GEOTECHNICAL SITE INVESTIGATION
510 KULI‘OU‘OU ROAD
TMK: 3-8-010:004

Project No. SRSS00112

March 20, 2012

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APRIL 30, 2012
EXPIRATION DATE OF THE LICENSE

THIS WORK WAS PREPARED BY ME
OR UNDER MY SUPERVISION AND
CONSTRUCTION OF THIS PROJECT
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TABLE OF CONTENTS

INTRODUCTION

This report presents the results of a geotechnical field investigation for the lower portion of the property at 510 Kuliouou Road (Figure 1). A previous study by Applied Geosciences, “Slope Hazards Investigation, 510 Kuliouou Road, TMK: 3-8-010:004 (March 19, 2012)”, addresses slope instability processes for the entire property related to landslide, creep and rockfall hazards. That report concluded that the lower portion of the lot, which is relatively flat and is located outside the limits of previous sliding areas, is suitable for residential construction. This study presents the results from additional field studies, laboratory testing and recommendations for the specific purpose of constructing single or double-story residential housing and associated infrastructure in Area A. This portion of the lot is located between street entrance and the lower limit of the slide area. The edge of the slide was marked in the field and is shown in Figure 2, along with the location of borings and test pits. For added precaution, the slope hazards investigation recommended that a 10-foot setback should be observed extending downslope from the edge of the area where previous sliding may have occurred. This setback applies to the construction of principal residential structures and pools, but not to yard areas, retaining walls not exceeding 6 feet in height, driveways, walkways, sheds, or any other non-living secondary structures.

The proposed development within Area A includes three units with one dwelling per unit, a paved driveway along the right side of the property, and water and sewer utilities. These are shown in drawings prepared by Bow Engineering & Development, Inc. (December 22, 2011), along with grading plans and other construction details.

The purpose of this report is to characterize surface and subsurface soil conditions and to present a set of geotechnical engineering recommendations for the purpose of constructing the envisioned structures. Drilling and sampling were conducted on October 29 and November 10, 2011, followed by laboratory testing and analysis. The findings and recommendations presented herein are subject to the limitations noted at the end of this report.

SCOPE OF WORK

Work carried out as part of this project consisted of:

- A review of available soil and geologic data related to the project site
- Coordination of field work with the drilling subcontractor
- Drilling and sampling of four borings and excavation of one test pit
- Performing a field reconnaissance to identify and characterize surface features
- Field sampling and laboratory testing of selected specimens to assist with classification and characterization of engineering properties
- Analysis of field and laboratory results to formulate a set of geotechnical recommendations
- Preparation of this report summarizing our work

The boring and test pit logs are presented in Appendix A. Specific results from the laboratory testing program are included in Appendix B.

GEOLOGIC SETTING

Kuliʻouʻou Valley is one of several deeply eroded valleys that cut into the Koolau mountain range in East Honolulu. It was formed over the last 1.8 to 2.6 million years, after the main stage of volcanic growth ceased on Oahu. The ridges that flank the valley, and indeed most of the Koolau volcanic edifice, are composed of layer upon layer of pahoehoe and aʻa lava flows dipping at about 20 degrees toward the ocean. Fractured and weathered basalt rock and layers of clinker rubble of variable cementation characterize the exposed face of the steep hillsides. Soil cover in the Kuliʻouʻou-Papahehi area disappears almost entirely above elevation 160 feet. Numerous outcrops of rocks and boulders are visible on the ridge that ascends above this elevation.

A gentler talus extends between elevations of 160 feet and 30 feet. The landslide and the lot at 510 Kuliʻouʻou Road are located on this apron. The talus merges with a similar fan that extends down from a side canyon located northwest of the property, which is aligned roughly parallel to Kuliʻouʻou Valley (see Figure 1). Whereas slopes on the flank of the ridge are typically in the 30^o to 60^o range, and sometimes steeper, they decrease to about 9^o, on average, on the talus apron. Soil materials are distinctly different from those found on the ridge and the valley floor. They are composed primarily of colluvium and some alluvium, which originates from higher elevations on the ridge. These materials have accumulated above the original basalt basement and have continued to weather with time. The colluvium is comprised of numerous boulders and cobbles, with sizes up to several feet in diameter, within a matrix of dark gray, high-plasticity sandy clay (MH/CH in the Unified Soil Classification System). Surface soils on the talus belong to the Lualualei soil series (Soil Conservation Service, 1972). In general, these soils are expansive and susceptible to strength loss and creep. The colluvium transitions to weathered bedrock at depths that increase from zero at elevation 160 feet to several tens of feet near the valley floor. The transition from apron deposits to bedrock is gradual, which indicates significant weathering of the parent lava flows that make up the bedrock.

SURFACE CONDITIONS

Although currently there are no dwellings on the property, there did at one time exist a single-story wood-frame structure and storage shed adjacent to Kuliouou Road. Reportedly, it was torn down a number of years ago due to its age. Remains such as concrete rubble, fill, and other construction material are still found scattered across the lower portion of the lot. In addition, there are a number of low rock walls and piles of large boulders at various locations. Vegetation has taken over in many places, consisting of a few large trees, shrubbery and overgrown grass.

The natural surface soil on the lower portion of the property consists of Lualualei stoney clay. This soil developed in alluvium and colluvium deposits and is variously weathered and relatively permeable. The fine-grained matrix of this soil is very dark brown to grayish and can be of high plasticity. As a result, it cracks easily upon drying. The larger fractions in the soil vary from sand to very large boulders, some several feet in size. Very large surface boulders are found pervasively where the lot begins to slope upward, but they appear to be less common in the relatively flat lower portion of the property. Nonetheless, cobbles and smaller boulders are widespread only a few inches below the ground surface, even in the flat area near the entrance.

The flank of the valley that ascends from the rear of the property consists of rock outcrops and very shallow soils.

SUBSURFACE CONDITIONS

The distribution and character of subsurface soils were determined from four borings, one test pit, a review of older boring drilled within the footprint of the property, soil maps, and geologic considerations for similar talus deposits elsewhere in East Honolulu. The location of the borings and test pit are shown in Figure 2. In addition, laboratory tests were conducted on selected field samples and the results are included in Appendix B.

The subsurface soils consist largely of colluvium material that has descended from the adjacent ridge, and to a much lesser extent of alluvium. Some of this colluvium has weathered in place, displaying the typical mottling from chemical and biological processes. The grain size of this material varies greatly from boulders several feet in size to very fine clay. The precise grain size distribution does change with location and depth. For example, within the upper few feet of test pit TP-1 the soil consists of a mix of highly plastic and stiff silt to clay, with cobbles and boulders occupying anywhere between 10% and 40% or more of the soil volume. The amount of sand and gravel also changes drastically within a few inches within the depth profile.

Plastic limits and swell pressures from consolidation tests on the fine-grained fraction of the soils are not very high. This can be misleading for shallow specimens since they have undergone multiple wetting and drying cycles in the field and therefore may not reflect their true swell-shrink potential. Lualualei soils are known to be highly expansive and the recommendations contained in this report account for this possibility. Pervasive cracking of the surface soils was observed to be extensive and its effects are accounted for in the recommendations.

No water table was encountered in any of the borings and none was expected. The talus deposit is considered quite permeable due to the nature of the geologic materials. Prior studies have indicated that transient perched water tables of limited extent may form in rare cases as a result of prolonged and intensive rainfall. These water accumulations are difficult to predict but are thought to dissipate rather quickly once rainfall ceases.

The frequency and distribution of large boulders in the subsurface is variable throughout the property and therefore excavation may be difficult and require the use of specialized equipment. Some of the largest boulders may have to be broken up before they can be removed.

GEOLOGIC HAZARDS

As already mentioned, an earlier study was conducted to assess geologic hazards associated with sliding, rockfall and creep processes. Here we only provide a brief synopsis of the results. The corresponding report should be consulted for further details regarding geologic hazards.

Slope Stability

The Kuliouou landslide, which crosses the middle section of the property, was last reactivated as a result of the 1987-1988 New Year's Eve storm. The lower portion of the property, i.e. Area A, was not affected by sliding or creep. The first 100 feet of the property, from street entrance to the rock wall perpendicular to the length of the lot, is virtually flat. The next 120 feet, from the rock wall to the lower boundary of the slide area (where Area A ends), has an average slope of 6.7H:1V. In general, colluvium slopes in East Honolulu sharing the general morphological soil characteristics found at the property are considered safe with regard to sliding as long as they do not exceed an inclination of 6H:1V. Detailed slope stability calculations carried out as part of the slope hazards investigation also concluded that slope instability is not a concern for Area A. Calculated safety factors were well in excess of 1.50.

A 10-foot setback is recommended, as described earlier, as an added precaution in the very unlikely event that large-scale sliding of the upper portion of the property was to be reactivated at a future time. Maximum downslope displacements during the 1987-1988 event were on the order of about 4 to 5 feet and therefore a 10-foot buffer zone is deemed adequate.

On the other hand, any future grading operations need to be evaluated carefully to insure that they do not pose detrimental effects with regard to slope stability and creep. Specific recommendations are provided below.

Creep

Creep deformations have occurred and may continue to take place within the confines of the landslide, particularly where the slope exceeds an inclination of 6H:1V. This is the case for Areas B and C. Creep is not a concern for most of Area A, except to a minimal extent for the portion of the lot immediately downhill of the slide boundary where the slope approaches (but is less than) 6H:1V. Due to the plastic and unsaturated nature of the fine-grained soils, creep is considered a minor issue that may affect isolated footings and slabs placed at or near ground surface. Recommendations in this report that address shrinking and swelling, such as placement of non-plastic select fill material beneath foundations and slabs, are also considered an effective means to prevent any potential creep problems.

Rockfall Hazard

A detailed rockfall hazard assessment, conducted as part of the slope hazards investigation, concluded that while the risk of individual boulders descending from the steep slope rising behind the property is not negligible, such boulders are not expected to reach Area A and would come to rest well above it. Thus while a rockfall fence may be required if and when the upper portions of the property are developed, it is not necessary for Area A.

SEISMIC CONSIDERATIONS

Based on the 2003 International Building Code (current code adopted by the City and County of Honolulu), the subsurface soil conditions at the property correspond most closely to site class D. For this type of soil profile, and given the location of the property, the maximum considered earthquake spectral response acceleration for short periods, S_{MS}, is estimated to be 80% of gravity, while the maximum considered earthquake spectral response acceleration for 1-second period, S_{m1}, is estimated to be 37% of gravity. The corresponding design spectral parameters can be taken as 2/3 of these values.

There are no soils present that are susceptible to liquefaction and associated strength loss. Also, there are no known faults nearby that could cause surface ruptures. On the other hand, minor to modest inertial shaking is possible, as evidenced by the 2006 earthquake that was felt throughout Oahu. Peak ground accelerations measured in Honolulu in connection with that earthquake were less than 8% of gravity, although there are reports of stronger shaking in certain alluvial clay deposits and other soils that may well have exceeded the recorded values. Stronger surface ground motions at these locations were probably the result of site-specific ground amplification effects.

RECOMMENDATIONS

Site Preparation and Grading

At the beginning of earthwork, the entire area from street level to the upper boundary of the lower lot (coinciding with the lower limit of the slide area) should be thoroughly cleared and grubbed of trees and their roots, all other vegetation, construction debris, rubbish, hard clay lumps or boulders exceeding four inches in largest dimension, and any other unsuitable materials. Large boulders that may be used for the construction of retaining walls or other structures may be separated and stored onsite for later use. Existing utilities should be located and shut off prior to grading operations. If existing utilities are to be abandoned, they should be removed, and the resulting excavation properly backfilled with select fill material and compacted to at least 90% relative compaction in accordance with ASTM Designation D 1557.

Excavations at the proposed building site will encounter numerous boulders, some as large as several feet in diameter, that will require specialized excavation equipment for their removal. The largest of these boulders may have to be broken up before they can be removed. The large pile of boulders about 140 feet from street entrance will have to be removed during the early phases of site preparation.

Excavation (and filling) will be required beneath all new structural areas to avoid the detrimental effects of shrinking and swelling associated with the highly plastic onsite soils. Appropriate excavation depths are listed in the following sections for shallow foundations, slabs, pavements, retaining structures and utility trenches. Structural areas are defined as locations encompassed within the final outermost perimeter of all new buildings, plus 7 feet beyond such a perimeter. Before filling operations begin, the excavated grade should be scarified to a minimum depth of 8 inches, moisture conditioned to between 2% and 4% above optimum moisture, and compacted to not less than 90% relative compaction, in accordance with ASTM Designation D 1557. If soft areas are encountered at the bottom of excavations, these areas should be over-excavated to firm material and the depression filled with properly compacted select fill material.

The onsite soils are unsuitable as fill or backfill materials. Select fill material should be used instead, consisting of non-expansive select granular soil of basaltic origin. It should be well graded from gravel to fines, with no particles larger than 3 inches in largest dimension and between 10 and 30 percent passing the No. 200 sieve. The plasticity index should not exceed 15. Fill materials should be free of vegetation, deleterious materials and clay lumps. Potential fill soils should be tested for conformance with these recommendations and approved by the project geotechnical engineer prior to delivery to the site.

Fill and backfill materials should be placed in level lifts not exceeding 8 inches in loose thickness, moisture-conditioned to above the optimum moisture, and compacted to at least 90 percent relative compaction in accordance with ASTM Designation D 1557. The compaction requirement should be increased to 95 percent relative compaction for fills placed within 5 lateral feet and 12 inches beneath any slab, foundation, pavement or walkway. Filling operations should start at the lowest point and continue up in level horizontal compacted layers in accordance with the above fill placement recommendations.

In order to avoid flooding, the final fill grade within all structural areas should be a minimum of 12 inches above the adjacent grade, and preferably more. Drainage swales, French drains, or other drainage provisions should be incorporated in the design so that the final grade does not pond excess water from surface runoff and does not direct such runoff toward structural areas. Drainage provisions are particularly important uphill of the last dwelling and adjacent to the limits of the slide zone. The setback area would be a suitable area for this purpose.

Existing slopes should not be steepened permanently by unsupported cuts into the existing hillside. If such slopes are contemplated, further input should be sought from Applied Geosciences to evaluate their safety and to provide recommendations on suitable retention systems.

Fill slopes can be considered if select fill material, having the characteristics described above and compacted to a minimum of 95% relative compaction in accordance with ASTM Designation D 1557, is used exclusively for fill earthwork. If fills are to be used in the steeper portion of Area A, i.e. starting at 100 feet inward from street access, the natural slope should be benched prior to filling, as indicated in Figure 3.

Any earthwork plans that change the existing grade, and that involve more than 30 cubic yards of material, should be reviewed prior to construction to evaluate the impact on slope stability.

Wherever compaction of soils is stipulated, field density tests should be performed to confirm the compaction requirements. All earthwork operations should be observed and the soils be tested by the project geotechnical engineer or his representative. The further recommendations in this report are contingent upon adherence to this and previous recommendations.

Foundations

The recommendations herein assume one or more single or double-story dwellings in Area A of the property. While the lower-most structure will be located on the nearly flat lower portion of the lot, those further away from street entrance will have to contend with a modestly steeper grade and the presence of numerous large boulders. Based on the subsurface conditions encountered at the site, we recommend that proposed homes and detached structures such as garages be supported by embedded continuous and isolated reinforced concrete footings.

Due to the expansive nature of the soils, the bottom of the reinforced concrete footings should be embedded at least 30 inches below the lowest adjacent exterior grade. Footing excavations should be over-excavated at least 8 inches below the 30 inches, compacted and leveled, and then filled back with 8 inches of select fill material compacted to 95 percent relative compaction in accordance with ASTM Designation D 1557, before placing of the reinforced concrete. If the bottom of footing excavations reveals unsuitable soils, excavation should be continued until proper bearing conditions are encountered. These recommendations are also considered an effective preventive measure to minimize creep effects, if any, in the steeper portions of Area A.

Continuous foundations should have a minimum width of 16 inches and single footings should be at least 30 inches in width or diameter.

The bottom surface of footings should have a slope not exceeding 10 percent, and preferably be level. Footings can be stepped where it is necessary to change the elevation of the top surface of the footing or where the surface of the ground slopes more than 10 percent.

The minimum lateral clearance between the outermost edge of any foundation and the nearest face of any slope exceeding 100 percent should be at least 6 feet.

An allowable bearing pressure of up to 1,800 pounds per square foot may be used for the design of spread footings for which the subgrade has been prepared in this manner and having the minimum dimensions provided. This bearing value is for dead plus live loads and may be increased by one-third for transient loads, such as those caused by wind or seismic forces.

If piers, mircopiles, helical piles or other types of deep foundations are chosen, additional input should be sought from Applied Geosciences to evaluate their effectiveness and safety.

Slabs on Grade

Due to the high swelling nature of the onsite plastic soils, living area and garage concrete slab floors should be constructed on top of a minimum of 24 inches of properly placed and compacted imported fill materials, as follows. The bottom 22 inches should consist of select fill material, as described earlier, compacted in loose lifts not exceeding 8 inches each to at least 90 percent relative compaction, in accordance with ASTM Designation D 1557. The compaction requirement should be raised to 95% within the uppermost 6 inches of the select fill. The next 4 inches should consist of damp, clean sand to act as a granular cushion. A moisture barrier immediately beneath the concrete slab should be added for protection from moisture damage and may be included as part of the 24-inch thickness. Termite protection barriers can also be incorporated into the design.

In order to minimize the detrimental effects of plastic soils where slopes exceed 10 percent, structural areas extending 7 feet beyond the outer limits of footings and slabs should incorporate a similar subgrade design, including all exterior walkways, lanais and other flatworks.

The minimum concrete thickness for interior and garage slabs should be 6 inches and sufficient reinforcement should be used throughout to prevent structural distress from uplift forces. Exterior flatworks, except for driveways and garage pads, can be 4 inches thick if sufficiently reinforced and underlain by properly compacted fill material as described above. To reduce the potential for shrinkage cracks in the walkway slabs, control joints should be provided at intervals equal to the width of the walkways with expansion joints at right-angle intersections.

Retaining Structures

The following recommendations are offered for the design of low retaining structures. If the height of any retaining structure is to exceed 6 feet, or if retaining structures are being contemplated to replace lateral support from cuts made into the existing hillside, additional input should sought from Applied Geosciences. Lateral forces in the latter case may be larger than those recommended below.

The footing of any retaining structure should be embedded a minimum of 30 inches below the lowest adjacent grade and they should otherwise adhere to the same recommendations as for spread footings described above. Allowable footing bearing pressures are 1,800 pounds per square foot. Lateral loads may be resisted by frictional resistance developed between the bottom of the wall footing and the bearing soil, and by passive earth pressure acting against the vertical face passing through toe of the wall footing. A coefficient of friction of 0.25 may be used for concrete footings in contact with the bearing soil. Resistance due to passive earth pressure may be estimated using an equivalent fluid pressure of 200 pounds per square foot per foot of depth assuming that the soils around the footings are well compacted. The passive resistance in the upper 12 inches of the soil should be neglected.

In general, retaining structures should be designed to resist lateral earth pressures due to the adjacent soils and surcharge effects. The on-site soils are not suitable as backfill material within the zone defined by the back of the wall and a 2 horizontal to 1 vertical plane projected upwards from the bottom of the wall footing. It is assumed that the backfill material within this zone will have the characteristics of the select fill described earlier and that it will be compacted to 90 percent relative compaction in accordance with ASTM Designation D 1557. Care should be taken not to over-compact the backfill. Recommended equivalent lateral earth pressures for design of earth retaining structures are as follows:

	Level Backfill		Maximum Backfill Slope 2H:1V	
	Horizontal	Vertical	Horizontal	Vertical
Active	45 pcf*	0	65 pcf	35 pcf
At-Rest	60 pcf	0	80 pcf	45 pcf

*For soils with Atterberg limits below the A-line and plasticity index not exceeding 12, a value of 40 pcf may be used.

These lateral earth pressures do not include hydrostatic pressures that may be caused by trapped groundwater. Retaining walls that are not free to deflect laterally should be designed for the at-rest condition.

Surcharge stresses due to areal surcharges, line loads, and point loads, within a horizontal distance equal to the overall height of the adjacent portion of any wall, should be considered in the design. Corresponding lateral surcharge soil pressures should be selected in consultation with Applied Geosciences.

In general, retaining walls should be well drained to reduce the build-up of hydrostatic pressures. Either granular material or a prefabricated drainage product should be used in the back of every retaining wall, in conjunction with a perforated collector pipe along the bottom and regularly spaced weep holes. If granular material is to be used as the means of draining the backfill, it should consist of #3 Fine aggregate extending back a minimum of 12 inches from the rear of the wall. This drainage aggregate should be separated from other soils by a properly selected geotextile to provide adequate separation and cross-plane drainage functions. The collector pipe at the bottom of the drainage aggregate should consist of a perforated pipe with a minimum diameter of 4 inches and should be inclined to drain by gravity to an appropriate discharge location. Weep holes in the retaining wall should be at least four inches in diameter and should be spaced no more than 4 feet apart and no more than 8 inches above ground. Overall filtration and drainage performance of the drainage system should be evaluated during the design stage.

Pavements

We envision that either flexible asphalt or concrete pavements will be used for the driveways and parking areas of the proposed project. It is anticipated that the vehicle loading for these pavements will consist of passenger vehicles and light pick-up trucks. Based on the onsite soils, it is recommended that the following pavement sections be used for preliminary design purposes:

Flexible Pavements
4 inches of asphalt concrete (AC)
12 inches of aggregate base course

Concrete Pavements
6 inches of Portland cement concrete (PCC)
12 inches of aggregate base course

The pavement subgrade soils should be scarified to a depth of 8 inches, moisture-conditioned to above the optimum moisture content, and compacted to a minimum of 95 percent relative compaction in accordance with ASTM Designation D 1557. If loose soils and/or soft soils are encountered at the pavement subgrade elevation, we recommend that they be over-excavated and replaced with well-compacted select fill material.

The aggregate base course materials should meet the requirements of Subsection 703.06 of the State of Hawaii Standard Specifications for Road and Bridge Construction (2005), with a maximum 1.5-inch nominal size. The aggregate base course should be moisture-conditioned to above the optimum moisture content, placed in level lifts not exceeding 8 inches in loose thickness, and compacted to at least 95 percent relative compaction in accordance with ASTM Designation D 1557.

If PCC is selected, it should be properly reinforced. Paved areas should be sloped, and drainage gradients should be maintained to avoid water ponding. Water draining off pavements should not be allowed to infiltrate into the subgrade. Instead, it should be collected and discharged appropriately as discussed below.

Underground Utility Trenches

It is anticipated that most utility lines and connections will be laid in relatively shallow trenches that are at most 5 feet deep. As an alternative, some of the utilities could be placed above ground to avoid detrimental effects due to the plastic nature of the fine grained soils. This might be an effective alternative in the slightly steeper portion of Area A, immediately downhill of Area C. If utilities and/or utility connections are to be placed in shallow trenches, the following recommendations should be followed.

Granular bedding consisting of 6 inches of free-draining granular materials (ASTM C 33, No. 67 gradation) should be provided under the pipes for uniform support. Where soft and/or loose soils are encountered at or near the invert of the pipes, a stabilization layer consisting of an additional 18 to 24 inches of open-graded gravel wrapped in a non-woven filter fabric designed for separation function should be provided below the bedding layer for uniform support.

Once the pipes are installed, an additional 6 inches of free-draining granular material (ASTM C 33, No. 67 gradation) should be placed around and above the pipes for adequate lateral support. The upper portion of the trench can be backfilled with select fill material. The backfill should be moisture-conditioned to above the optimum moisture content, placed in level lifts not exceeding 8 inches in loose thickness, and compacted to at least 90 percent relative compaction in accordance with ASTM Designation D 1557. Where trenches are below pavement areas, the upper 3 feet of the trench backfill below the pavement grade should be compacted to a minimum of 95% relative compaction.

Drainage and Erosion Control

The potential for creep and volume changes of surface soils is best addressed by prudent drainage provisions that seek to minimize the infiltration of water into the subsurface and thereby attempt to stabilize the moisture content in the vadose zone, especially beneath and adjacent to structural areas. Much of this is accomplished by recommendations above regarding the replacement of unsuitable soils beneath footings, slabs, retaining walls and exterior flatworks.

It is possible that surface runoff and subsurface seepage through an extensive network of cracks descending from uphill areas of the property might have negative repercussions during intense or prolonged rainfall events. Therefore it is recommended to install a drainage system consisting of a lined surface collection ditch, along with a subdrain, both located within the setback area at the base of the slide limit and stretching between the lateral boundaries of the property. The subdrain should be at least 8 feet deep and 18 inches wide. The excavation should be lined and enclosed on top with a suitable geotextile selected for appropriate filtration and separation functions. A 4-inch perforated PVC collector pipe should be placed along the centerline of the trench, no more than 2 inches above the bottom of the excavation. The collector pipe should be sloped to drain by gravity to an appropriate discharge point. Free draining granular material (ASTM C 33, No. 67 gradation) should be used to backfill the trench.

The surface 8 inches of onsite soils in non-structural areas should be screened to take out particles larger than 3 inches and rolled back in place with a modest level of compaction. Imported topsoil suitable for planting should then be spread over the surface and all areas covered with grass or other plantings.

Finished grades adjacent to any structure or foundation should be sloped to carry water away from them. Gutter systems should be installed on all roofs and the discharge diverted away from the perimeter of the foundations and the back of retaining walls. Discharge from roofs, pavements and flatwork areas should be collected and directed through a closed piping system to discharge into appropriate collection systems that prevent water from infiltrating into the subsurface anywhere on the property. This may require the installation of ditches, drainage lines and French drains.

All exposed surfaces should be protected from erosion by appropriate means during and after construction. Foundation excavations should be properly backfilled against the walls or slab edges immediately after setting of the concrete to reduce potential excessive water infiltration into the subsurface.

Planting and irrigation systems, as well as other long-term erosion control measures, should be implemented as soon as finished grades have been completed. Excessive landscape watering near foundations, retaining walls and slopes should be avoided. Planters within 3 feet of foundations or retaining walls should be avoided as well, or they should have concrete bottoms and drains to reduce the potential for excessive water infiltration into the subsurface. Trees and hedges with large roots should not be planted in the back of retaining walls since they can cause distress to the walls.

Design Review

Preliminary and final drawings and specifications for the project should be forwarded to Applied Geosciences for review and written comments prior to solicitation for construction bids. This review is necessary to evaluate conformance of the plans and specifications with the intent of the geotechnical engineering recommendations provided herein. If this review is not made, Applied Geosciences cannot be responsible for misinterpretation of our recommendations.

LIMITATIONS

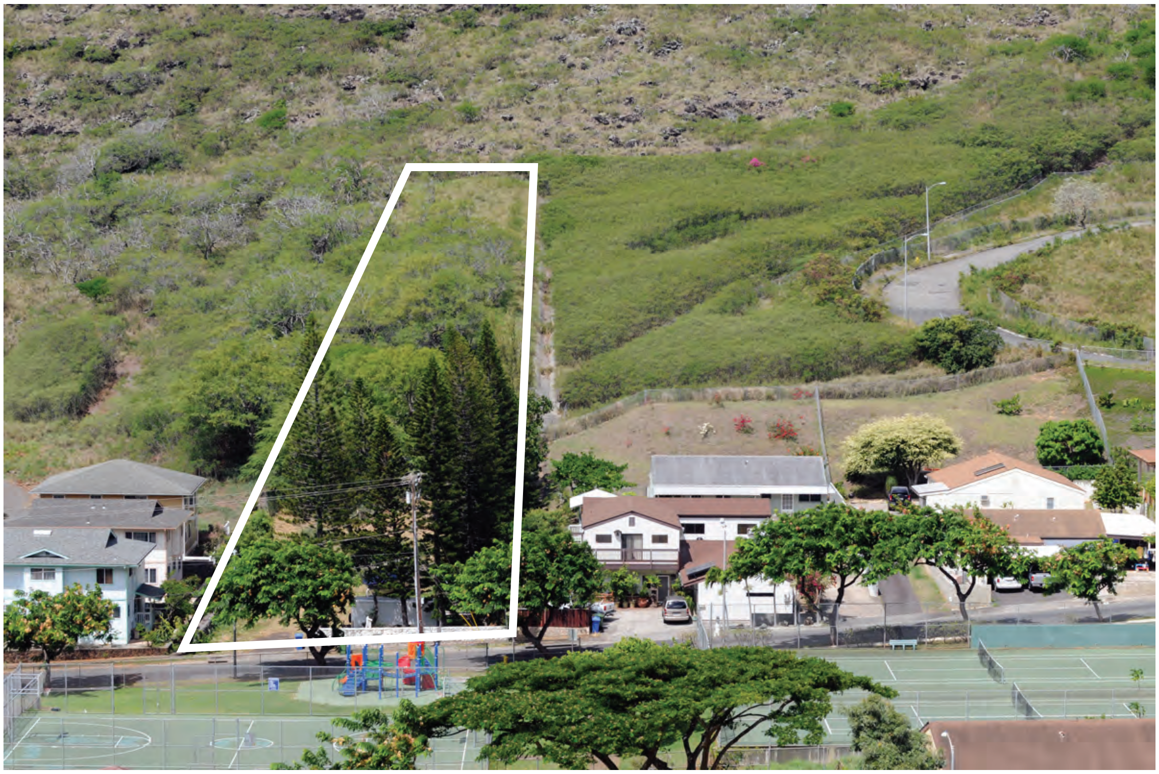
The comments and recommendations presented in this report are based, in part, on the soil conditions encountered in four borings and one test pit, earlier subsurface investigations conducted for the property, and upon information obtained from literature research and field exploration. Actual conditions beyond the location of the principal borings may differ from those described in this report. The nature and extent of these variations may not become evident until construction is underway. Applied Geosciences should be notified and retained to check if modifications to the recommendations presented in this report are needed if variations appear evident.

The stratification lines shown on the graphic representation of all the borings depict the approximate boundaries between the various soil and rock units, and as such may denote a gradual transition. Fluctuations in the groundwater level may occur due to variations in rainfall, temperature, tides and other factors that may be different from the conditions that existed at the time the boreholes were drilled. This report does not reflect variations that may result in the subsurface and groundwater conditions. Such subsurface and groundwater conditions may not become evident until construction.

The field exploration portion of this study may not have disclosed the presence of underground structures such as cesspools, drywells, storage tanks, sumps, pits, landfills, buried debris, cavities, voids, etc., that may be present at the site. Should these items be encountered during construction, Applied Geosciences should be notified and retained to provide recommendations for their disposal and/or treatment. Assessment of the presence or absence of these structures was not included in the scope of this study. The scope of Applied Geosciences exploration services was limited to conventional geotechnical engineering services and did not include any environmental assessment or evaluation of potential subsurface and groundwater contamination. Silence in this report regarding any environmental aspects of the site subsurface and groundwater materials does not indicate the absence of potential environmental problems.

This geotechnical report has been prepared for the use of the client, 510 Kuliouou LLC, and its designated engineering consultants in accordance with generally accepted soils and foundation engineering practices. No other warranty, expressed or implied, is made as to the professional advice included in this report and none should be inferred. This report has been developed for the purpose of site grading and construction as described elsewhere in this report. In addition, this report may not contain sufficient data or proper information to serve as the basis for preparation of construction estimates. A contractor wishing to bid on this project is urged to retain a qualified geotechnical engineer to assist in the interpretation of this report and/or in the performance of additional site-specific exploration for bid estimating purposes.

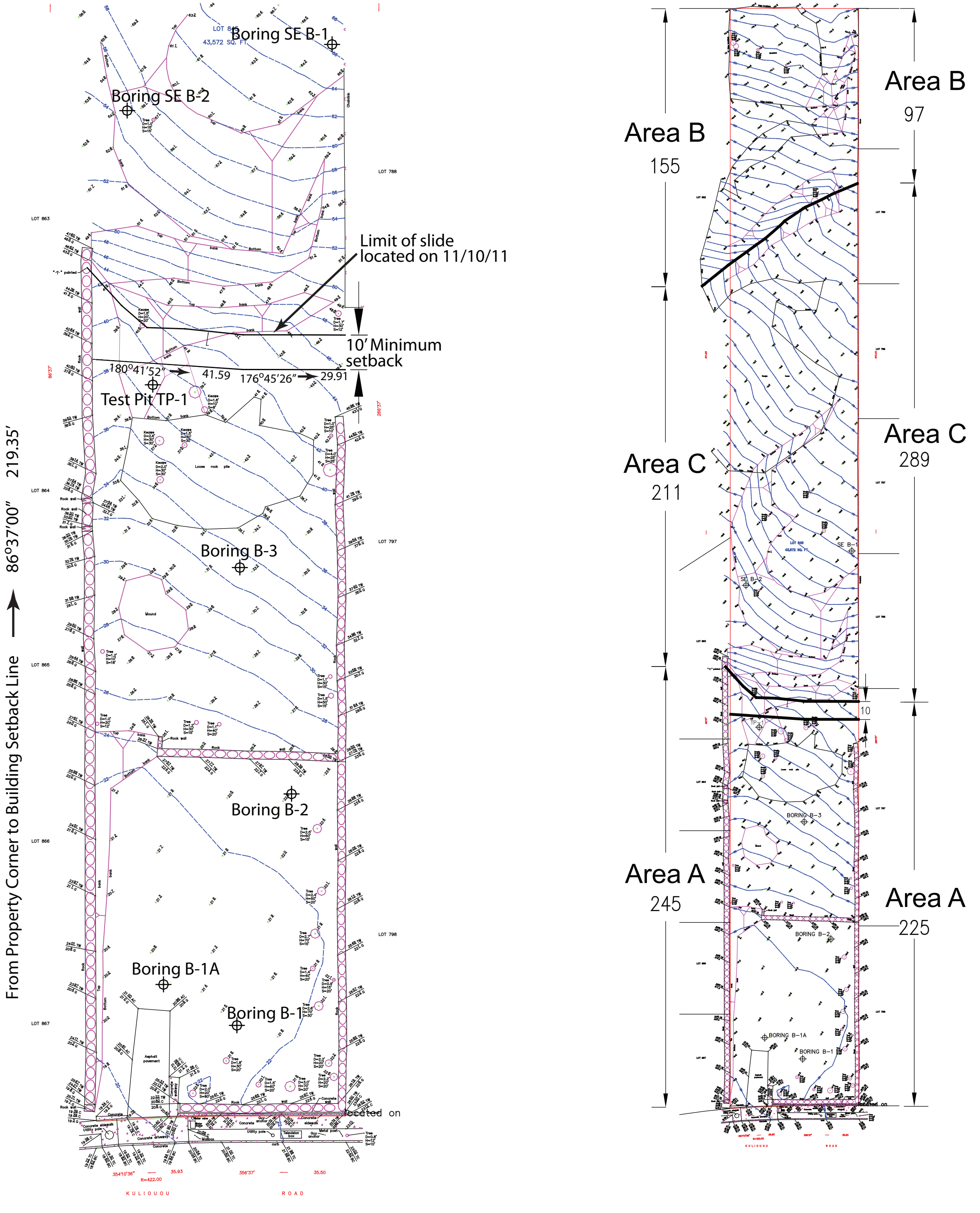
The owner/client should be aware that unanticipated subsurface conditions are commonly encountered. Unforeseen subsurface conditions, such as perched groundwater, soft deposits, hard layers, or cavities, may occur in localized areas and may require additional probing or corrections in the field (which may result in construction delays) to attain a properly constructed project. Therefore, a sufficient contingency fund is recommended to accommodate these possible extra costs.



Project: Geotechnical Investigation
Location: 510 Kuli‘ou‘ou Road
Applied Geosciences, LLC

Property Location in Kuliouou Valley

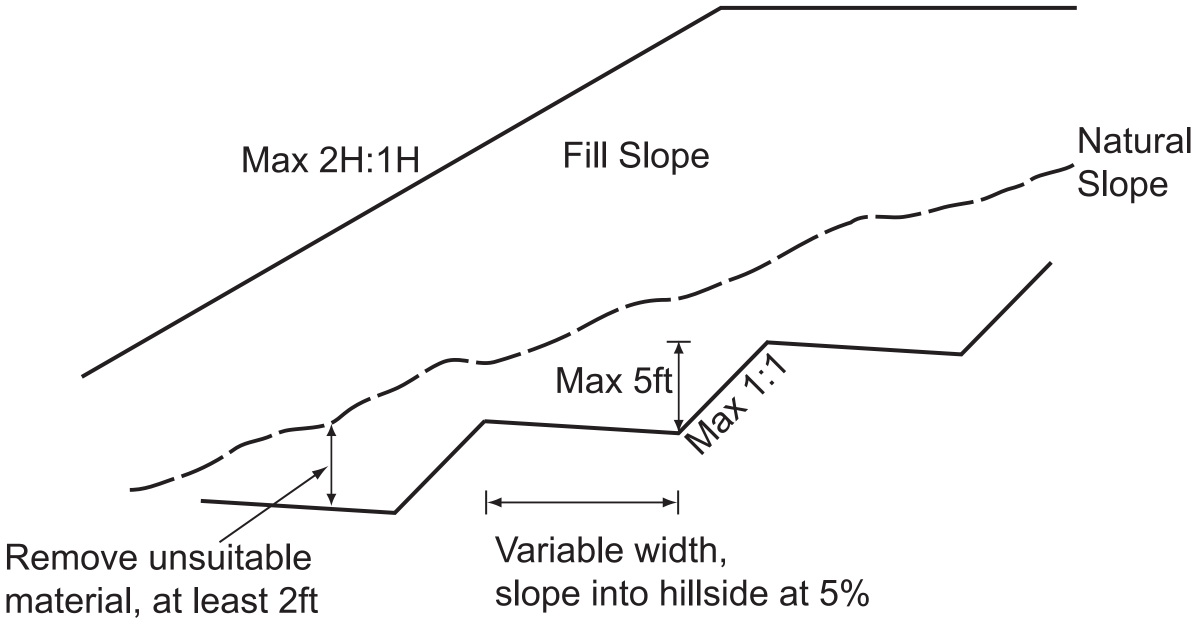
Figure No: 1



Project: Geotechnical Investigation
Location: 510 Kuli'ou'ou Road
Applied Geosciences, LLC

Location of Soil Borings, Test Pit, Slide Limits and Project Area A

Figure No: 2



Notes: 1. Not drawn to scale
2. All earhwork involving more than 30 yd^3 to be approved and construction supervised by project geotechnical engineer

Project: Geotechnical Investigation Location: 510 Kuli‘ou‘ou Road ***Applied Geosciences, LLC***	Fill Slope Details	Figure No: 3

APPENDIX A
Field Exploration

The subsurface conditions at the project site were explored by drilling and sampling four borings and one test pit. The location of the borings is shown in Figure 2.

The boreholes were drilled using an ATV-mounted rig that advanced a 4-inch auger. Samples were obtained with a standard 2-inch split-spoon sampler driven by a 140-lb weight descending a distance of 30 inches, or with an equivalent California sampler. Penetration numbers (blow counts) represent the number of blows needed to advance the sampler 12 inches, following an initial penetration of 6 inches, unless noted otherwise. Soil specimens collected from the boreholes and the test pit were inspected, described, and stored in sealed bags for laboratory testing. The test pit was excavated with a backhoe. The results from laboratory testing are included in Appendix B.

Subsurface Logs:

Figures 4-7:	Borings B-1, B-1A, B-2, B-3
Figure 8:	Test Pit TP-1

BORING NO. B-1

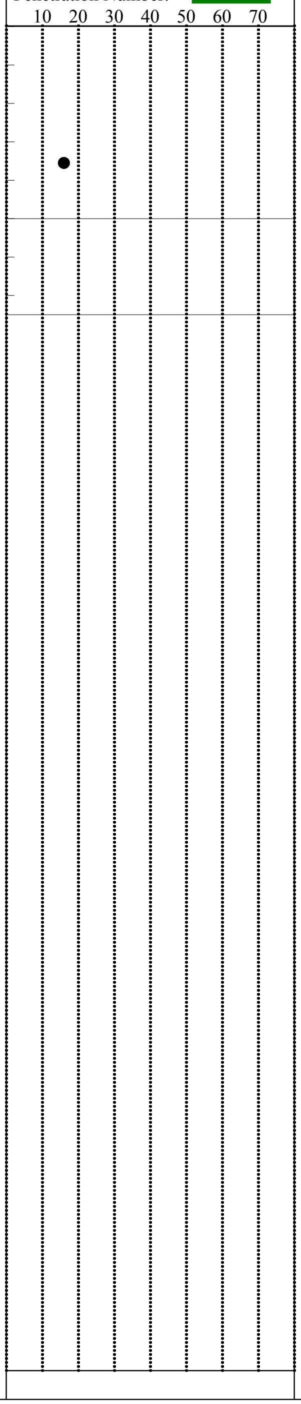
Applied Geosciences, LLC

File: November Borings
Date Printed: 12/15/2011

PROJECT: 510 Kuliouou Soils Investigation
PROJECT NO.: SRSS00112
CLIENT: 510 Kuliouou, LLC
DATE: 11/10/11
LOCATION: 510 Kuliouou Road, TMK: 3-8-010:004
ELEVATION:
DRILLER: SEI
LOGGED BY: HB
DRILLING METHOD: Badger 4" solid stem auger
DEPTH TO - WATER> INITIAL: ▽ None
AFTER 24 HOURS: ▼ None

DEPTH (feet)	DESCRIPTION	Soil Type	Sampler	N-Value (Blows/6")
0	Sandy clayey SILT with fractured basalt, roots, very stiff			
2	@ 2' start extensive fractured and weathered basalt rock (cobbles and boulders) in silt/clay matrix		◣	(14,18,40/2")
	END OF BORING at 3 FT			

Plastic Limit ├──┤ Liquid Limit
Water Content: ●
Penetration Number: 🟩



This information pertains only to this boring and should not be interpreted as being indicative of the site

Figure 4

◣ SPT Sampler
■ California 3-inch Sampler

BORING NO. B-1A

Applied Geosciences, LLC

File: November Borings

Date Printed: 12/15/2011

PROJECT: 510 Kuliouou Soils Investigation

PROJECT NO.: SRSS00112

CLIENT: 510 Kuliouou, LLC

DATE: 11/10/11

LOCATION: 510 Kuliouou Road, TMK: 3-8-010:004

ELEVATION:

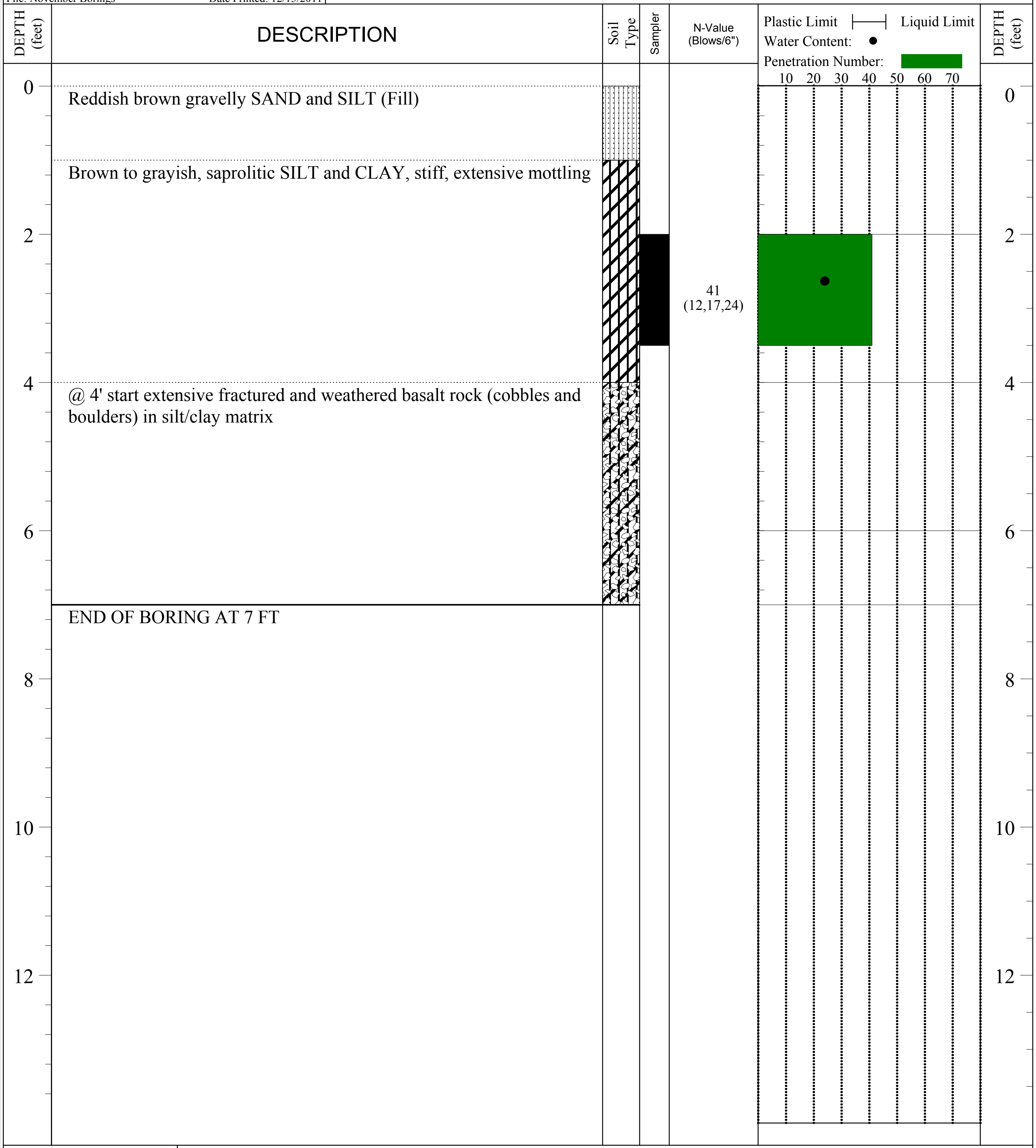
DRILLER: SEI

LOGGED BY: HB

DRILLING METHOD:

DEPTH TO - WATER> INITIAL: ∇ None

AFTER 24 HOURS: ▼ None



Reddish brown gravelly SAND and SILT (Fill)

Brown to grayish, saprolitic SILT and CLAY, stiff, extensive mottling

@ 4' start extensive fractured and weathered basalt rock (cobbles and boulders) in silt/clay matrix

END OF BORING AT 7 FT

This information pertains only to this boring and should not be interpreted as being indicative of the site

Figure 5

◣ SPT Sampler

■ California 3-inch Sampler

BORING NO. B-2

Applied Geosciences, LLC

File: November Borings   Date Printed: 12/15/2011

PROJECT: 510 Kuliouou Soils Investigation   PROJECT NO.: SRSS00112
CLIENT: 510 Kuliouou, LLC   DATE: 11/10/11
LOCATION: 510 Kuliouou Road, TMK: 3-8-010:004   ELEVATION:
DRILLER: SEI   LOGGED BY: HB
DRILLING METHOD:
DEPTH TO - WATER> INITIAL: ▽ None   AFTER 24 HOURS: ▼ None

DEPTH (feet)	DESCRIPTION
0	Gray clayey SILT (MH), very stiff, high plasticity, expansive
2.5	@ 2.5' start extensive fractured and weathered basalt rock (cobbles and boulders) in silt/clay matrix
5	END OF BORING at 5 FT

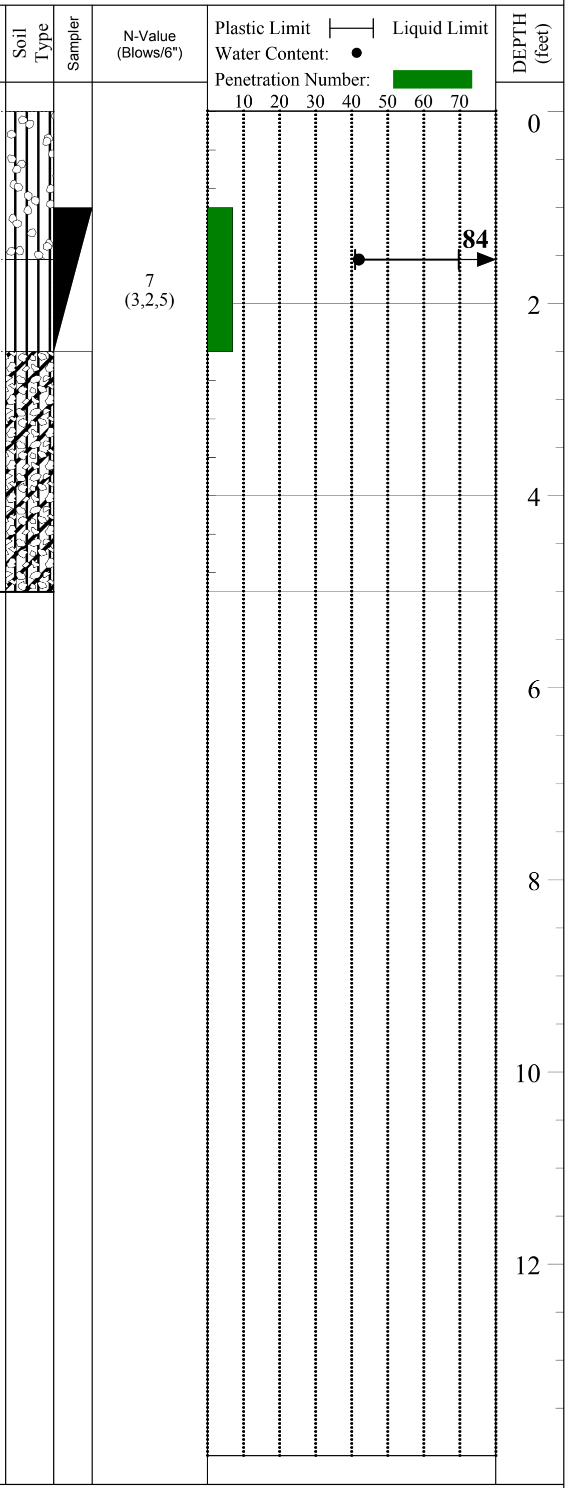


Figure 6

This information pertains only to this boring and should not be interpreted as being indicative of the site

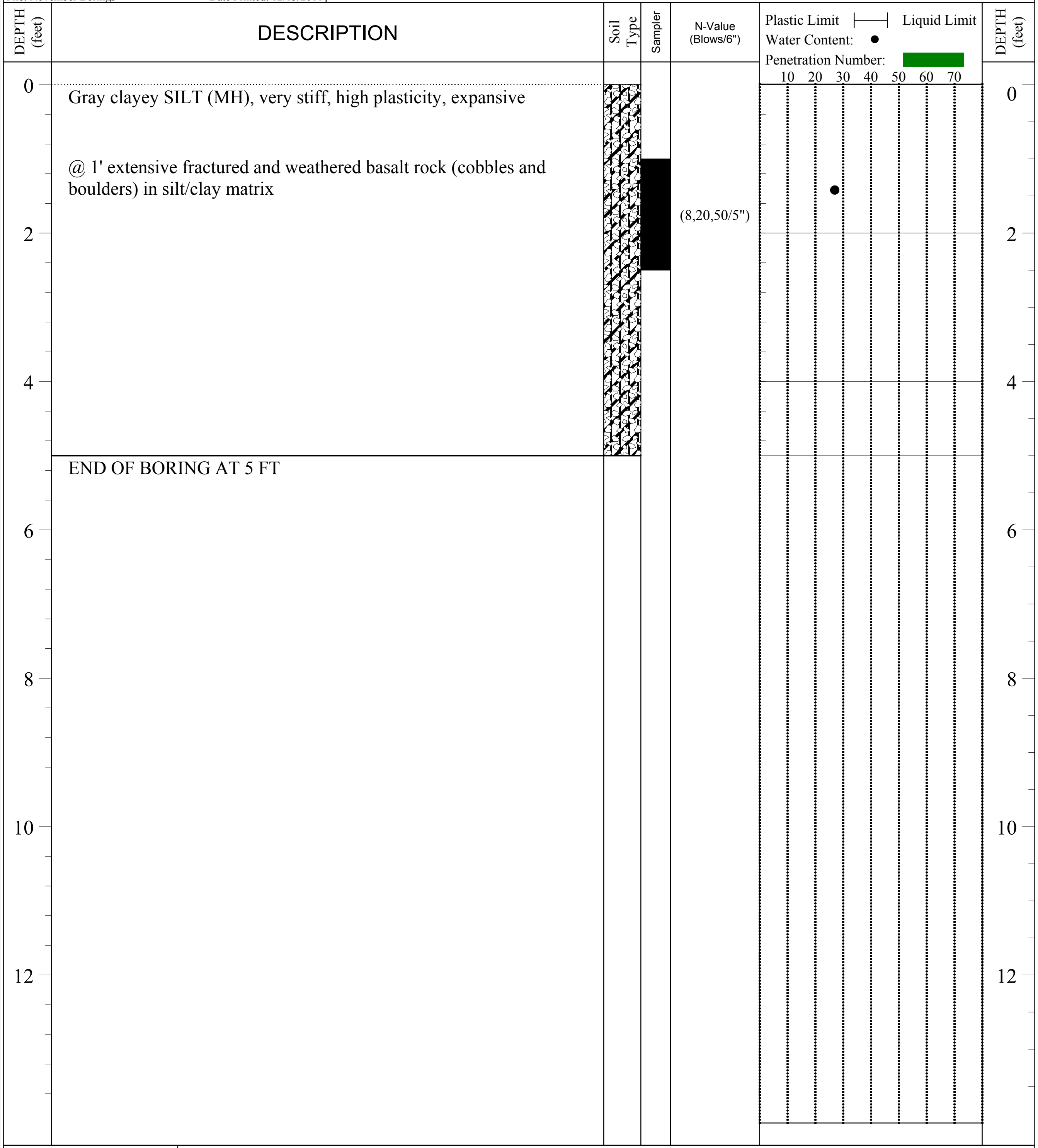
◤ SPT Sampler   ■ California 3-inch Sampler

BORING NO. B-3

Applied Geosciences, LLC

File: November Borings
Date Printed: 12/15/2011

PROJECT: 510 Kuliouou Soils Investigation
PROJECT NO.: SRSS00112
CLIENT: 510 Kuliouou, LLC
DATE: 11/10/11
LOCATION: 510 Kuliouou Road, TMK: 3-8-010:004
ELEVATION:
DRILLER: SEI
LOGGED BY: HB
DRILLING METHOD:
DEPTH TO - WATER> INITIAL: ▽ None
AFTER 24 HOURS: ▼ None



Gray clayey SILT (MH), very stiff, high plasticity, expansive

@ 1' extensive fractured and weathered basalt rock (cobbles and boulders) in silt/clay matrix

END OF BORING AT 5 FT

This information pertains only to this boring and should not be interpreted as being indicative of the site

Figure 7

◤ SPT Sampler

■ California 3-inch Sampler

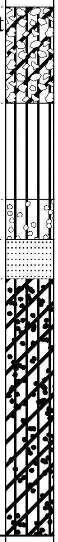
TEST PIT NO. TP-1

Applied Geosciences, LLC

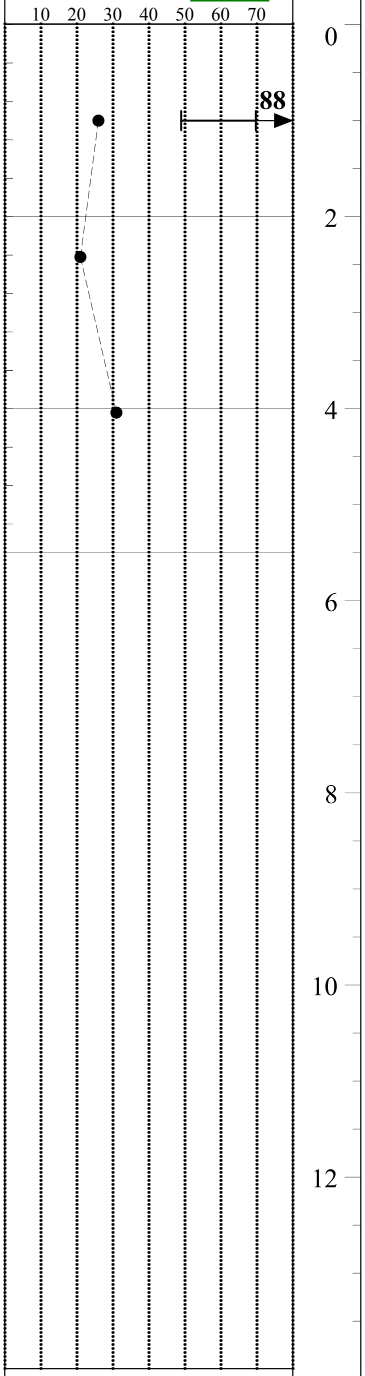
File: November Borings
Date Printed: 12/15/2011

PROJECT: 510 Kuliouou Soils Investigation
PROJECT NO.: SRSS00112
CLIENT: 510 Kuliouou, LLC
DATE: 10/29/11
LOCATION: 510 Kuliouou Road, TMK: 3-8-010:004
ELEVATION:
DRILLER:
LOGGED BY: HB
DRILLING METHOD:
DEPTH TO - WATER> INITIAL: ▽ None
AFTER 24 HOURS: ▼ None

DEPTH (feet)	DESCRIPTION	Soil Type	Sampler	N-Value (Blows/6")
0	Gray clayey SILT (MH), very stiff, high plasticity, expansive, abundant shrinkage cracks, 10-30% rounded cobbles and boulders up to 2'			
2	Light brown moderately cemented SAND, 30-40% rounded gravel and cobbles up to 4"			
	Black organic SILT and CLAY, moist, rounded gravel and cobbles up to 4"			
4				
	BOTTOM OF TEST PIT AT 5.5 FT			
6				
8				
10				
12				



Plastic Limit ├─┤ Liquid Limit
Water Content: ●
Penetration Number: 🟩



This information pertains only to this boring and should not be interpreted as being indicative of the site

Figure 8

◤ SPT Sampler

■ California 3-inch Sampler

APPENDIX B
Laboratory Testing

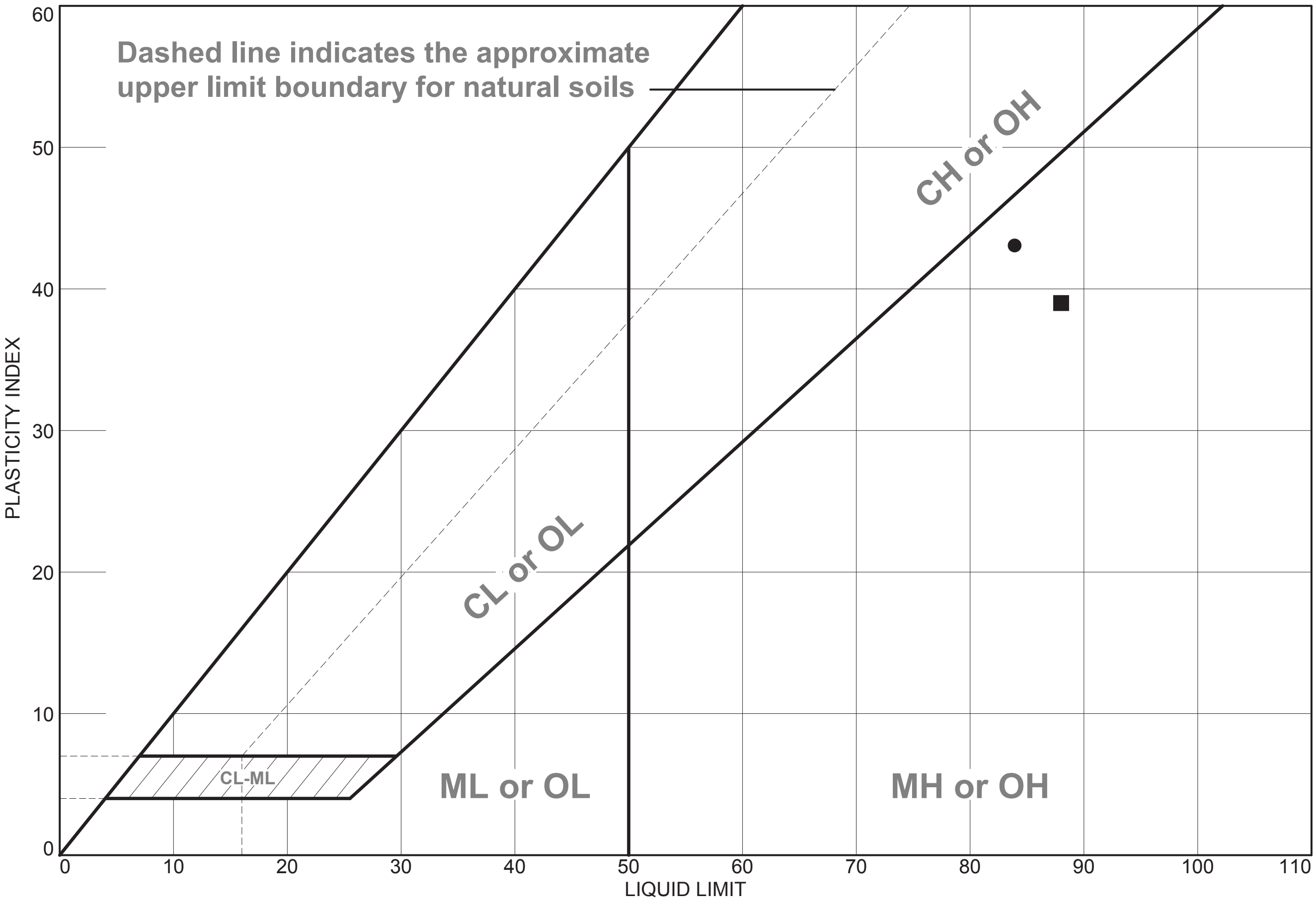
Water contents were determined on recovered specimens that were sealed in the field to preserve their in situ moisture (ASTM Designation D 2216).

Grain size distributions are based on the results from mechanical sieving and hydrometer testing (ASTM Designation D 422). It should be noted that these tests were carried out on samples recovered with a standard split-spoon sampler, which is unable to retrieve particles larger than 1-3/8 inches. Very coarse gravel, cobbles and boulders are not accounted for in the gradation curves.

Atterberg Limits were determined from specimens that maintained their field moisture levels at the time of sampling and were not allowed to dry out prior to testing (ASTM Designation D 4318).

Two 1D consolidometer tests were conducted on largely undisturbed ring samples obtained with the 3-inch California sampler. The specimen were saturated prior to testing and their swell pressure was measured under no-volume change conditions. The tests were conducted in general accordance with ASTM Designation D 2435.

Figure 9:	Atterberg Limits
Figure 10-11:	Particle Size Distributions
Figure 12:	1D Consolidation Test, B-1A at 2.0 feet
Figure 13:	1D Consolidation Test, B-3 at 1.0 feet



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Gray clayey silt	84	41	43	91.3	83.0	MH
■	Gray clayey silt	88	49	39	78.8	75.4	MH

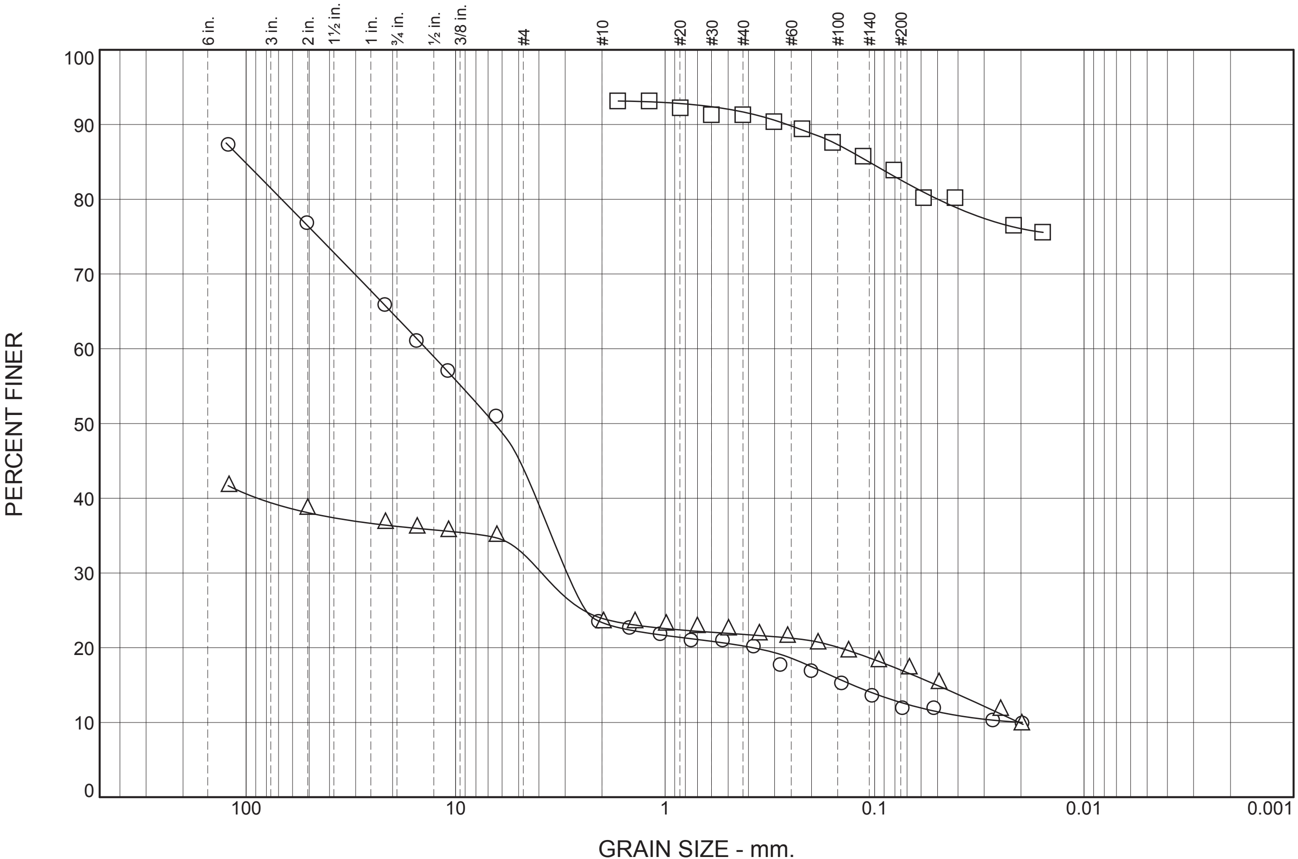
●**Source of Sample:** B-2 **Depth:** 1.54 ft
■**Source of Sample:** TP-1 **Depth:** 1.0 ft

Tested By: MP **Checked By:** HB

Project: Geotechnical Investigation
Location: 510 Kuli‘ou‘ou Road
Applied Geosciences, LLC

Atterberg Limits Test Results

Figure No: 9



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○		17.6	15.8	24.9	2.6	8.8	11.9	
□						8.3	83.0	
△		3.5	1.8	10.8	1.7	4.5	17.9	

SOIL DATA

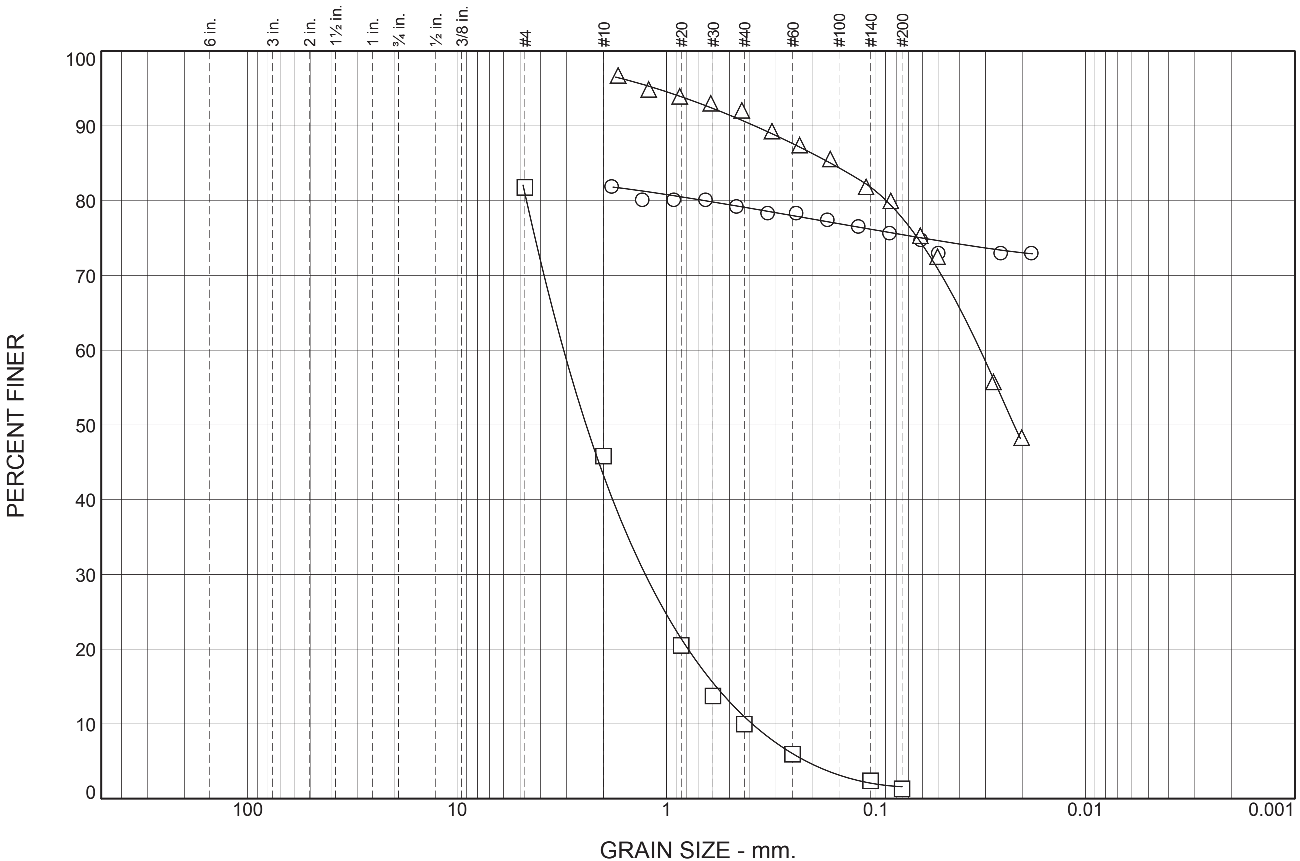
SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	Material Description	USCS
○	B-1A		2.63	Saprolitic silt and clay	
□	B-2		1.54	Gray clayey silt	MH
△	B-3		1.42	Fractured basalt rock and plastic silt	

Tested By: MP **Checked By:** HB

Project: Geotechnical Investigation
Location: 510 Kuli‘ou‘ou Road
Applied Geosciences, LLC

Grain Size Test Results: B-1A, B-2 and B-3

Figure No: 10



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○						3.4	75.4	
□				36.0	35.8	8.7	1.3	
△						13.5	78.4	

SOIL DATA

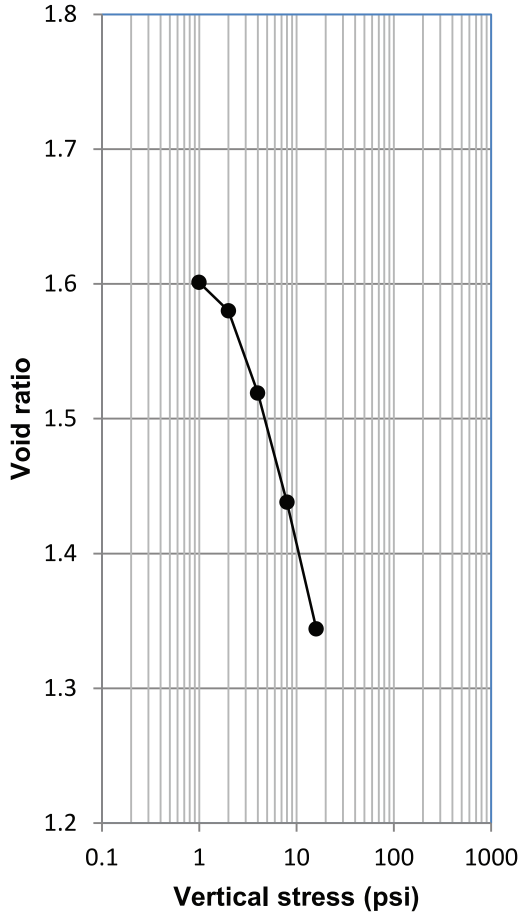
SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	Material Description	USCS
○	TP-1		1	Gray clayey silt	MH
□	TP-1		2.42	Black sand - minus gravel and larger (about 30-50%)	SW
△	TP-1		4.04	Gray plastic silt - minus gravel and larger (about 10%)	

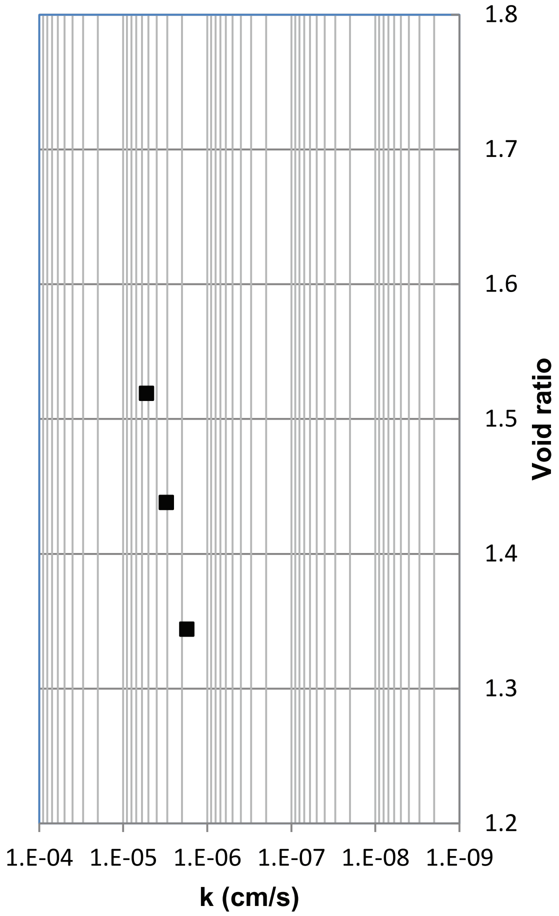
Tested By: MP **Checked By:** HB

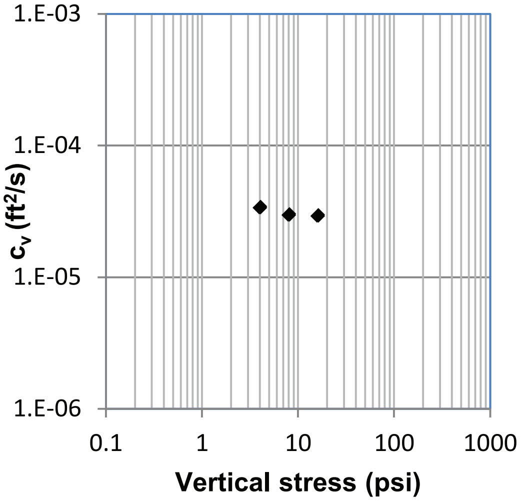
Project: Geotechnical Investigation
Location: 510 Kuli‘ou‘ou Road
Applied Geosciences, LLC

Grain Size Test Results: TP-1

Figure No: 11

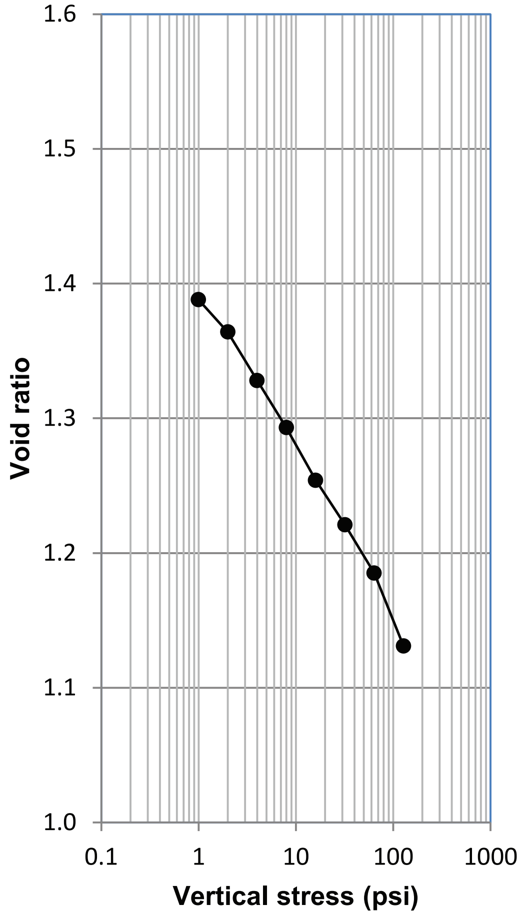


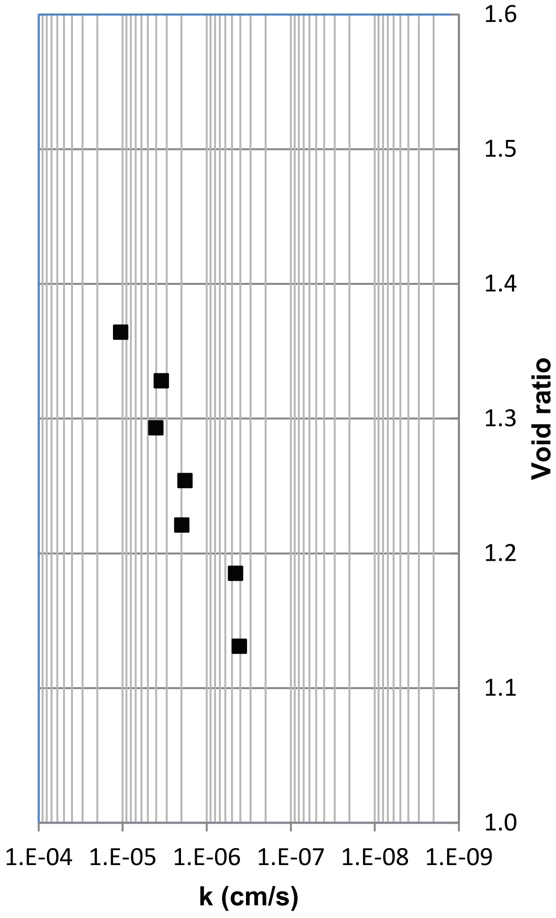


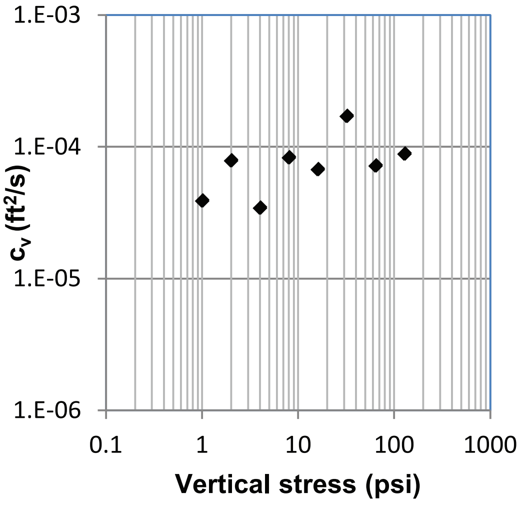


Initial water content = 24%
Initial seating pressure = 0.11psi (16psf)
Constant-volume swell pressure = 0.74psi (107psf)

Project: Geotechnical Investigation Location: 510 Kuli‘ou‘ou Road ***Applied Geosciences, LLC***	1D Consolidation Test Results Boring B-1A, Depth 2ft	Figure No: 12







Initial water content = 38%
Initial seating pressure = 0.11psi (16psf)
Constant-volume swell pressure = 0.56psi (81psf)

Project: Geotechnical Investigation
Location: 510 Kuli‘ou‘ou Road
Applied Geosciences, LLC

1D Consolidation Test Results
Boring B-3, Depth 1ft

Figure No: 13