borings performed for this study.

4.2.1 Previously Placed Fill

Soil test borings B-1 through B-6 encountered previously placed fill materials to depths extending from the current ground surface to a maximum depth of 13 feet. The fill was classified primarily as silty sands (SM), with Standard Penetration Tests (SPT) ranging from 4 to 49 blows per foot (bpf). In borings B-2 through B-5 the SPT values generally ranged from 4 to 7 bpf in the upper 5 to 8 feet but were generally greater than 10 bpf in borings B-1 and B-6. The higher SPT values in borings B-1 and B-6 may indicate better soil compaction away from the creek or the presence of rock and concrete pieces in the soil matrix that may have amplified the SPT values. The lower SPT values in borings B-2 through B-5 may be indicative of lower compaction in non-structural or utility backfill areas. Boring B-6 encountered a zone of possible fill from 8 to 13 feet. The possible fill materials contained significant organics. In general, the SPT values varied significantly. SPT values less than 6 generally indicate soils that are considered low consistency (poorly compacted). During the drilling of boring B-5, we encountered a buried electrical line. Boring B-5 reached refusal at a depth of 13 feet on a large log or stump. Due to the presence of underground utilities in this area, boring B-5 was not offset.

Hand auger borings HA-4 through HA-6 encountered previously placed fill materials extending from the ground surface to depths ranging from 6 to 10 feet. The fill materials were classified as silts (ML) and silty sands (SM). Dynamic cone penetrometer testing indicated the fill materials were of low consistency, indicating lower compaction.

4.2.2 Alluvium

Underlying the fill materials in all six soil test borings, alluvial (water deposited) soils were encountered to depths ranging from 24 to 28 feet below existing grades. Boring B-5 reached auger refusal (large stump/log) at a depth of 15 feet. The alluvium encountered in these soil test borings was classified as silty sands (SM) and clayey silts (ML) with SPT ranging from 3 to 80 bpf, but typically ranged from about 5 to 15 bpf. The higher SPT values generally occurred just above the alluvium/residuum interface and likely were amplified by the presence of large gravel at the sample interval. This basal gravel layer is fairly common in large streams in the Piedmont Geology.

Alluvial materials were encountered from the ground surface in hand auger borings HA-1 through HA-3 and HA-7 through HA-10 to depths of 1.5 to 10+ feet below existing grades. Hand auger borings HA-1 encountered refusal at 1.5 feet and was offset and continued as boring HA-1A.

Alluvium was encountered in boring HA-4 at 6 feet and continued 10 feet below existing grade where the boring was terminated. The alluvium encountered in these borings was classified as silty sands (SM) and clayey silts (ML). These soils were generally considered to be of low consistency based on the DCP testing. The higher blow counts were due to coarse sands and small gravel at the drive increment. Groundwater was typically encountered in the alluvial strata. Refusal depths in borings HA-3, HA-7, HA-8, HA-9, and HA-10 likely indicate the upper boundary of the basal gravel that was encountered in the soil test borings just above the alluvium /residuum interface. The thickness of the gravel layer was not determined. Based on the conditions encountered in the soil testing borings, we estimate the gravel layer to be a few feet thick.

4.2.3 Residuum

Residual materials were encountered in borings B-1 through B-4 and B-6. Residual soils weather from the underlying parent rock through chemical and physical processes. Residual soils were encountered at depths of 24 to 28 feet below the ground surface and continued to depths of 32 to 40 feet below existing grades before encountering partially weathered rock or achieving planned boring termination depths. The residuum was classified as silty sands (SM) and exhibited SPT test values ranging from 10 to 66 blows per foot, with 30 bpf being typical.

4.2.4 Partially Weathered Rock

Underlying the residual soils in borings B-1, B-4 and B-6, partially weathered rock (PWR) was encountered. Partially weathered rock is a transitional material between weathered soil and unweathered rock that can be drilled with the soil auger, exhibits SPT values greater than 100 bpf, and retains the relic structure of the parent rock. Partially weathered rock was encountered in boring B-1 from 32 to 35 feet, boring B-4 from 34 to 38 feet, and boring B-6 from 26 to 27 feet.

4.2.5 Refusal Materials

Refusal to the drilling process was encountered in borings B-1, B-4, and B-6 and depths of 35 feet, 38 feet and 27 feet, respectively. The nature of the refusal material can only be determined by advancing the hole below the refusal depth by using rock coring techniques, which were beyond the scope of this work. Refusal to the power auger may be indicative of rock pinnacles, large boulders, rock lenses or the top of mass rock (bedrock).

We note that boring B-5 refused at 13 feet on top of a large stump or log within the alluvium strata and the refusal material was verified by wood pieces in the split-spoon.

4.2.6 Groundwater

Groundwater was encountered in borings B-1 through B-4 and B-6 at the time of drilling. Groundwater ranged in depths of 16 to 17 feet below the existing grades. Small diameter PVC casing was temporarily installed in borings B-1 and B-6 to prevent the boring from collapsing to facilitate stabilized groundwater measurements. Stabilized groundwater levels in borings B-1 and B-6 were measured 15.5 feet and 16 feet below exiting grades, respectively, and generally matched the water level in the creek. We anticipate groundwater levels to be equal or slightly above prevailing creek levels in close proximity to Rottenwood Creek, and likely fluctuate with changing creek levels.

The conditions described in the preceding paragraphs, and those shown in the Appendix, have been based on interpolation of the results of the previously described data using generally accepted principles and practices of geotechnical engineering. However, conditions in this geology may vary intermediate of the tested locations, and even more so on previously developed property.

Although individual soil test borings and hand auger borings are representative of the subsurface conditions at the precise boring locations on the day drilled, they are not necessarily indicative of the subsurface conditions at other locations or other times. The nature and extent of variation between the borings may not become evident until the course of construction. If such variations are then noted, it will be necessary to reevaluate the recommendations of this report after on-site observation of the conditions.

5.0 PROJECT DEVELOPMENT

During the course of this study, beginning with the development of the PGC proposal P9039, dated February 5, 2009, we have posed several issues that do not directly involve geotechnical engineering, but ultimately may impact the slope remediation option chosen. To our knowledge, these issues remain unsettled as of this report. The following key issues need to be properly considered during the development of the slope:

1. Regulatory agency(s) involvement, guidelines, limitations.

2. Impact of existing utilities, especially sanitary sewer line.

3. Determine if buildings are experiencing movement due to current stream bank erosion.

4. Allowable physical stream encroachment limits.

5. Construction limitations due to physical constraints.

It is our understanding no available geotechnical, civil design and/or construction documents have been located that provide information about the original project design or construction. As such, we have made certain assumptions based purely on visual observation and historical performance.

On April 7, 2009, Mr. Robinson visited the site with Mr. Daniel Brahana, P.E. of ABE Enterprises, Inc., a local specialty geotechnical contractor, experienced in excavation/slope bracing. We observed the site conditions and discussed potential slope remediation solutions.

On April 10, 2009, we participated in a meeting at the site with representatives of the HOA and Mr. Mark Lee. During this meeting, we discussed our findings and our preliminary recommendations for the slope remediation, addressing only the two specific areas of obvious distress at this time. Preliminary costing for the remediation project was briefly discussed. During this meeting, we discussed the need to determine the stream encroachment limit in which the regulatory agencies would allow Ashborough Village HOA to re-establish their buildable limits. The HOA was going to research the availability of historical site topography that might reflect earlier stream boundaries that could be the basis for obtaining permits and developing design documents. Mr. Lee recommended consulting with Mr. Bob Kendall of Kendall and Associates for determining regulatory limitations and obtaining environmental related permits. PLCEA will consult with Marietta/Cobb County regarding their regulatory or utility involvement. PLCEA will provide survey monitoring of the existing buildings to determine if structure movements are occurring. We understand the existing sewer is a 12-inch diameter clay pipe and Cobb County is considering relocating this sewer to in front of the buildings further away from the creek. At this time we are unaware as to whether Cobb County will be willing to relocate this sewer in advance of the slope remediation construction.

On April 15, 2009, we met with Mr. Bob Kendall of Kendall and Associates, Inc. and Mr. Mark Lee to review the site conditions. Mr. Kendall will provide environmental consultation and serve as the liaison with the USACE.

On June 16, 2006, PGC provided sketches of our suggested preliminary wall alignments to Mr. Bob Kendall and Mr. Mark Lee for their use in discussing this project with the USACE. These same sketches are presented as Figures 1 and 2 in this report.

6.0 FINDINGS, CONCLUSIONS AND RECOMMENDATIONS

6.1 Findings

The findings discussed in this section are based primarily on observations at the site during various site visits and the subsurface data collected during the field investigation program. No historical site, design or construction documents have been provided.

Based on our visual observations, the Ashborough Village side of Rottenwood Creek (southern side) is noticeably several feet higher than the opposite side of the stream channel. We anticipate the original site development consisted of clearing the trees and placement of primarily soil materials to raise existing grades along Rottenwood Creek within the floodplain so that the existing structures could be constructed above the then required flood elevation. The boring data suggests that about 8 to 13 feet of fill materials were placed to raise original grades to the existing grades of about 947 to 948 feet at the buildings and the fill materials were placed directly over the alluvial (floodplain sediment) soils historically deposited by Rottenwood Creek. The quality and consistency of the fill materials appears to vary depending on its proximity to the creek. In general, the fill consistency (percent compaction) appears to be lowest nearest the creek, at the outer edges of the slope. We suspect the fill materials could have been placed in a thickened "bridge" lift over the weaker, possibly unstable, alluvial soils until a stable platform was created followed by more reasonably controlled lifts and improved compaction effort to reach the final grades. This practice was and is still a fairly common approach when filling over marginal subgrade conditions. Fill materials encountered near the front of the buildings (soil borings B-1 and B-6) appear to have received more compactive effort than towards the creek side.

The lower consistency fill materials found generally between the buildings and creek may also be impacted by the presence of several existing below grade utilities. Compaction of utility backfill is often lower than fills placed during mass grading efforts. It is likely some of these utilities have also been replaced/repaired since their original installation.

An existing sanitary sewer generally parallels Rottenwood Creek and its presence will likely impact or interfere with any planned slope remediation measures. We suspect the sanitary sewer was installed prior to the construction of the buildings, but we are not sure if the sewer pre-dates the overall site development. The presence of the gas line, electric service and telephone/cable utilities will all have to be properly considered during the remediation project, but likely pose less impact/risk to the planned construction, but their relocation/replacement may be necessary.

The existing buildings appear to be supported by conventional shallow foundations; either isolated, continuous, and/or monolithic slab/foundation. No plans have been located to verify the foundation system. To our knowledge, based on our discussions with you, no building distress has been reported to date by the owners in Building 1189.

We observed the placement of small riprap and chain link fence in the area of our boring B-4 along the near stream channel bank. We understand you placed these materials in hopes of stabilizing the stream bank some time ago. To some degree, these efforts may have helped mitigate the downstream extent of the current failure behind Building 1811.

At both active slope failures, we observed the presence of nearby storm drain pipes emptying into Rottenwood Creek. Both pipes are damaged and in need of repair, especially downstream of Building 1189 where a large scour in the stream bank has formed and the headwall has fallen into the creek. The placement and orientation of the storm drain outfalls relative to stream flow (pipes discharging perpendicular to upstream of stream flow) may be aggravating the stream flow regime causing turbulence and exacerbating the erosion processes of the stream bank.

6.2 Conclusions and Recommendations

The following geotechnical conclusions and recommendations are based on the data gathered during this exploration, our understanding of the proposed project, our experience with similar site and subsurface conditions and generally accepted principles and practices of geotechnical engineering. Should the project parameters construction change significantly from that described in this report, we request that we be advised so that we may amend these recommendations accordingly. This report and the conclusions and recommendations provided herein are provided exclusively for the use of the Ashborough Village Condominium Homeowners Association and are intended solely for design of the referenced project.

6.2.1 General

At the time of this report, there remained several issues that may ultimately impact the design and construction of the needed slope remediation activities. These unresolved issues primarily involve regulatory and environmental concerns, access, existing utilities and which ultimately can impact the design and cost for construction. As will be discussed subsequently in this report, our recommendations for the slope remediation will require specialty geotechnical construction. In most cases, these type projects are best accomplished using a design/build concept where the selected specialty contractor will evaluate the geotechnical and civil data provided to determine the most cost effective solution to accomplish the desired results. As such, we are of the opinion the HOA should begin discussions with qualified specialty contractors so that their experienced input can be utilized in developing the final design solution. We also suggest discussions with Marietta/Cobb County be conducted regarding the existing sanitary sewer line and their requirements for maintaining, upgrading and/or relocating the sewer so that its impact on the construction and the development are properly considered. Until the physical encroachment limits at the stream are established by others, some of the information and recommendations provided herein are based on our best assumptions and approximations. As such, some of the commentary in this report may have to be revised; however, we have assumed the maximum encroachment for any constructed slope remediation cannot go beyond the current toe of slope where the current stream bank and creek channel meet. Furthermore, we understand the chosen slope remediation option cannot restrict flows or otherwise reduce the stream hydraulics. The hydraulics and hydrology (H and H) aspects of this project are outside our area of expertise, but are important design considerations and should be addressed by others in the overall evaluation of the slope rehabilitation project.

As part of this geotechnical evaluation, we have considered various slope stabilization options available, including riprap stabilization, rock filled gabions, and tied-back wall systems. Based on our understanding of the project conditions, requirements and limitations that are further complicated by the presence of the sewer and the close proximity of the buildings to the active stream channel, we are of the opinion typical slope armoring options are not considered valid. As such, we feel a relatively impermeable structural tied-back wall system is required to stabilize the active slope failure areas and prevent future ground loss that could impact the building and utility support conditions. On June 16, 2009, PGC provided two sketches (Figures 1 and 2) that illustrate our suggested wall alignment. The illustrated wall alignments are based on our site observations and do not reflect any other regulatory or H and H considerations. As such, the wall alignments depicted should be considered approximate and preliminary at this time. Other remediation options may be suitable for this project, but have not been evaluated to date.

The following report sections will provide limited discussion of the various options considered, and will elaborate somewhat on the system recommended.

6.2.2 Riprap Rock Stabilization Blanket

Large riprap rock placed as a stabilization blanket on a prepared subgrade to provide armoring against stream erosion was considered. Typically the rock is placed on a prepared (smoothed, compacted, sloped) subgrade. The rock size and thickness varies based on calculated or measured stream velocity. Geotextile fabrics or small filter aggregates are placed between the soil and large rock to provide filtering or separation. This type of protection although common should be considered temporary and requires routine maintenance. The HOA has previously placed riprap rock and chain link fencing behind Building 1811. The stone/fence appears to have helped protect a portion of the stream bank in this area, but has failed to provide positive protection. This system is porous and soil loss can occur over time.

It is our opinion that while placement of large (Type I or larger) rock riprap may help buttress the failing slope, and provide protection against future soil erosion, this method of slope protection/remediation does not provide positive long term protection for the nearby structures. To accomplish proper installation the impacted stream banks would need to be flattened [1(H):1(V) or flatter] so that proper sized bedding stones and geotextile fabric can be placed prior to installing the large riprap stone to lessen the risk for future erosion problems. This option, while likely the most economical will result in reduction of yard space behind Buildings 1809 and 1811, and is not considered practical for the Building 1189 location due to the limited distance between the back of building and top of slope (currently less than 20 feet). It is our opinion a more positive protection system is required for long term protection. However, if an interim repair method is considered by the HOA, it is possible the riprap option could be used behind Building 1811. Flattening of the existing stream channel banks may encroach into the sewer and utility easements requiring modifications to this option or relocation of the utilities. Initially, the existing slopes would need to be flattened by cutting to at least 1(H):1(V). A minimum rock section would consist of an 8 ounce or heavier needle-punched, non-woven filter fabric overlain by 9 inches of #57 stone, overlain by 12 inches of small surge stone, overlain by at least 3 feet of Type I riprap. Depending on stream velocities, larger or thicker Type I may be needed.

6.2.3 Gabion Wall

We have also considered a Gabion Wall system for stabilization/remediation. Generally speaking gabions are interlocking aggregate filled wire baskets that are stacked to create a stable gravity wall section. The wire baskets allow the rock to be stacked in near vertical building blocks and retained. This option provides more positive protection to the slope than a layer of loose riprap because a more substantial zone of rock is formed that has an added structural component provided by the wire baskets. The rock filled wire baskets allow for a gravity wall section to be built using medium size aggregate (3 inch to 6 inch diameter) to create a more compact, less porous zone. The smaller stone is used in conjunction with geotextile filters to minimize soil loss.. This option requires more substantial site preparation to facilitate construction of a stable gabion wall unit. The typical gabion wall geometry includes a ½ to 1(H):1(V) front slope and near vertical back slope (earth side). Sometimes the back slope of the gravity section is sloped. The base width of the wall is about one-half to two-thirds the wall height based on soil conditions to prevent a soil bearing capacity failure. The wall construction requires a properly sloped and dewatered excavation because the lower portion of the gabion wall would extend several feet below the flow line in the stream to prevent future scour and undermining that could collapse the wall. To accomplish construction of this type wall, a sufficiently sized stable excavation would be required to accommodate a wall section with a base width of about 15 to 20 feet plus a flattened back slope and offsets for equipment and personnel. The stream would have to be forcibly relocated away from the excavation and the excavation positively dewatered so work could be accomplished in the dry. Based on our evaluation of the site conditions, the sloped excavation necessary for this option undermines Building 1189 and comes close to Building 1811 and impacts the existing sewer at both locations. Therefore, it is our opinion the site limitations coupled with the stream diversion and dewatering requirements make this option unacceptable. Therefore, this option has not been further considered.

6.2.4 Tied-Back Wall System

We have also considered using some form of tied-back wall system to stabilize the stream banks. This option involves the installation of a structural wall element that is restrained by earth/rock anchors to resist "at-rest" earth and hydrostatic pressures. Two different wall systems are acceptable. Other wall options such as modular block or cast-in-place may be acceptable. We have considered both an H-pile with lagging and shotcrete facing and an interlocking sheet pile wall system for this project; however, each has specific limitations or shortcomings that need proper consideration. In our opinion, the H-pile/lagging/shotcrete wall offers the most flexible installation methods, but typically does not provide adequate scour protection because the shotcrete placement below the stream level is not easily accomplished, requiring additional scour protection considerations. The interlocking sheetpile wall provides adequate scour protection which is an advantage over the H-pile wall, but we are concerned that adequate tip embedment cannot be achieved due to the subsurface conditions, possibly requiring a second row of tiebacks near the tip and additional armoring. The sheetpile wall system also requires installation using a driven or vibrating hammer that could cause structural damage to the buildings (i.e., cracking of sheetrock, masonry) that could result in additional repairs and cost. Historically, the sheetpile wall has been a more expensive option to the H-pile wall primarily because of material cost; however the material cost may not be as significantly different now as a few years ago.

In the Atlanta market, both wall systems typically are installed by specialty contractors whom are accustomed to performing their own independent design/build engineering evaluation using available geotechnical subsurface data supplemented by their independent studies, if desired. In this report, we have suggested wall alignments and lengths based primarily on our site observations, not geotechnical or hydraulic aspects. The subsurface conditions appear relatively uniform in the areas explored. The actual alignment of the wall should be determined based on specific hydraulic modeling, understanding that a vertical wall face will be created not a trapezoidal shaped, sloped stream channel and other aspects such as regulatory limitations, property boundaries, historic streambank location, etc., that are not geotechnical parameters could also have some impact on the final design. If the wall alignment can be located at about one-half to two-thirds of the slope height, it is possible some of the area lost behind Building 1189 could be regained. The wall alignment behind Buildings 1809/1811 would essentially range from mid to top of slope of the current slope. Both wall systems considered will likely each require at least one row of earth/rock tie-backs that extend from the wall face into the restrained soil mass. These anchors will likely extend beneath the adjacent buildings in order to achieve their design capacity. The tie-back anchors placement and orientation will have to consider the location of the existing sanitary sewer line, unless the sewer is relocated and abandoned. The wall embedment at the stream should extend deep enough to protect the wall from scour which could result in loss of ground from behind the wall. The civil engineer should determine the depth of maximum scour. The wall height should be set at least the current 100 year flood elevation, if not a few feet above. Due to the fluctuating stream flows, we recommend the wall be designed for at-rest earth pressures and full hydrostatic loading. The anchor capacity with adequate safety factors should be determined based on actual pull out test. In order to provide scour protection for the H-pile wall system, we suggest consideration be given to using a short sheet pile section installed on the creek side of the H-piles that is incorporated into the shotcrete wall finish. In addition, since these are relatively short wall segments, it may be necessary to flare the ends and sufficiently bury them into the streambank to minimize the potential for scour at the ends of each wall segment. Additional erosion protection using riprap to prevent flows from exposing the ends of the walls may be required. Figures 1 and 2 reflect our suggested alignment for the restrained wall system. As we have stated previously, these suggested alignments are based on our observations at the site and do not take into account other environmental, regulatory or physical factors that may ultimately position the wall alignment. The illustrations show angled wall sections. In general, the final wall design should consist of radial sections that do not impede stream flow or create additional stream turbulence.

Within the Atlanta area, we are aware of three specialty contractors capable of designing and constructing either of the recommended tie-back wall systems. Others may be local or nearby that are equally qualified. These contractors are not listed in any order of preference (alphabetical).

1. ABE Enterprises, Inc. – Kennesaw
2. Hayward-Baker – Alpharetta
3. Schabel Foundation - Marietta

6.3 Other Design and Construction Considerations

6.3.1 Survey Monitoring of Adjacent Structures

During the course of this study we have recommended routine survey monitoring of the buildings (especially Building 1189) be performed to determine if the loss of ground is currently impacting the buildings and affecting the structural integrity of the shallow foundation supported buildings (suspected not verified). To our knowledge, we are unaware that movement of the buildings has been recorded or the property owners have noted any structural damage. Should movement of the buildings occur prior to beginning construction, underpinning/shoring of the impacted areas may be required in advance of the slope remediation.

As the design and construction move forward, survey monitoring should continue and will become more important to the contractor as construction begins. The recommended tie-back option will likely require earth/rock anchors to be installed adjacent and beneath the existing structures. As these anchors are tensioned to carry the restrained loads, some movement of the restrained mass and the buildings may occur as the loads are engaged. Accurate survey monitoring will allow the contractor to judge when the tensioning process should be terminated; however, some measurable movement should be anticipated. We suggest a pre-construction building survey be performed by an independent third party company who specializes in this type documentation to help safeguard the HOA and owners against unwarranted claim dispute.

Throughout the course of this study we have assumed these buildings are supported on shallow foundations. However, no documentation has been provided to verify the foundation support system. The design/build contractor needs to verify the foundation support conditions before beginning the wall construction.

6.3.2 Underpining Support of Building 1189

Prior to the issuance of this report, we have been in discussions about the need to partially underpin Building 1189. To date, we are unaware of any noticeable movement in Building 1189 has been recorded or reported. However, the current slope failure is encroaching close to this structure. As such, we are concerned that continued loss of soil, especially at Building 1189, could result in future damage to the structure. One way to further minimize the potential risk for building damage caused by loss of foundation support would be to underpin the portions of the buildings at risk using either a helical pier, pinpile or other deep foundation support alternative. To be proactive, prior to noticeable structural distress, may result in an unwarranted cost plus the placement of foundation elements that will likely encumber and interfere with the wall construction/tie-backs. To wait too long could result in additional building repairs. The need and timing for underpinning is not so much an engineering decision as it may be more that of risk assessment and insurance. Ultimately the need to underpin prior to the wall construction may be required. Underpinning of the structure should be performed if detectable movements be measured. The specialty contractors listed previously have the expertise to provide the building underpinning and can provide this service at the same time as the wall construction, if deemed necessary or if deteriorating site conditions dictate. If the wall can be properly installed prior to any future substantial loss of ground, underpinning may not be required.

6.3.3 Temporary/Permanent Easements

The recommendations presented involve various construction activities that will result in both temporary and permanent encroachment of equipment and structural wall components that may require special permits or easements. We anticipate the typical stream buffers will be encroached and we understand Kendall and Associates is addressing these environmental/regulatory issues with USACE. As these and other permitting processes are undertaken, inquiries about issues presented earlier in this report should be considered. PGC does not propose to have expertise in this area, but only offers this discussion for consideration by others involved in this project more knowledgeable in these areas. The following list describes items for consideration:

1. Typical stream buffers will be impacted at the suggested wall alignments.

2. The contractor will likely want to access the stream area for temporary diversion and other activities that could involve equipment and heavy mats for equipment support.

3. Tree Ordinances.

4. HOA/owner easements for installation of tie-backs beneath structures.

5. Marietta/Cobb County easements to install tie-backs across utility easements. We would suggest a camera survey of the sewer pipe prior to construction as part of the pre-construction activities.

7.0 QUALIFICATIONS OF RECOMMENDATIONS

This evaluation of the geotechnical aspects of the proposed design and construction has been based on our understanding of the project and the data obtained during this study. The general subsurface conditions used in our evaluation were based on interpolation of the subsurface data between the borings. Regardless of the thoroughness of a subsurface exploration, there is the possibility that conditions will differ between boring locations, that conditions are not as anticipated by the designers, or that the construction process has modified the soil conditions. Therefore, experienced soil engineers and technicians should evaluate earthwork and foundation construction to verify that the conditions anticipated in design actually exist. Otherwise, we assume no responsibility for construction compliance with the design concepts, specifications or recommendations.

The recommendations contained in this report have been developed on the basis of the previously described project characteristics and subsurface conditions. If project criteria change, we should be permitted to determine if the recommendations should be modified. The findings of such a review will be presented in a supplemental report. Even after completion of a subsurface study, the nature and extent of variation between borings may not become evident until the course of construction. If such variations then become evident, it will be necessary to reevaluate the recommendations of this report after on-site observations of the conditions.

These professional services have been performed, the findings derived, and recommendations prepared in accordance with generally accepted geotechnical engineering principles and practices. This warranty is in lieu of all warranties either expressed or implied. This company is not responsible for the conclusions, opinions or recommendations of others based on these data.

The scope of services does not include any environmental assessment for the presence or absence of hazardous or toxic materials in the soil, groundwater, or surface water within or beyond the site. Any statements in this report or on the test boring records regarding odors, staining of soils, or other unusual conditions observed are strictly for general information only.

APPENDIX

SOIL TEST BORING PROCEDURES (ASTM D-1586)

The soil test borings were advanced by twisting continuous auger flights into the ground. At selected intervals, soil samples were obtained by driving a standard 1.4 inch I.D., 2.0 inch O.D., split tube sampler into the ground. The sampler was initially seated six inches to penetrate any loose cuttings created in the boring process. The sampler is then driven an additional 12 inches by blows of a 140 pound "hammer" falling 30 inches. The number of blows required to drive the sampler the final foot is designated the Standard Penetration Resistance.

The samples recovered were sealed in glass jars and were transported to the office where they were classified by an engineer in general accordance with the Unified Soil Classification System (USCS).

CORRELATION OF STANDARD PENETRATION RESISTANCE WITH RELATIVE COMPACTNESS AND CONSISTENCY

Sand and Gravel

Standard Penetration Resistance Blows / Foot	Relative Compactness
0 - 4	Very Loose
5 - 10	Loose
11 - 30	Medium Dense
31 - 50	Dense
Over 50	Very Dense

Silt and Clay

Standard Penetration Resistance Blows / Foot	Relative Compactness
0 - 1	Very Soft
2 - 4	Soft
5 - 8	Firm
9 - 15	Stiff
16 - 30	Very Stiff
31 - 50	Hard
Over 50	Very Hard

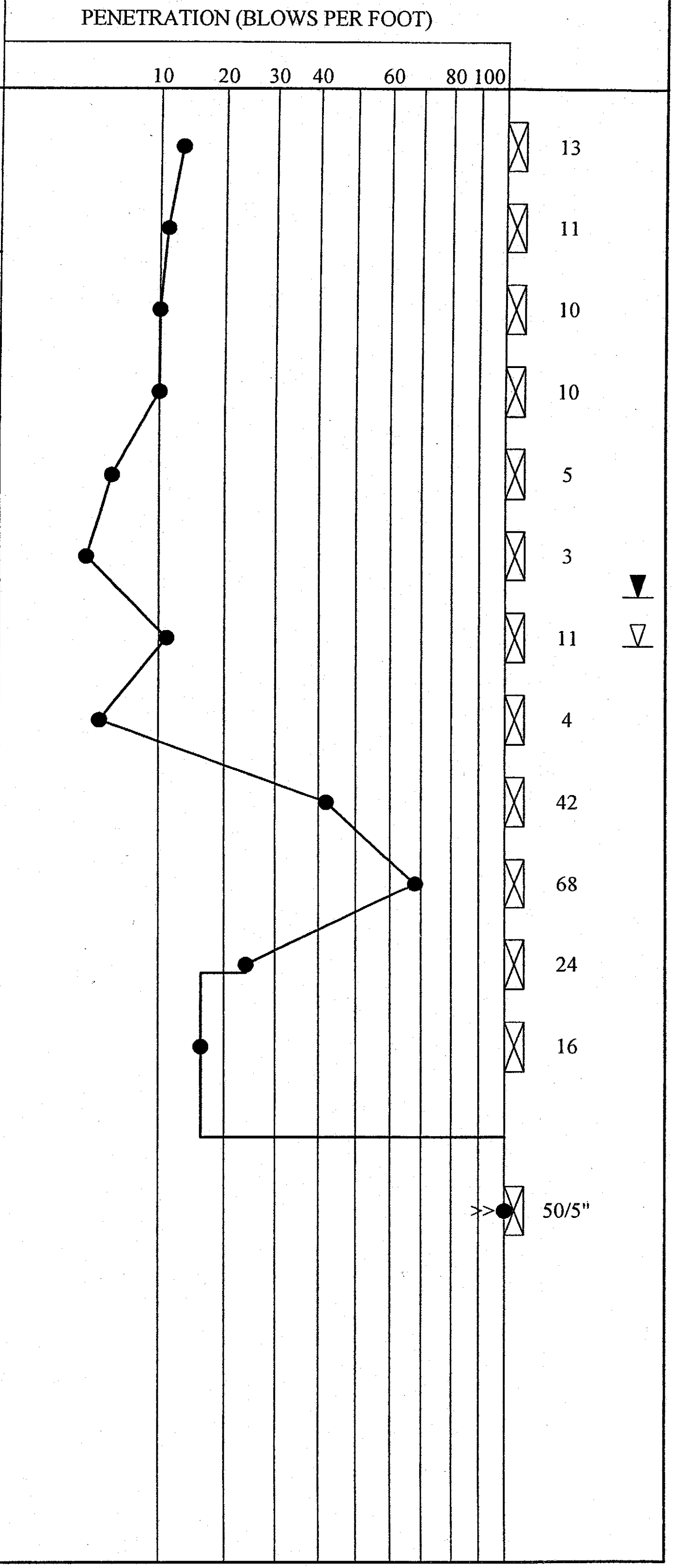
SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS		TYPICAL DESCRIPTIONS
			GRAPH	LETTER	
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)	●	GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
			○	GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)	○	GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
			▨	GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)	⁘	SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
			∴	SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF 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 SIZE	SILTS AND 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 AND CLAYS	LIQUID LIMIT GREATER THAN 50	▥	MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
			▨	CH	INORGANIC CLAYS OF HIGH PLASTICITY
			≋	OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
ALLUVIUM			⩛	PT	ALLUVIUM, PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS
FILL			▩	FILL	MATERIAL PLACED BY MAN

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

PIEDMONT
GEOTECHNICAL CONSULTANTS, INC. PGC

PROJECT: Ashborough Village Slope Evaluation



DEPTH (FT)	DESCRIPTION	ELEV. 947	PENETRATION (BLOWS PER FOOT)	
	FILL: Medium dense to loose red brown silty medium to fine SAND (SM), trace of clay			
			13	
5		942	11	
			10	
	ALLUVIUM: Stiff to firm brown clayey SILT (ML), organics			
10		937	10	
			5	
	Very loose gray silty medium to fine SAND (SM), very wet, trace of clay			
15		932	3	
				▼
	Medium dense gray silty medium to fine SAND (SM), very wet		11	▽
20	Very loose gray silty medium to fine SAND (SM), very wet	927	4	
	Dense to very dense tan brown silty medium to coarse SAND (SP), with gravel, very wet (SPT values may be amplified due to presence of gravel at sample interval.)		42	
25		922	68	
			24	
	RESIDUUM: Medium dense tan brown silty medium to fine SAND (SM)			
30		917	16	
	PARTIALLY WEATHERED ROCK: Sampled as very dense gray brown silty medium to fine SAND (SM)			
35		912	50/5"	
	Auger refusal at 35 feet.			
40				
45				

SOIL BORING RECORD

BORING NUMBER	B-1
DATE DRILLED	3/12/2009
PROJECT NUMBER	109027
PAGE	1 of 1

▽ Groundwater level at time of boring
▼ Groundwater level - 24 hrs
C Caved depth - 24 hrs
■ Undisturbed sample

SOIL BORING RECORD 109027 ASHBOROUGH VILLAGE.GPJ PIEDMONT GEO.GDT 5/7/09

PIEDMONT
GEOTECHNICAL CONSULTANTS, INC. PGC

PROJECT: Ashborough Village Slope Evaluation

DEPTH (FT)	ELEV.
	946
5	941
10	936
15	931
20	926
25	921
30	916
35	911
40	
45	

DESCRIPTION

FILL: Loose red brown silty medium to fine SAND (SM), trace of clay

Medium dense red brown silty medium to fine SAND (SM), trace of clay

ALLUVIUM: Medium dense to loose brown silty medium to fine SAND (SM)

Firm gray clayey SILT (ML), wet

Loose gray silty medium to fine SAND (SM), trace of clay, very wet

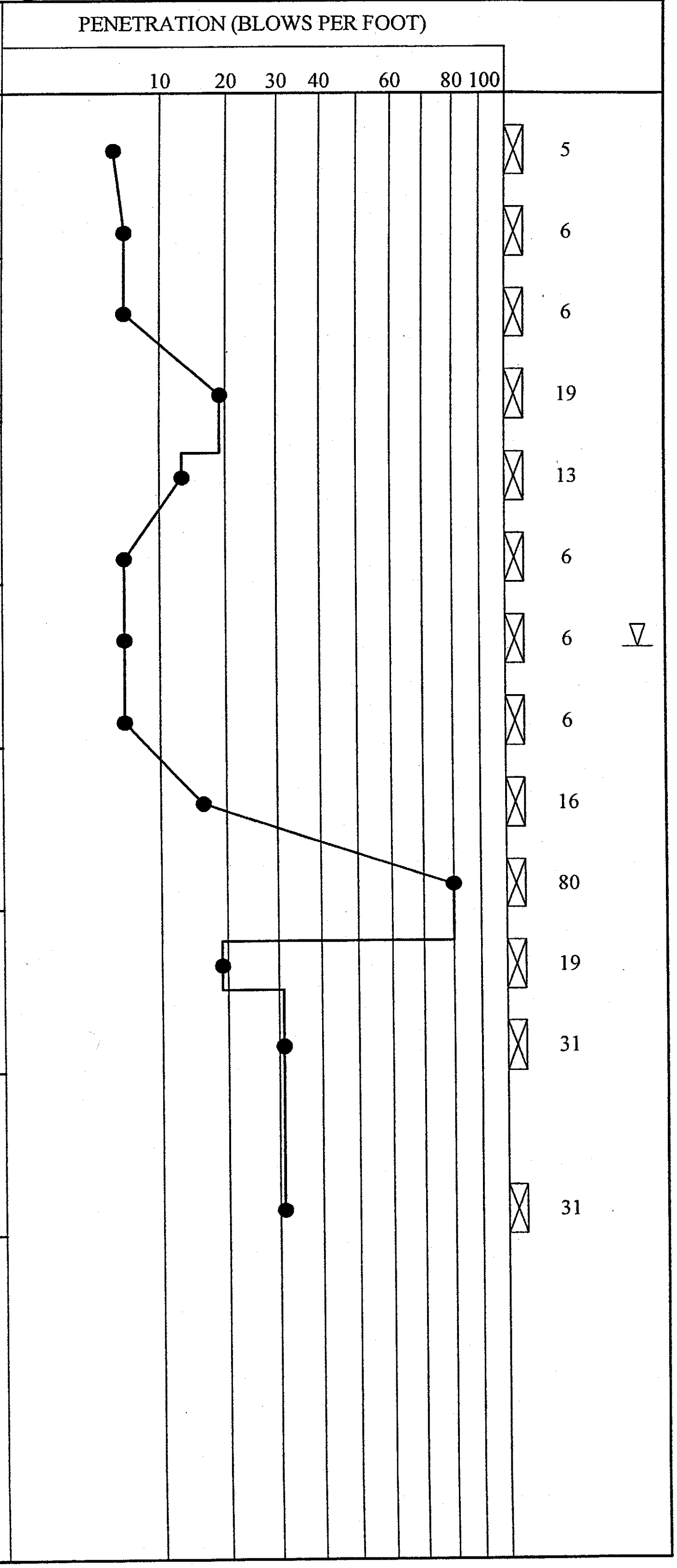
Medium dense gray brown silty medium to fine SAND (SM), trace of clay, very wet

Very dense tan gray silty medium to fine SAND (SP), gravel
(SPT value may be amplified due to presence of gravel at sample interval.)

No Recovery

RESIDUUM: Dense gray white silty medium to fine SAND (SM), micaceous

Boring terminated at 35 feet.



REMARKS: Undisturbed samples collected in offset boring B-2A from 15 to 17 feet and 17 to 19 feet.

SOIL BORING RECORD

BORING NUMBER	B-2
DATE DRILLED	3/13/2009
PROJECT NUMBER	109027
PAGE	1 of 1

∇ Groundwater level at time of boring

▼ Groundwater level - 24 hrs

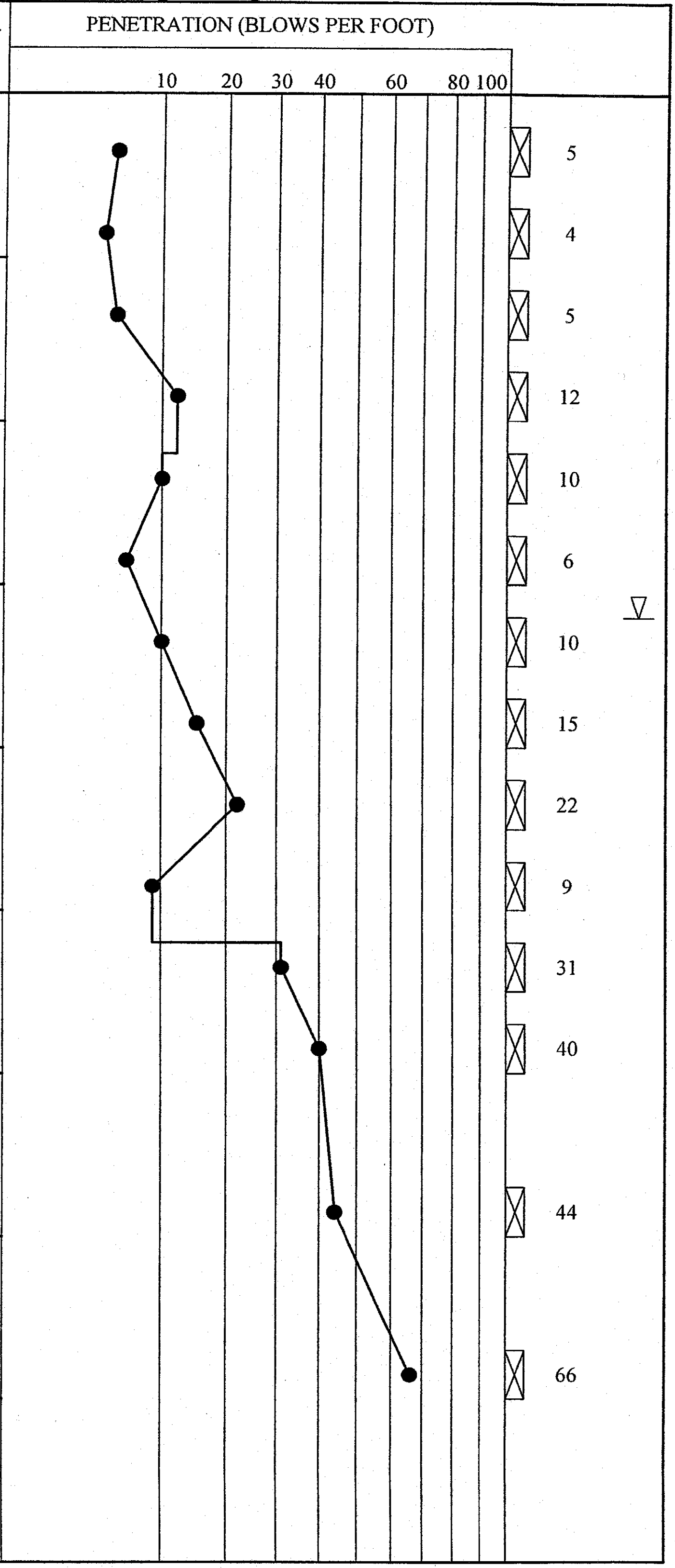
C Caved depth - 24 hrs

■ Undisturbed sample

PIEDMONT
GEOTECHNICAL CONSULTANTS, INC. PGC

PROJECT: Ashborough Village Slope Evaluation

DEPTH (FT)	DESCRIPTION	ELEV. 946
	FILL: Loose to very loose red brown silty medium to fine SAND (SM), trace of clay	
5		941
	Loose brown silty medium to fine SAND (SM), trace of clay	
10	Stiff red brown fine sandy SILT (ML), trace of clay	936
	ALLUVIUM: Stiff to firm brown clayey SILT (ML), wet	
15		931
	Loose gray silty coarse to fine SAND (SP), very wet	
20	Medium dense gray silty coarse to fine SAND (SP), very wet	926
	Loose tan brown silty coarse to fine SAND (SP), very wet, with gravel	
25		921
	RESIDUUM: Dense gray tan to gray white silty medium to fine SAND (SM)	
30		916
35		911
	Very dense tan gray silty medium to fine SAND (SM)	
40		906
	Boring terminated at 40 feet.	
45		



REMARKS: Undisturbed samples collected in offset boring B-3A from 6 to 8 feet and 11 to 13 feet.

SOIL BORING RECORD

BORING NUMBER B-3
DATE DRILLED 3/13/2009
PROJECT NUMBER 109027
PAGE 1 of 1

∇ Groundwater level at time of boring
▼ Groundwater level - 24 hrs
C Caved depth - 24 hrs
▮ Undisturbed sample

PIEDMONT
GEOTECHNICAL CONSULTANTS, INC.
PGC

PROJECT: Ashborough Village Slope Evaluation

DEPTH (FT)	ELEV.
	946
5	941
10	936
15	931
20	926
25	921
30	916
35	911
40	
45	

DESCRIPTION

FILL: Loose orange tan silty medium to fine SAND (SM)

Medium dense red brown silty medium to fine SAND (SM)

ALLUVIUM: Soft brown clayey SILT (ML), very wet

Medium dense brown gray silty coarse to medium SAND (SP), very wet

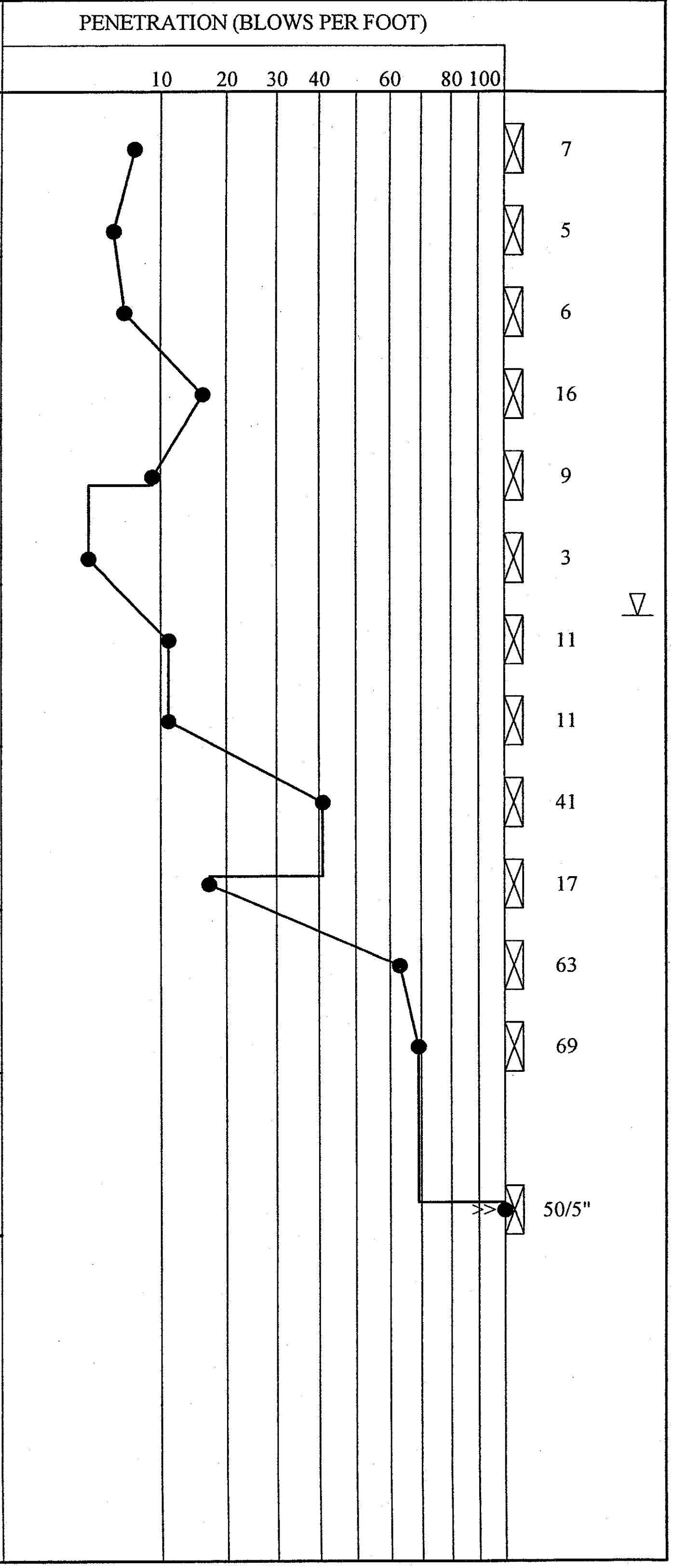
Very dense gray brown silty coarse to medium SAND (SP), very wet
(SPT value may be amplified due to presence of gravel at the sample interval.)

RESIDUUM: Medium dense gray white silty medium to fine SAND (SM)

Very dense gray white silty medium to fine SAND (SM)

PARTIALLY WEATHERED ROCK: Sampled as very dense tan brown silty medium to fine SAND (SM)

Auger refusal at 38 feet.



SOIL BORING RECORD

BORING NUMBER B-4
DATE DRILLED 3/12/2009
PROJECT NUMBER 109027
PAGE 1 of 1

▽ Groundwater level at time of boring
▼ Groundwater level - 24 hrs
C Caved depth - 24 hrs
■ Undisturbed sample

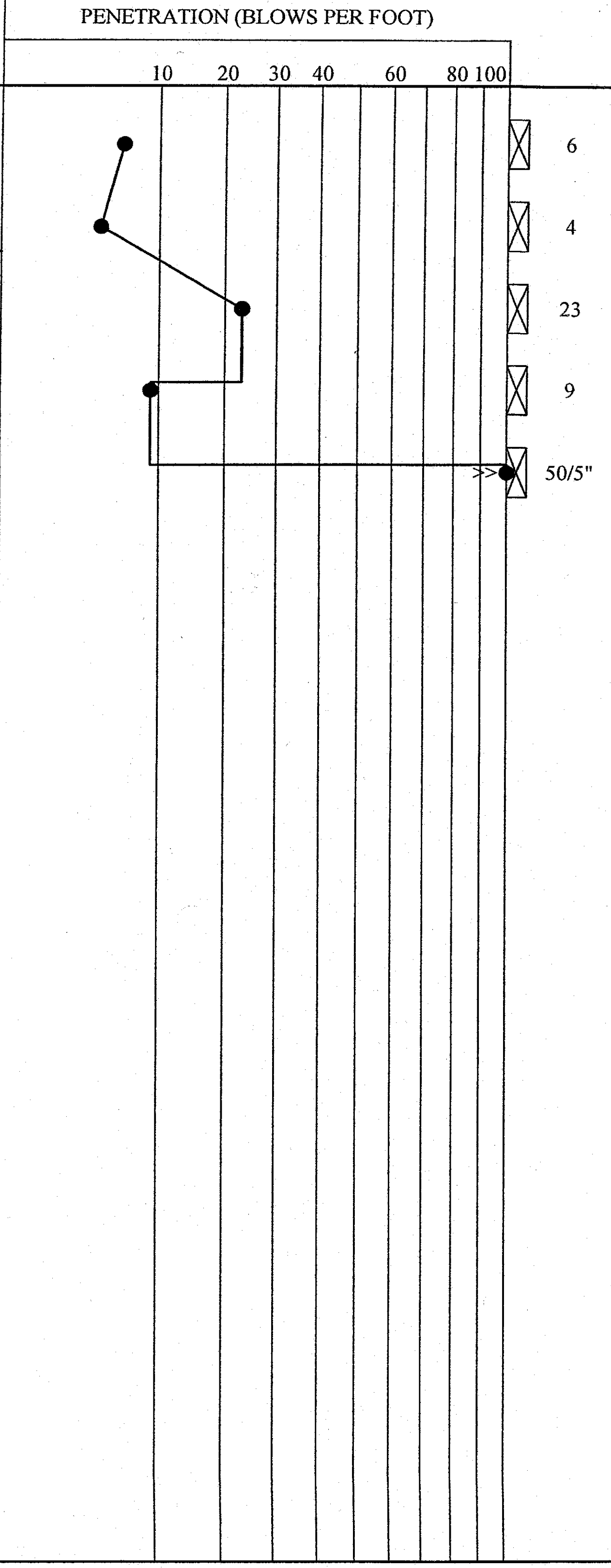
SOIL BORING RECORD 109027 ASHBOROUGH VILLAGE.GPJ PIEDMONT GEO.GDT 5/7/09

PIEDMONT
GEOTECHNICAL CONSULTANTS, INC. PGC

PROJECT: Ashborough Village Slope Evaluation

DEPTH (FT)	DESCRIPTION	ELEV. 948
	FILL: Loose red brown silty medium to fine SAND (SM), trace clay	
5		943
	Medium dense gray silty medium to fine SAND (SM), possible ground up rock/concrete	
10	ALLUVIUM: Loose red brown silty medium to fine SAND (SM), trace clay	938
	WOOD	
	Auger refusal at 13 feet.	
15	Note: Hit power line during drilling at about 6 feet.	
20		
25		
30		
35		
40		
45		

PENETRATION (BLOWS PER FOOT)



REMARKS: No groundwater encountered at time of boring.

SOIL BORING RECORD

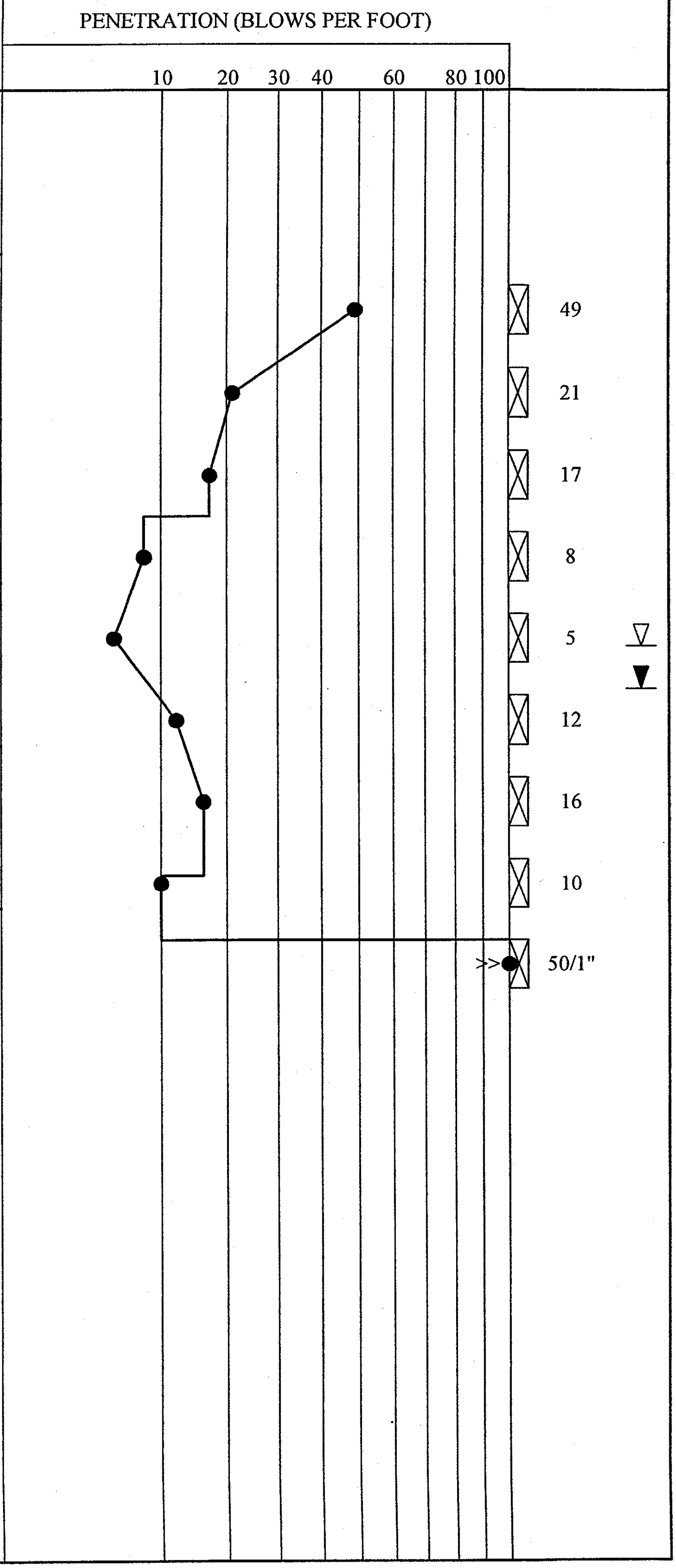
BORING NUMBER: B-5
DATE DRILLED: 3/11/2009
PROJECT NUMBER: 109027
PAGE: 1 of 1

▽ Groundwater level at time of boring
▼ Groundwater level - 24 hrs
C Caved depth - 24 hrs
■ Undisturbed sample

PIEDMONT
GEOTECHNICAL CONSULTANTS, INC. PGC

PROJECT: Ashborough Village Slope Evaluation

DEPTH (FT)	DESCRIPTION	ELEV. 949
	Auger Boring: No samples taken. (See Summary of Hand Auger Borings.)	
5		944
	FILL: Dense tan silty medium to fine SAND (SM), with rock/concrete pieces (SPT values may be amplified due to presence of rock/concrete pieces.)	
10	Medium dense brown to dark brown silty medium to fine SAND (SM), heavy organics	939
	ALLUVIUM: Firm brown clayey SILT (ML), very wet	
15		934
	Medium dense gray silty coarse to medium SAND (SM), very wet	
20		929
25	RESIDUUM: Loose tan brown silty medium to fine SAND (SM), very wet	924
	PARTIALLY WEATHERED ROCK: Sampled as very dense tan brown silty medium to fine SAND (SM) Auger refusal at 27 feet.	
30		
35		
40		
45		



SOIL BORING RECORD 109027 ASHBOROUGH VILLAGE.GPJ PIEDMONT GEO.GDT 5/7/09

SOIL BORING RECORD

BORING NUMBER B-6
DATE DRILLED 3/11/2009
PROJECT NUMBER 109027
PAGE 1 of 1

∇ Groundwater level at time of boring
▼ Groundwater level - 24 hrs
C Caved depth - 24 hrs
■ Undisturbed sample

SUMMARY OF HAND AUGER BORINGS

Ashborough Village Slope Evaluation
Marietta, Georgia
PGC Project No. 109027

Boring No.	Depth (ft.)	Description	Dynamic Cone Penetrometer	
			Depth (ft.)	n (bpi)
HA-1	0 – 1.5 1.5	ALLUVIUM: Tan brown silty coarse to medium SAND (SP), with gravel Hand auger boring refusal at 1.5 feet. Groundwater encountered at top of boring.	1	16
HA-1A	0 – 4 4	ALLUVIUM: Tan brown silty coarse to medium SAND (SP), with gravel Hand auger boring refusal at 4 feet. Groundwater encountered at top of boring. HA-1A offset 5 feet right of HA-1 (looking upstream) towards center of creek.	1 3 4	9 3 11
HA-2	0 – 3 3 – 6 6 – 7 7	ALLUVIUM: Tan brown silty coarse to medium SAND (SP), with organics Gray clayey medium to fine SAND (SC), with trace of organics; no organics below 5 feet Brown silty coarse to medium SAND (SM-SP), gravel Hand auger boring refusal at 7 feet. Groundwater encountered at top of boring.	1 3 5 7	7 5 10 20+
HA-3	0 – 5 5 – 8 8	ALLUVIUM: Tan brown silty coarse to medium SAND (SM), trace of organics Gray fine sandy SILT (ML) Hand auger boring refusal at 8 feet. Groundwater encountered at 6 inches.	1 3 5 7	2 4 10 11

Boring No.	Depth (ft.)	Description	Dynamic Cone Penetrometer	
			Depth (ft.)	n (bpi)
HA-4	0 – 6 6 – 10 10	FILL: Brown orange silty coarse to medium SAND (SM), clean ALLUVIUM: Gray tan medium to fine sandy SILT (ML), mottled, wet Hand auger boring terminated at 10 feet. No groundwater encounter at time of boring.	1 3 5 7 10	3 8 7 13 5
HA-5	0 – 10 10	FILL: Brown red medium to fine sandy SILT (ML), trace of clay Hand auger boring terminated at 10 feet. No groundwater encountered at time of boring.	1 3 5 7 10	6 3 3 3 3
HA-6	0 – 10 10	FILL: Brown red medium to fine sandy SILT (ML), trace of clay, wet at 5 feet to 10 feet, softer zone from 6 feet to 7 feet Hand auger boring terminated at 10 feet. No groundwater encountered at time of boring.	1 3 5 7 10	5 3 2 5 8
HA-7	0 – 3 3 – 4 4 – 7 7 – 9 9	ALLUVIUM: Tan silty coarse to medium SAND (SM), some organics Tan silty coarse SAND (SP), clean Gray silty medium to fine SAND (SM) Gray fine sandy SILT (ML) Hand auger boring refusal at 9 feet on gravel. Groundwater encountered at 8 inches.	1 3 5 7 9	3 5 11 2 20+

Boring No.	Depth (ft.)	Description	Dynamic Cone Penetrometer	
			Depth (ft.)	n (bpi)
HA-8	0 – 2	ALLUVIUM: Tan gray slightly clayey silty coarse to medium SAND (SM), organics	1 3 5	7 3 8
	2 – 4	Gray silty clayey medium to fine SAND (SC), many organics	7 9	8 2
	4 – 7	Gray fine sandy SILT (MH)	10	18
	7 – 10	Gray silty medium to fine SAND (SM), trace of clay		
	10	Hand auger boring refusal at 10 feet on gravel.		
		Groundwater encountered at 12 inches.		
HA-9	0 – 3	ALLUVIUM: Brown tan silty coarse to medium SAND (SP)	2 4	3 4
	3 – 6	Gray clayey medium to fine SAND (SC), some organics	6 8	4 2
	6 – 9	Gray silty medium to fine SAND (SM), trace of clay		
	9	Hand auger boring refusal at 9 feet.		
		Groundwater encountered at 6 inches.		
HA-10	0 – 3	ALLUVIUM: Tan brown silty coarse SAND (SP), with gravel	2 4	3 6
	3 – 7	Tan gray silty coarse to medium SAND (SM)	6	12
	7	Hand auger boring refusal at 7 feet		
		Groundwater encountered at 6 inches.		
HA-11	0 – 6	Fill: Red orange silty medium to fine SAND (SM)	1 3	8 7
	6	Hand auger boring terminated at 6 feet.		
		Boring B-6 continues. See Soil Boring Record B-6.		