# GEOTECHNICAL ENGINEERING EXPLORATION
# CASTLE HILLS ACCESS ROAD
# DRAINAGE IMPROVEMENTS
# PROJECT NO. HWY-O-04-98
# KANEOHE, OAHU, HAWAII

**APRIL 29, 2009**

*Prepared for*

*PAREN, INC. dba PARK ENGINEERING*

*and*

*STATE OF HAWAII*
*DEPARTMENT OF TRANSPORTATION*
*HIGHWAYS DIVISION*

**GEOLABS, INC.**

*Geotechnical Engineering and Drilling Services*

W.O. 4515-00

# GEOTECHNICAL ENGINEERING EXPLORATION
# CASTLE HILLS ACCESS ROAD
# DRAINAGE IMPROVEMENTS
# PROJECT NO. HWY-O-04-98
# KANEOHE, OAHU, HAWAII

**W.O. 4515-00 APRIL 29, 2009**

**PAREN, INC. dba PARK ENGINEERING**

and

**STATE OF HAWAII**
**DEPARTMENT OF TRANSPORTATION**
**HIGHWAYS DIVISION**

CLAYTON S. MIMURA
LICENSED PROFESSIONAL ENGINEER
No. 4176-C
HAWAII, U.S.A.

THIS WORK WAS PREPARED BY
ME OR UNDER MY SUPERVISION.

Clayton S. Mimura
SIGNATURE

4-30-10
EXPIRATION DATE OF THE LICENSE


**GEOLABS, INC.**
Geotechnical Engineering and Drilling Services
2006 Kalihi Street • Honolulu, HI 96819

Hawaii • California

**GEOLABS, INC.**
*Geotechnical Engineering and Drilling Services*

April 29, 2009
W.O. 4515-00

**Mr. Russell Arakaki**
**ParEn, Inc. dba Park Engineering**
711 Kapiolani Boulevard, Suite 1500
Honolulu, HI 96813

Dear **Mr. Arakaki:**

Geolabs, Inc. is pleased to submit our report entitled "Geotechnical Engineering Exploration, Castle Hills Access Road Drainage Improvements, Project No. Hwy-O-04-98, Kaneohe, Oahu, Hawaii" prepared in support of the design of the drainage improvement project.

Our work was performed in general accordance with the scope of services outlined in our fee proposal dated July 7, 1999.

Detailed discussion and specific recommendations for design of the project are contained in the body of this report. If there is any point that is not clear, please contact our office.

Very truly yours,

**GEOLABS, INC.**

Clayton S. Mimura

**Clayton S. Mimura P.E.**
President

CSM:JC:cj

2006 Kalihi Street • Honolulu, Hawaii 96819
Phone: (808) 841-5064 • Facsimile: (808) 847-1749 • E-mail: hawaii@geolabs.net

Hawaii • California

**GEOTECHNICAL ENGINEERING EXPLORATION**
**CASTLE HILLS ACCESS ROAD**
**DRAINAGE IMPROVEMENTS**
**PROJECT NO. HWY-O-04-98**
**KANEOHE, OAHU, HAWAII**
**W.O. 4515-00 APRIL 29, 2009**

---

## TABLE OF CONTENTS

Page

**GEOTECHNICAL ENGINEERING EXPLORATION**

**CASTLE HILLS ACCESS ROAD**

**DRAINAGE IMPROVEMENTSPROJECT NO. HWY-O-04-98**

**KANEOHE, OAHU, HAWAII**

**W.O. 4515-00 APRIL 24, 2009**

## SUMMARY OF FINDINGS AND RECOMMENDATIONS


Based on the borings drilled at the site, the project site generally is underlain by surface clayey soils overlying soft compressible peat and loose sandy alluvium to depths of 45 to 50 feet below the ground surface. The medium dense and/or medium stiff residual/saprolite materials were then encountered under the alluvium, extending to the maximum depth explored of approximately 80 feet below the existing ground surface.

Groundwater was encountered in the boring at a depth of approximately 6 feet below the existing ground surface during our field exploration. The groundwater levels encountered generally correspond to approximately Elevation +176 feet MSL. Groundwater levels will be affected by seasonal precipitation and other factors. Additionally, artesian pressures may be encountered in localized areas in the project vicinity.

Based on the subsurface conditions and structural loads provided, we recommend using micropile foundation to support the new concrete outlet structure. Due to higher lateral load demands, we recommend installing battered micropiles and extending to a minimum depth of 80 feet below the bottom of the outlet structure, including 40 feet of bonded and 40 feet of unbounded lengths. We envision designing the micropile foundation with a minimum 7-inch diameter grouted bulb to achieve compressive resistance capacities of 160 and 88 kips for extreme event and strength limit state, respectively.

To reduce continuous erosion on the stream banks and to provide a flexible retaining system accommodating soft compressible subsurface conditions, we recommend installing gabion walls on the downstream side of the concrete outlet structure. Bearing capacities of 3,300 and 1,500 psf may be used to design extreme event and strength limit states of the gabion wall foundation bearing on a 24-inch thick stabilization layer. The stabilization layer may consist of open-graded gravel (AASHTO M 43, No. 67 gradation) wrapped by the geotextile fabric and reinforced with geo-grid.

Provisions for excavation support by sheet pile shoring with associated dewatering and placement of a working platform is required. We recommend construction surveys by the contractor, consisting of pre-construction photographic and surface points surveying, surface settlement points and inclinometer monitoring during construction, and post-construction photographic and surface points surveying. The contractor is solely responsible for the adequacy and safety of the shoring system, as well as dewatering plan to address the impact and safety to the construction site and vicinity.

The text of this report should be referred to for detailed discussion and specific recommendations for design of the project.

END OF SUMMARY OF FINDINGS AND RECOMMENDATIONS

## SECTION 1. GENERAL

### 1.1 Introduction

This report presents the results of our geotechnical engineering exploration performed for the proposed Castle Hills Access Road Drainage Improvements project located in the Kaneohe District on the Island of Oahu, Hawaii. The project location and general vicinity are shown on the Project Location Map, Plate 1.

This report summarizes the findings from our field exploration and presents our geotechnical recommendations derived from our analyses for the proposed drainage improvements project. These recommendations are intended for the design of micropile foundation, gabion wall, and the provisions for excavation support and dewatering only. The findings and recommendations presented herein are subject to the limitations noted at the end of this report.

### 1.2 Project Considerations

The proposed drainage improvements project consists of constructing a new concrete drainage outlet structure and gabion walls on both sides of the existing stream banks at the downstream side of the outlet structure. The project vicinity is presented on the Site Plan, Plate 2

The existing drain culvert and outlet structure under Po'okela Street was constructed in the early to mid 1990s and is supported on gravel compaction piles. The existing stream banks are approximately 10 to 15 feet high. The southern stream banks, along the existing Castle Hills Subdivision, have rip-rap lining over the lower portions with ground slopes ranging from four horizontal to one vertical (4H:1V) to about 5H:1V above the lining. The northern stream banks are generally steep with many sections at near-vertical.

Based on the available as-built information, we understand that the vicinity of the project site was previously stabilized during mass grading of the Castle Hills subdivision, including surcharge and sand drains.

We understand that the rip-rap lining on the southern stream bank was undermined by the stream water, causing local instability in the upper slopes. In addition, portions of the northern stream bank collapsed due to the continuous erosion at the toe of the near-vertical slope.

It is desired to replace the existing concrete outlet structure and to install retaining structures at the downstream side of the outlet, to reduce the on-going erosion. Due to the potential slope instability and to provide construction access, a total of seven homes along the southern side of the stream and three homes along the northern side of the stream were purchased by the State of Hawaii. Additionally, one home on the northern side of the stream was acquired previously. The land of the acquired homes will be converted to a State Park in the future.

### 1.3 Purpose and Scope

The purpose of our exploration was to obtain an overview of the surface and subsurface conditions to develop a soil/rock data set to formulate geotechnical recommendations for the design of the outlet foundations and gabion walls for the proposed Castle Hills Access Road Drainage Improvements project. In order to accomplish this, we conducted an exploration program consisting of the following tasks and efforts:

1. Research of in-house and readily available soil and geologic information and as-built plans.
2. Mobilization and demobilization of potable and truck-mounted drill rigs and two operators to and from the project site.
3. Drilling and sampling of two boreholes extending to depths ranging from about 70 to 80 feet below the existing ground surface.
4. Installation of inclinometer casing in a selected borehole, including surface completion with flush-mounted manhole covers.
5. Coordination of the field exploration, boring stakeout, utility toning, and logging of the borings by our geologist.

6. Geotechnical laboratory testing of selected samples obtained during the field exploration as an aid in classifying the materials and evaluating their engineering properties.

7. Analyses of the field and laboratory data to formulate geotechnical recommendations for the design of outlet foundations and gabion retaining wall.

8. Preparation of this report summarizing our work on the project and presenting our findings and recommendations.

9. Coordination of our overall work on the project by our senior project engineer.

10. Quality assurance of our work and client/design team consultation by our principal engineer.

11. Miscellaneous work efforts such as drafting, word processing, and clerical support.

Detailed descriptions of our field exploration methodology and the Logs of Borings are presented in Appendix A. Results of the geotechnical laboratory tests performed on selected samples retrieved from our field exploration are presented in Appendix B. Results of the inclinometer monitoring are presented in Appendix C.

END OF GENERAL

# SECTION 2. SITE CHARACTERIZATION

## 2.1 Regional Geology

The Island of Oahu was built by the extrusion of basalt and basaltic lavas from two shield volcanoes, Waianae and Koolau. The older Waianae Volcano is estimated to be middle to late Pliocene in age, and the younger Koolau Volcano is estimated to be late Pliocene to early Pleistocene in age. After a long period of volcanic inactivity, during which time erosion incised deep valleys into the Koolau Shield, volcanic activity returned with a series of lava flows followed by cinder and tuff cone formations. These series are referred to as the Honolulu Volcanic Series.

The caldera of the Koolau Volcano extends from Kaneohe to Waimanalo, and from the base of the Pali to Lanikai. The project site is within the Koolau caldera area.

During the Pleistocene Epoch (Ice Age), sea levels fluctuated in response to the cycles of continental glaciation. As the glaciers grew and advanced, less water was available to fill the oceanic basins such that sea levels fell below the present stands of the sea. When the glaciers melted and receded, an excess of water became available such that the sea levels rose to above the present sea level.

The processes of erosion and deposition were affected by these glacio-eustatic sea level fluctuations. When the sea level was low, the erosion base level was correspondingly lower, and valleys were carved to depths below the present sea level. When the sea level was high, the erosion base level was raised such that sediments accumulated at higher elevations.

In the mountainous regions of Hawaii, the erosion processes are dominated by detachment of soil and rock masses from the valley walls and are transported downslope toward the axis of a valley, primarily by gravity, as colluvium. Once these materials reach the stream in the central portion of a valley, alluvial processes become dominant, and the sediments are transported and deposited as alluvium.

In general, stream flows in Hawaii are intermittent and flashy, such that the stream flows transmit large volumes of water for very short durations. Because of this, the transport of sediments is intermittent, and the bulk of the stream's hydraulic load consists of a poorly sorted mixture of boulders, cobbles, gravel, sands, and fines. When the erosional base levels change, these sediment loads are left as deposits.

When deposits are left in place for long periods of time, chemical processes begin to alter the materials simultaneously causing a breakdown or weathering of the material. Chemical processes also cause induration or cementation of the coarse-grained portion of the sediment into a poorly consolidated sedimentary rock, or conglomerate. Simultaneously, erosion continues in the areas above the valley floors and upstream in headwaters. This continued erosion generates material that is transported downslope covering the older alluvial deposits.

Depending on the local base level and rate of transport, these newer sediments are generally transient in terms of geologic time. In addition, their consistency and density are generally less than those of the older, partially consolidated deposits. The deposits of low-permeability soil horizons at the base of the Koolau Range have created a confined aquifer condition. At the Kaneohe area, these confined aquifers are supplied by groundwater from dikes within the Koolau Range. Since water within a confined aquifer is under pressure, boreholes that penetrate these aquifers will encounter a piezometric or artesian head of groundwater.

## 2.2 Existing Site Conditions

The project site is within the vicinity of the existing Castle Hills Subdivision in the Kaneohe district on the Island of Oahu, Hawaii. It is bounded Kupohu Street (private) on the south, Po'okela Street (State) on the west, and Pilina Place (City) on the north, as shown on the Site Plan, Plate 2. The existing box culvert below Po'okela Street is supported on gravel compaction piles. The existing Kapunahala Stream traverses from a pond (upstream) through the box culvert to the approximately 300-foot long winding open natural stream. The stream extends into a box culvert further to the east.

Seven homes along the southern bank of the stream and two homes along the northern bank of the stream were purchased by the State and are vacant. Two additional homes were still occupied pending completion of acquisition.

Based on the topographic map provided, the ground elevations within the downstream project limit vary from +160 to +150 feet Mean Sea Level (MSL). The existing southern stream bank was about 15 to 20 feet high. Severe erosion undermining at the slope toe has resulted in localized slope failure on the upper portion of the southern slope.

The northern stream bank was about 15 feet high consisting of steep slopes with some near-vertical portions collapsed due to slope toe undermining by stream erosion. The collapsed near-vertical slope exposed soft to medium stiff clayey soil with cobbles and organic matter.

## 2.3 Subsurface Conditions

The subsurface conditions at the project site were explored by drilling and sampling two borings, designated as Boring Nos. 1 and 2, one on each side of the stream bank. The borings were advanced to depths of approximately 70 to 80 feet below the ground surface. The approximate boring locations are shown on the Site Plan, Plate 2.

In general, the project site is underlain by about 5 to 10 feet of surface clayey soil overlying 5 to 15 feet of highly compressible soft peat. Loose to medium dense sandy and gravelly alluvial deposits were encountered below the peat and extended to depths of about 45 to 50 feet below the ground surface.

The alluvial deposits were underlain by medium stiff and/or medium dense residual/saprolite materials extending to the maximum depth drilled of about 80 feet below the ground surface.

Groundwater was encountered at a depth of approximately 6 feet below the existing ground surface during our field exploration. The groundwater level encountered generally correspond to an elevation of approximately +176 feet MSL. Groundwater level at the project site likely will be affected by seasonal precipitation, and other factors. Additionally, artesian pressures may be encountered in localized areas in the project vicinity.

Inclinometer casing was installed in Boring No. 1 to monitor slope movements. The inclinometer was read between September 2001 and August 2005. The monitoring program was terminated due to significant ground movement which prevented access into the casing. The maximum lateral movement measured was about 7 to 9 inches at the ground surface and the depth of movement was about 20 feet.

Detailed descriptions of the materials encountered and water levels observed in the borings are presented on the Logs of Borings in Appendix A. The results of the geotechnical laboratory tests performed on selected samples are presented in Appendix B. The results of the inclinometer monitoring are presented in the Appendix C.

END OF SITE CHARACTERIZATION

## SECTION 3. DISCUSSION AND RECOMMENDATIONS

Our borings at the project site generally encountered soft to medium stiff surface clayey soils overlying soft compressible peat and loose to medium dense sandy and gravelly alluvial deposits, extending to the depths of about 45 to 50 feet below the ground surface. The alluvial deposits were underlain by medium stiff and/or medium dense residual/saprolite materials to the maximum depth explored of approximately 80 feet below the existing ground surface.

Groundwater was encountered at approximately 6 feet below the existing ground surface during our field exploration, which corresponds to an elevation of approximately +176 feet MSL. Groundwater levels will likely be affected by seasonal precipitation and other factors.

The inclinometer installed in Boring No. 1 indicated approximately 7 to 9 inches of lateral ground movement at the ground surface. The depth of the movement was about 20 feet. We believe that the appreciable ground movement was induced primarily by the erosion at the toe of the stream bank.

Based on the subsurface conditions and structural loads provided, we recommend using micropiles to support the new concrete outlet structure. The micropiles should be installed in a batter position of approximately 1H:8V. We recommend designing each micropile based on compressive resistance capacities of 160 and 88 kips for extreme event and strength limit states, respectively. We believe that each micropile would need to extend to a depth of about 80 feet below the bottom of the outlet structure based on a minimum 7-inch diameter grout bulb.

To reduce erosion on the stream banks and to provide a flexible retaining system, we recommend installing gabion walls to protect the stream banks along the downstream side of the outlet structure. The gabion walls may be constructed 15 feet high with a minimum of 24 inches embedment below the adjacent lowest grade. Bearing capacities of up to 3,300 and 1,500 psf for extreme event and strength limit states may be used to design the gabion wall foundation supported on a 24-inch thick stabilization layer, consisting of open-grade gravels (AASHTO M 43, No. 67 gradation). The stabilization layer should be wrapped by geotextile fabric and reinforced by geo-grid such as Tensar TriAx TX160 or equivalent.

The construction of the outlet structure and the gabion walls will require excavation support and dewatering. We envision that a sheet pile shoring will need to be installed to depths of 40 to 60 feet to provide sufficient lateral support and to reduce the ground movement in the project vicinity. It should be noted that the contractor should be solely responsible for the adequacy and safety of the shoring installation and dewatering operation.

We also recommend implementing construction surveys consisting of photographic survey, surface settlement points and inclinometers to monitor ground movement by the contractor. The construction surveys should be conducted prior to, during and post construction to evaluate if damage and movement have occurred due to the construction. Detailed discussion of these items and our geotechnical recommendations for design of the project are presented in the following sections.

## 3.1 Micropile Foundations

Based on the information provided, we understand a new concrete outlet structure will be constructed to replace the existing deteriorated outlet structure at the project site. The new outlet structure will be designed to accommodate an approximately 15-foot water drop from the downstream end of the existing box culvert and to resist lateral earth pressure exerted on the outlet structure walls.

Based on the subsurface conditions encountered in the borings drilled, we recommend using micropiles to support the new outlet structure. We recommend that the capacity of the micropile foundation be derived primarily from skin friction between the micropile grout bulb and the residual and saprolite materials encountered in the borings.

Due to the tight access conditions and limited available working space (within the existing stream) at the project site, we anticipate that a temporary coffer dam using sheet piles may be required for the new concrete outlet structure construction. We envision that the sheet piles may be installed to depths of 40 to 60 feet with bracing support, such as walers and struts, providing lateral stiffness. We also envision that dewatering will be required to provide a dry working platform for micropile installation and construction. The contractor should be responsible for the temporary coffer dam construction and dewatering operation.

A micropile consists of a small diameter (usually less than 12 inches), drilled and grouted, pile with steel reinforcing. The micropile foundation typically is constructed by drilling a borehole, placing reinforcing steel in the hole, and grouting the borehole. Micropiles are desirable because they can be installed in access restrictive environments and in numerous soil types and ground conditions. In addition, installation of the micropiles generally causes minimal disturbance to adjacent structures, the adjacent soils, and the environment.

We believe that a micropile system with a minimum grout bulb diameter of 7 inches may be used to support the new outlet structure. Based on the structural loads provided, we recommend designing each micropile with compressive load capacities of 160 and 88 kips for the extreme event and strength limit states, respectively. The compressive load capacities of the micropiles were computed generally based on the requirements contained in the AASHTO LRFD Bridge Design Specifications, 2008 Interim. In order to arrive at the micropile capacities for the strength limit state, a resistance factor of 0.55 has been applied to the extreme event limit state capacities for design of the micropiles. Based on the subsurface conditions encountered at the site, we recommend that the micropiles extend through the soft compressible peat and loose to medium dense alluvium deposits and derive load bearing support from the medium dense and/or medium stiff residual and saprolite materials encountered at greater depths.

In general, the micropiles should be spaced a minimum of 2.5 feet or 3 times the micropile diameter (measured from center-to-center), whichever is greater, to avoid further reduction in vertical load capacity due to group action and to facilitate drilling of the micropile holes. We understand the spacing of the micropile to be about 5.5 to 12 feet center-to-center. Therefore, the group deduction factor is not applied to the extreme event and strength limit state capacities.

Based on the subsurface conditions at the site and experience with similar conditions, we believe that an ultimate bond stress of about 2,500 pounds per square foot (psf) may be developed at the interface between the grout bulb and the adjacent bearing materials. We believe that each micropile would need to extend to a minimum depth of about 80 feet below the bottom of the outlet structure with a minimum 7-inch diameter grout bulb. The 80-foot depth is based on a 40-foot deep load bearing grout bulb developed within the residual and saprolite materials with a 40-foot grout zone (neglected) within the soft peat and loose alluvial deposits.

In general, the micropile foundation system provides low lateral load resistance due to the relatively small diameter of the micropile. The lateral load resistance contributed by the micropiles installed vertically should be neglected. However, the design of the concrete outlet structure requires higher lateral load demands, exerted on the 15-foot high concrete wall from lateral earth pressure and dynamic water force loads. Therefore, the micropiles should be installed in a batter position of approximately 1H:8V to provide sufficient lateral resistance.

It should be noted that the bond stress between the grout bulb and the soil is highly dependent on the drilling procedures and the grouting methods employed by the contractor to install the micropile. Therefore, the bond stress between the grout bulb and the soil may vary considerably between different contractors and micropile foundation systems. To determine whether the contractor's methods of micropile installation are adequate and to determine the ultimate grout-to-soil bond stress, we recommend installing a sacrificial pre-production load test micropile. In general, the purpose of the pre-production load test on the sacrificial micropile is to fulfill the following objectives:

- To examine the adequacy of the methods and equipment proposed by the contractor to install the micropiles to the required depth.

- To confirm or modify the estimated minimum depths of the micropiles by determining the ultimate grout-to-soil bond stress.
- To assess the contractor's method of drilling and grout injection.

Installation of the micropiles should be performed by a specialty contractor experienced in the construction of a micropile foundation system (minimum ten projects). Due to the specialized nature of the micropile foundation construction, observation and testing of the micropile foundation system should be designated a "Special Inspection" item. Therefore, we recommend that a Geolabs representative (Special Inspector) be present to observe the geotechnical aspects of the micropile foundation construction and testing.

## 3.2 Retaining Structures

We envision constructing a retaining wall system for the drop structure of the new concrete outlet. Parameters for design of foundations for retaining structures should be designed in accordance with the "Micropile Foundations" section of this report. In general, the following guidelines may be used for the preliminary design of the retaining structures.

### 3.2.1 Static Lateral Earth Pressures

Retaining structures should be designed to resist the lateral earth pressures due to the adjacent soils and surcharge effects. The recommended lateral earth pressures for design of retaining structures, expressed in equivalent fluid pressures of pounds per square foot per foot of depth (pcf), are presented below.

| LATERAL EARTH PRESSURES FOR DESIGN OF RETAINING STRUCTURES | | | |
|---|---|---|---|
| **Backfill Condition** | **Earth Pressure Component** | **Active** (pcf) | **At-Rest** (pcf) |
| Level Backfill | Horizontal | 45 | 60 |
| | Vertical | None | None |
| Maximum 5H:1V Sloping Backfill | Horizontal | 48 | 64 |
| | Vertical | 10 | 14 |

The values provided in the table above assume that the open-graded gravel (AASHTO M 43, No. 67 gradation) materials will be used to backfill behind the walls. It is assumed that the backfill behind retaining structures will be compacted by a tamper plate for densification.

In general, an active condition may be used for gravity walls and walls that are free to deflect by as much as 0.5 percent of the wall height. If the tops of the walls are not free to deflect beyond this degree, or are restrained, the walls should be designed for the at-rest condition. The lateral earth pressures presented do not include hydrostatic pressures that might be caused by groundwater trapped behind the walls.

Surcharge stresses due to areal surcharges, line loads, and point loads within a horizontal distance equal to the depth of the wall should be considered in the design. For uniform surcharge stresses imposed on the loaded side of the wall, a rectangular distribution with uniform pressure equal to 50 percent of the vertical surcharge pressure acting over the entire height of the wall, which is free to deflect (cantilever), may be used in design. For walls that are restrained, a rectangular distribution equal to 66 percent of the vertical surcharge pressure acting over the entire height of the wall may be used for design. Additional analyses during design may be needed to evaluate the surcharge effects of point loads and line loads.

### 3.2.2 Drainage

Retaining structures should be well drained to reduce the potential for build-up of hydrostatic pressures. As previously mentioned, the project site is in the vicinity of the confined pressurized aquifer. Seepage and/or artesian groundwater may be present within the excavation depths. Therefore, we recommend using permeable material, such as open graded gravel (AASHTO M 43, No. 67 gradation), to backfill the space between the temporary coffer dam and the retaining wall. The open-graded gravel should be completely wrapped with non-woven filter fabric (Mirafi 180N or equivalent). The weepholes should be installed at the middle and near bottom levels of the retaining wall and spaced not more than 6 feet apart.

In addition, the upper 12 inches of the retaining wall backfill should consist of relatively impervious material to reduce the potential for significant water infiltration behind the retaining structure unless covered by concrete slabs at the surface.

### 3.2.3 Other Considerations

As previously mentioned, we anticipate that a temporary coffer dam using sheet piles may be constructed for excavation support. Excavation equipment and excavated soils should not be stockpiled closer than a horizontal distance equal to the depth of the excavation from the edge of the excavation to reduce the potential for excessive ground movement.

Based on the subsurface conditions encountered in the borings, soft compressible peat deposits with organic matter will be present within the excavation depths. We believe that these excavated unsuitable soils should be hauled off the site.

Excavations for the construction of the retaining structures will need to be adequately shored, as open-cut excavations are not practical. Because of anticipated seepage and/or artesian groundwater within the excavation depths, we anticipate that dewatering may be necessary for retaining wall construction. The excavation support/shoring system and dewatering operation used must comply with applicable safety requirements, and the adequacy and safety of the shoring installation and dewatering operation should be made the sole responsibility of the contractor. His/her representative, who should be required to be continuously present on site during excavation and construction work, will have the best opportunity to promptly observe changing conditions during construction, such as unforeseen subsurface conditions, unexpectedly high groundwater table, inappropriate construction sequence or techniques, etc., which may affect the shoring stability.

## 3.3 Gabion Walls

Based on our field exploration and observation, the stream banks have experienced severe erosion causing localized slope stability failure. Our inclinometer monitoring indicated about 7 to 9 inches of lateral deflections near the top edge of the southern stream bank. The depth of the movement was about 15 to 20 feet below the ground surface. A portion of the northern stream bank collapsed due to undermining of the near-vertical slope from continuous erosion.

To protect the existing stream banks from continuous erosion and to reduce the potential risk of global slope stability failure affecting adjacent roadways and underground utilities, it is desired to construct a retaining system along the stream banks on the downstream side of the outlet structure.

Due to the minimum grading work allowed within the existing wetland boundary and soft compressible peat and loose sandy/gravelly alluvial deposits encountered in the borings, we recommend constructing a gabion wall to resist erosion of the stream banks. We believe that the gabion wall will provide a flexible retaining system that can allow some settlements and provide good drainage.

A gabion wall consists of rectangular wire mesh gabion baskets filled with rock to form a flexible and permeable gravity retaining structure. The gabion basket requires double twisted hexagonal steel wire mesh with further reinforcing by lacing the basket perimeters and diaphragm edges. Additional internal connecting wires are generally placed in between the rock fill providing a stiffer gabion basket. Series of filled gabion baskets are assembled together to form the retaining structure in the specified geometry.

To provide a stabilization layer for support of the gabion wall and construction working platform, we recommend over-excavating a minimum of 2 feet below the bottom of the gabion wall and backfilling with open-grade gravels (AASHTO M 43, No. 67 gradation) wrapped with non-woven filter fabric (Mirafi 180N or equivalent). The stabilization layer may be reinforced by placing a layer of geo-grid, such as Tensar TriAx TX160 or equivalent.

Based on the AASHTO LRFD Bridge Design Specifications, 2008 Interim, an ultimate bearing capacity of up to 3,300 pounds per square foot (psf) may be used to evaluate the extreme event limit state of the gabion wall bearing on the 2-foot thick stabilization layer. To evaluate the strength limit state of the retaining structure, a bearing pressure of up to 1,500 psf may be used with a resistance factor of 0.45. We recommend embedding the gabion wall a minimum of 2 feet below the stream bed.

The non-woven filter fabric is to provide separation between the open-graded aggregate and the fine-grained soils. In general, a minimum overlap of 2 feet should be provided between the ends of each roll of the filter fabric placed along the trench under dry conditions. However, we envision that placement of the stabilization layers would likely be in wet conditions and it would be difficult to observe the fabric installation process below the groundwater level. In addition, we believe that it would be difficult for the contractor to assure proper minimum overlap is maintained.

Therefore, we believe that the seams of fabric should be sewn (in lieu of overlapping) to provide a higher degree of assurance that gaps will not occur in the filter fabric. As a result, we recommend incorporating that sewing the seams of the filter fabric into the construction procedure to reduce the potential for large-scale settlements.

If the filter fabric is not properly overlapped, it is possible that the coarse aggregate placed would migrate through openings in the filter fabric and mix with the underlying soft soils. This may result in excessive settlement of the gabion wall due to loss of stabilization materials into the soft soils. Therefore, the filter fabric should be pulled taut prior to placement of the stabilization material.

It is critical to implement the proper installation procedures during gabion wall construction to provide a stable and durable gravity retaining structure. A manufacturer's representative should be on-site to provide the proper gabion wall installation guidance at the beginning of the gabion wall construction.

### 3.4 Excavation

In order to install the new concrete outlet structure and gabion retaining wall, we anticipate that excavations below the existing ground surface will be required for the project construction. Based on the planned outlet structure and gabion wall inverts and the soft and/or loose subsurface soil conditions encountered in the borings, we envision that temporary shoring of the excavations will likely be required for the construction.

Our field exploration at the project site encountered surface clayey soils overlying soft peat and loose sandy alluvial deposits extending to depths of about 45 to 50 feet below the ground surface. Therefore, we anticipate that conventional excavation techniques using backhoe equipment may be considered for the planned excavations. In general, we believe that interlocking steel sheet piling may be used for temporary shoring purposes, especially where dewatering of the excavations will be necessary. For shallow excavations and excavations in open areas, we believe that a cantilever sheet pile shoring system may be considered. For deeper excavations and excavations located adjacent to buildings, utilities, and pavements, we recommend using interlocking steel sheet piling with horizontal bracing such as wales and struts for excavation support in order to reduce the potential for significant adjacent ground movement.

To reduce risk of potential ground movement, we recommend constructing the excavation for the gabion retaining wall in sections no longer than 25 feet along the stream bank. The excavation equipment should not be placed behind the sheet pile shoring.

In addition, the contractor should develop and implement a monitoring program to detect ground movement and/or subsidence adjacent to the excavations, which may result in damage to nearby structures and pavements. It should be noted that minor settlements may occur during and after installation of the sheet piles. Therefore, we recommend that the contractor retain a qualified geotechnical engineer to design and evaluate the shoring system used.

### 3.4.1 Excavation Method

In general, the contractor should determine the method and equipment used for excavation subject to practical limits and safety considerations. Based on our field exploration and the available information, the surface soils and underlying soft peat and loose sandy alluvial deposits encountered in the borings may be excavated with conventional earthmoving equipment. However, it should be noted that some larger boulders may be embedded in the alluvial deposits, especially near the stream bed. In addition, gravel compaction piles were previously installed to support the existing concrete outlet structure. The limits of these gravel compaction piles installed are not clear. Excavation in these materials may require the use of heavy excavation equipment.

Due to the soft compressible peat deposit encountered in the borings, the excavated soils should not be stockpiled on site, to reduce potential of appreciable ground settlement and/or movement. In addition, the peat deposits have organic contents and are unsuitable for fills. Therefore, we recommend hauling the excavated soils off site.

### 3.4.2 Excavation Support

We anticipate that excavation depths up to about 15 to 20 feet may be required for the project construction. Based on the subsurface soil conditions encountered in the borings, we believe that shoring of the sides of the excavations will be necessary to protect personnel working in the following excavations. Temporary shoring of the excavations using steel sheet pile shoring, especially where dewatering of the excavations will be necessary, should be considered.

Use of a sheet pile shoring system may also serve as a cut-off wall to aid in the dewatering operations, which is further discussed in the following "Dewatering" section. The sheet piles should be driven with a suitable hammer to a sufficient depth to reduce the potential for areal ground subsidence and to reduce the amount of dewatering within the excavations. It should be noted that the excavations will likely encounter soft and/or loose soil deposits at the bottom. Therefore, there is a potential for bottom heave in these soft and/or loose soil conditions. The contractor should carefully evaluate the potential for bottom heave and design the shoring system accordingly.

It is important to install adequate sheeting prior to the excavation and to maintain it tight against the excavation walls with proper bracing during excavation. The properly braced sheeting is essential to reduce the potential for appreciable lateral movements of the adjacent ground into the excavation, which may result in potential settlements or distress to adjacent structures or other improvements, such as the roads or utilities.

The contractor should be soley responsible for the adequacy and safety of the shoring installation. His/her representative, who should be required to be continuously present on site during excavation and construction work, will have the best opportunity to promptly observe changing conditions during construction, such as unforeseen subsurface soil conditions, unexpectedly high groundwater table, inappropriate construction sequence or techniques, etc., which may adversely affect shoring stability.

However, it must be noted that some minor movements of the shoring system and the adjacent ground may still occur due to changes in earth stresses during excavation. Due to the complexity of the stress changes, it is difficult to accurately estimate the magnitude of movement. The magnitude also depends greatly upon workmanship, such as how quickly and tightly the shoring and bracing supports are installed, the subsoil conditions, the size of the excavation, and the rate of excavation. In addition, it should be noted that settlement of the existing ground may occur as a result of the vibrations generated during the extraction of the sheet pile shoring. Therefore, the contractor should give special attention during the sheet pile removal process to reduce the potential for appreciable ground settlement.

In addition, it is important to realize that the excavation shoring should be installed properly and as early as practical, if necessary, and that the adjacent ground should be continuously monitored for cracks, dips and/or other indications of movements with instruments.

## 3.5 Dewatering

Dewatering of excavations will be necessary where the existing groundwater level is above the bottom of the proposed excavation. Groundwater was encountered at a depth of about 6 feet below the existing ground surface in the borings drilled during our field exploration. Based on the groundwater levels anticipated and possible confined pressurized groundwater aquifer, we believe that dewatering will be required at the planned excavations for the new concrete outlet structure. Therefore, dewatering provisions will need to be included in the contract documents for the proposed construction. Because the excavation may involve discharge of groundwater, a National Pollutant Discharge Elimination System (NPDES) permit may be required for this discharge. The contractor should consult their independent consultant for the latest regulations and information pertaining to the permit application.

Because of the mixture of cohesive and granular subsurface soils, it is anticipated that moderately permeable soils may be encountered within the excavation depths. Dewatering by means of a well point system along the outside of the excavations generally is not recommended. The resultant areal depression of the natural groundwater table could induce consolidation of the compressible subsurface soils resulting in potential ground settlements, which could affect the existing structures. The potential impact of the dewatering system selected on depressing the natural groundwater table must be carefully evaluated by the contractor prior to dewatering.

It is our opinion that a cut-off wall system, such as interlocking steel sheet piles, should be considered to aid in dewatering the excavation. However, sumps will be needed to collect water that percolates up into the base of the excavation or infiltrates through the sheet piles. The sheet piles should be driven to a sufficient depth to reduce the potential for areal ground subsidence and to reduce the amount of dewatering within the excavations in areas underlain by granular subsoils with generally high permeability. Use of an interlocking steel sheet pile shoring support system is relatively watertight, which should allow the groundwater levels outside the excavations to be maintained close to the original pre-construction levels. Therefore, some type of groundwater control requirement should be specified in the contract documents.

The contractor is responsible for construction dewatering. Selection of equipment and methods of dewatering should be left up to the contractor, and he/she should be aware that modifications to the dewatering system may be required during construction depending on the conditions encountered. The dewatering method selected should have minimal impact on the groundwater level surrounding the proposed excavation. The dewatering operations should be coordinated with the shoring support such that the stability of the excavations is not jeopardized. The operations should be carried-out without softening the bottom of the excavations.

It is our opinion that the definition of "Dewatering" in the contract documents should be written to include all works or systems required to lower the natural groundwater table and/or to exclude water from the excavations to allow construction of the proposed utility structures under safe and dry conditions. These works or systems may include, but are not limited to, pumping.

#### 3.5.1 Subsurface Soil Permeability

We anticipate that the moderate permeability of the subsurface materials may vary due to the normally heterogeneous nature of the alluvial subsurface materials. The actual subsoil permeability may range broadly and also vary locally in terms of orders of magnitude. It should be noted that the permeability of the subsoils at the site appears to be moderate based on the field observations at the site. Therefore, special attention should be given to the site-specific dewatering plan for the proposed project.

#### 3.5.2 Dewatering Considerations

We suggest that the following three basic criteria be considered in selecting a suitable method of dewatering:

a. The dewatering method should result in the least disturbance or damage to existing buildings, roads, and environment.

b. The dewatering method should maintain stability of, and also provide safe and dry working conditions in, the excavation.

c. The dewatering method should be sufficiently flexible to allow modifications to accommodate various ground conditions.

#### 3.5.3 Dewatering Precaution and Monitoring

The contractor must carefully evaluate the potential impact of the dewatering system selected on depressing the natural groundwater table prior to dewatering. We recommend that the contractor retain a qualified geotechnical engineer to design and evaluate the dewatering system used.

The contractor should be solely responsible for the impact and safety of the dewatering operations. His/her qualified representative, who should be continuously present on-site during dewatering activities, will have the best opportunity to promptly observe the effects of dewatering during construction and to implement, as soon as possible, necessary precautionary or remedial measures including, but not limited to, slowing down or stopping the dewatering operations.

Where encountered at the bottom of excavations, permeable granular subsoils may be susceptible to piping and "quick" conditions. The dewatering operations should be carried-out without creating a "quick" condition or softening at the excavation bottoms. Therefore, the project dewatering operations should be performed without pumping out soil fines (pumping clear water only) and should be coordinated with the shoring installation such that the excavation stability is not adversely affected. Excessive pumping, which removes soil fines, may result in "blowing" or heaving of the excavation bottom or sides.

Groundwater drawdown outside the excavation will cause additional settlement resulting from consolidation of the soft and/or loose compressible soils. Therefore, the use of a deep well system outside the excavations to draw down the groundwater level should not be allowed.

## 3.6 Design Review

Preliminary and final drawings and specifications for the proposed Construction of Castle Hills Access Road Drainage Improvements project should be forwarded to Geolabs for review and written comments prior to bid advertisement. This review is necessary to evaluate conformance of the plans and specifications with the intent of the geotechnical recommendations provided herein. If this review is not made, Geolabs cannot be responsible for misinterpretation of our recommendations.

## 3.7 Construction Monitoring

It is recommended to retain Geolabs to provide geotechnical services during construction. The critical items of construction monitoring that require "Special Inspection" include observation of the micropile foundation installation, gabion wall installation, and excavation and backfill operations. A Geolabs representative should monitor other aspects of earthwork construction to observe compliance with the intent of the design concepts, specifications, or recommendations and to expedite suggestions for design changes that may be required in the event that subsurface conditions differ from those anticipated at the time this report was prepared. The recommendations provided herein are contingent upon such observations.

If the actual exposed subsurface conditions encountered during construction are different from those assumed or considered in this report, then appropriate modifications to the design should be made.

END OF DISCUSSION AND RECOMMENDATIONS

## SECTION 4. LIMITATIONS

The analyses and recommendations submitted herein are based, in part, upon information obtained from field borings drilled and available as-built drawings. Variations of subsurface conditions between and beyond the borings may occur, and the nature and extent of these variations may not become evident until construction is underway. If variations then appear evident, it will be necessary to re-evaluate the recommendations provided herein.

The field boring locations indicated in this report are approximate; having been taped from features shown on the General Layout plan provided by ParEn, Inc. dba Park Engineering dated April 13, 2009. Elevations on the boring logs were obtained based on interpolation between the spot elevations and contour lines shown on the same plan. The physical locations and elevations of the borings should be considered accurate only to the degree implied by the methods used.

The stratification lines shown on the graphic representations of the borings depict the approximate boundaries between soil/rock types and, as such, may denote a gradual transition. Water level data from the borings were measured at the times shown on the graphic representations and/or presented in the text of this report. These data have been reviewed and interpretations made in the formulation of this report. However, it must be noted that fluctuation may occur due to variation in rainfall, temperature, and other factors.

This report has been prepared for the exclusive use of ParEn, Inc. dba Park Engineering and their client, State of Hawaii - Department of Transportation Highways Division, for specific application to the proposed Castle Hills Access Road Drainage Improvements project in accordance with generally accepted geotechnical engineering principles and practices. No warranty is expressed or implied.

This report has been prepared solely for the purpose of assisting the engineer in the design of the proposed drainage improvements project. Therefore, this report may not contain sufficient data, or the proper information, to serve as the basis for preparation of construction cost estimates. A contractor wishing to bid on this project is urged to retain a competent 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 soil/rock conditions are commonly encountered. Unforeseen subsurface conditions, such as soft deposits, hard layers, cavities or perched groundwater, 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.

This geotechnical exploration conducted at the project site was not intended to investigate the potential presence of hazardous materials existing at the site. It should be noted the equipment, techniques, and personnel used to make a geo-environmental exploration differ substantially from those applied in geotechnical engineering.

END OF LIMITATIONS

## CLOSURE

The following plates and appendices are attached and complete this report:



-ΩΩΩΩΩΩΩΩΩ-

Respectfully submitted,

**GEOLABS, INC.**

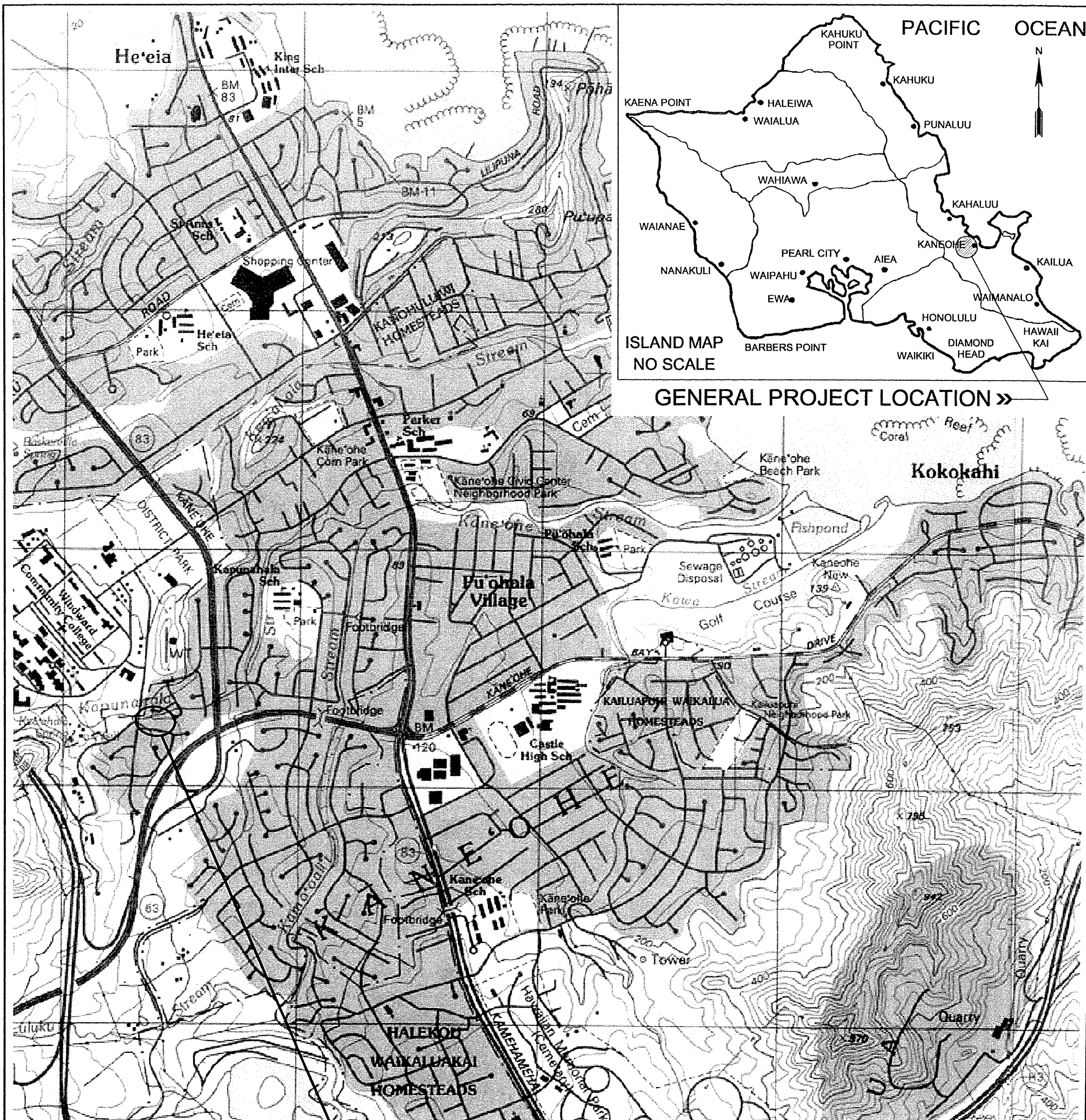
By ______________________
**John Y.L. Chen, P.E.**
Senior Project Engineer

By ______________________
**Clayton S. Mimura, P.E.**
President

CSM:JC:cj

h:\4500 Series\4515-00.jc1

# PLATES



PROJECT LOCATION »

# PROJECT LOCATION MAP

CASTLE HILLS ACCESS ROAD
DRAINAGE IMPROVEMENTS
PROJECT NO. HWY-0-04-98
KANEOHE, OAHU, HAWAII

N

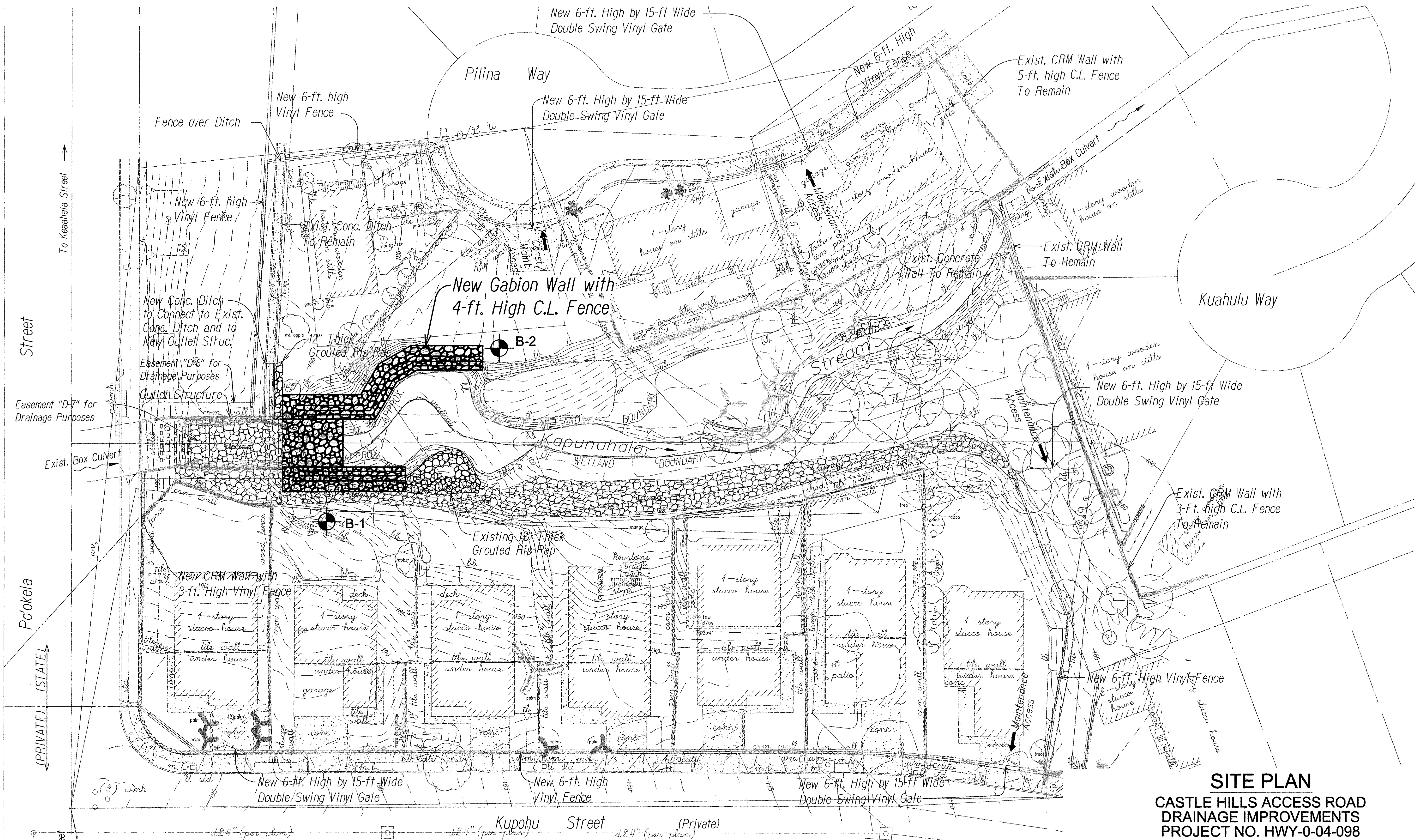
2000 1000 0 2000 FT.

GRAPHIC SCALE

REFERENCE: MAP CREATED WITH TOPO!® ©2001 NATIONAL GEOGRAPHIC (WWW.NATIONALGEOGRAPHIC.COM/TOPO).

| | | |
|---|---|---|
| GEOLABS, INC. Geotechnical Engineering | | |
| DATE APRIL 2009 | DRAWN BY KHN | PLATE |
| SCALE 1" = 2,000' | W.O. 4515-00 | 1 |

CAD User: KIM File Last Updated: April 20, 2009 2:03:25pm Plot Date: April 20, 2009 - 2:05:52pm
File: T:\Drafting-9904\Working\4515-00CastleHillAccessRoad\4515-00PLM.dwg\PLM
Plotter: DocuColor 240 PCL Plotstyle: GEOLABS-No-Dithering.ctb




**LEGEND:**

⊕ APPROXIMATE BORING LOCATION

0 20 40 80 120 FT.

GRAPHIC SCALE

N

# SITE PLAN

CASTLE HILLS ACCESS ROAD
DRAINAGE IMPROVEMENTS
PROJECT NO. HWY-0-04-098
KANEOHE, OAHU, HAWAII

GEOLABS, INC.
*Geotechnical Engineering*

| DATE | DRAWN BY | PLATE |
|---|---|---|
| APRIL 2009 | KHN | |
| SCALE | W.O. | 2 |
| 1" = 40' | 4515-00 | |

REFERENCE: GENERAL LAYOUT PLAN TRANSMITTED BY PAREN, INC. ON APRIL 13, 2009.

Plotter: DocuColor 240 PCL Plotstyle: GEOLABS-No-Dithering.ctb

# APPENDIX A

# APPENDIX A

Field Exploration

The subsurface conditions at the project site were explored by drilling and sampling two boreholes, designated as Boring Nos. 1 and 2, extending to depths of about 70 to 80 feet below the existing ground surface. The borings were drilled using a portable drill rig equipped with continuous flight augers and rotary wash drilling tools. The approximate boring locations are shown on the Site Plan, Plate 2.

The materials encountered in the borings were classified by visual and textural examination in the field by our geologist, who monitored the drilling operations on a near-continuous basis. Soils were classified in general conformance with the Unified Soil Classification System, as shown on Plate A. Graphic representations of the materials encountered are presented on the Logs of Borings, Plates A-1.1 through A-2.3.

Relatively "undisturbed" soil samples were obtained from the borings drilled in general accordance with ASTM D 3550, Ring-Lined Barrel Sampling of Soils, by driving a 3-inch OD Modified California sampler with a 140-pound hammer falling 30 inches. In addition, some samples were obtained from the borings drilled in general accordance with ASTM Test Designation D 1586, Penetration Test and Split-Barrel Sampling of Soils, by driving a 2-inch OD standard penetration sampler using the same hammer and drop. The blow counts needed to drive the sampler the second and third 6 inches of an 18-inch drive are shown as the "Penetration Resistance" on the Logs of Borings at the appropriate sample depths.

| | |
|---|---|
| **GEOLABS, INC.**<br>Geotechnical Engineering | Soil Log Legend |

## UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)

| MAJOR DIVISIONS | | | USCS | TYPICAL DESCRIPTIONS |
|---|---|---|---|---|
| COARSE-GRAINED SOILS<br>MORE THAN 50% OF MATERIAL RETAINED ON NO. 200 SIEVE | GRAVELS<br>MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE | CLEAN GRAVELS<br>LESS THAN 5% 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<br>MORE THAN 12% FINES | **GM** | SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES |
| | | | **GC** | CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES |
| | SANDS<br>50% OR MORE OF COARSE FRACTION PASSING THROUGH NO. 4 SIEVE | CLEAN SANDS<br>LESS THAN 5% FINES | **SW** | WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES |
| | | | **SP** | POORLY-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES |
| | | SANDS WITH FINES<br>MORE THAN 12% FINES | **SM** | SILTY SANDS, SAND-SILT MIXTURES |
| | | | **SC** | CLAYEY SANDS, SAND-CLAY MIXTURES |
| FINE-GRAINED SOILS<br>50% OR MORE OF MATERIAL PASSING THROUGH NO. 200 SIEVE | 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 50 OR MORE | **MH** | INORGANIC SILT, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS |
| | | | **CH** | INORGANIC CLAYS OF HIGH PLASTICITY |
| | | | **OH** | ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS |
| HIGHLY ORGANIC SOILS | | | **PT** | PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS |

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

LEGEND

(2-INCH) O.D. STANDARD PENETRATION TEST

(3-INCH) O.D. MODIFIED CALIFORNIA SAMPLE

SHELBY TUBE SAMPLE

GRAB SAMPLE

CORE SAMPLE

WATER LEVEL OBSERVED IN BORING

LL LIQUID LIMIT (NP=NON-PLASTIC)

PI PLASTICITY INDEX (NP=NON-PLASTIC)

TV TORVANE SHEAR (tsf)

PEN POCKET PENETROMETER (tsf)

UC UNCONFINED COMPRESSION (psi)

UU UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION (ksf)

Plate

A

| GEOLABS, INC.<br>Geotechnical Engineering | CASTLE HILLS ACCESS ROAD<br>DRAINAGE IMPROVEMENTS<br>PROJECT NO. HWY-0-04-98<br>KANEOHE, OAHU, HAWAII | Log of Boring<br>1 |
|---|---|---|

| Laboratory | | | Field | | | | | | | | Approximate Ground Surface Elevation (feet MSL): 182 * |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Other Tests | Moisture Content (%) | Dry Density (pcf) | Core Recovery (%) | RQD (%) | Penetration Resistance (blows/foot) | Pocket Pen. (tsf) | Depth (feet) | Sample | Graphic | USCS | Description |
| | | | | | | | | | | MH | Brown **CLAYEY SILT** with traces of highly weathered gravel and sand (basaltic), medium stiff, moist (fill) |
| | 41 | 55 | | | 25 | | | | | | |
| | 47 | | | | 8 | | | | | | |
| LL=99<br>PI=63 | 110 | 38 | | | 2 | 0.0 | 5 | | | OH | Grayish brown **CLAY** with traces of roots and organic matter, very soft (peat) |
| | 125 | | | | 2 | | 10 | | | | |
| | 226 | 22 | | | 3 | | 15 | | | | |
| | 53 | | | | 4 | | 20 | | | SM | Dark gray **SILTY SAND** with some weathered gravel (basaltic), loose (alluvium) |
| | 25 | | | | 12 | | 25 | | | | grades to loose to medium dense |
| | 53 | | | | 11 | | 30 | | | SW | Dark gray **SAND** with traces of weathered gravel (basaltic), loose to medium dense (alluvium) |
| | | | | | | | 35 | | | | |

| | | | | |
|---|---|---|---|---|
| Date Started: | July 9, 2001 | Water Level: | 6.3 ft. 7/9/01 1310 HRS | Plate |
| Date Completed: | July 11, 2001 | | | |
| Logged By: | E. Shinsato | Drill Rig: | CONCORE | A - 1.1 |
| Total Depth: | 81.5 feet | Drilling Method: | 4" Auger & 4" Casing | |
| Work Order: | 4515-00 | Driving Energy: | 140 lb. wt., 30 in. drop | |

BORING_LOG 4515-00.GPJ GEOLABS.GDT 5/1/09

| GEOLABS, INC. Geotechnical Engineering | CASTLE HILLS ACCESS ROAD DRAINAGE IMPROVEMENTS PROJECT NO. HWY-0-04-98 KANEOHE, OAHU, HAWAII | Log of Boring 1 |
|---|---|---|

(Continued from previous plate)

| Laboratory: Other Tests | Laboratory: Moisture Content (%) | Laboratory: Dry Density (pcf) | Field: Core Recovery (%) | Field: RQD (%) | Field: Penetration Resistance (blows/foot) | Field: Pocket Pen. (tsf) | Depth (feet) | USCS | Description |
|---|---|---|---|---|---|---|---|---|---|
| | 70 | | | | 8 | | | SW | grades to orange-brown with black and gray mottling with some sub-rounded gravel, loose |
| | 32 | | | | 12 | | 40 | | grades to dark grayish brown, loose to medium dense |
| | 42 | | | | 8 | | 45 | | grades to orange-grayish brown |
| LL=107 PI=72 | 37 | 75 | | | 13 | 0.8 | 50 | CH | Orange-grayish brown **CLAY** with traces of highly weathered gravel and fine sand (basaltic), medium stiff (residual soil) |
| | 95 | | | | 13 | 0.8 | 55 | | grades to orange-brown with black and gray mottling, stiff |
| | 75 | 54 | | | 48 | | 60 | | grades to very stiff |
| | | | | | | | | SW-SM | Grayish brown with red mottling **SAND** with silt and highly weathered gravel (basaltic), medium dense (residual soil) |
| | 78 | | | | 15 | | 65 | | grades to orange-brown with gray seams |
| | | | | | | | 70 | | |

| | | | | |
|---|---|---|---|---|
| Date Started: | July 9, 2001 | Water Level: ▽ | 6.3 ft. 7/9/01 1310 HRS | Plate A - 1.2 |
| Date Completed: | July 11, 2001 | | | |
| Logged By: | E. Shinsato | Drill Rig: | CONCORE | |
| Total Depth: | 81.5 feet | Drilling Method: | 4" Auger & 4" Casing | |
| Work Order: | 4515-00 | Driving Energy: | 140 lb. wt., 30 in. drop | |

| GEOLABS, INC. Geotechnical Engineering | CASTLE HILLS ACCESS ROAD<br>DRAINAGE IMPROVEMENTS<br>PROJECT NO. HWY-0-04-98<br>KANEOHE, OAHU, HAWAII | Log of Boring 1 |
|---|---|---|

| Laboratory: Other Tests | Laboratory: Moisture Content (%) | Laboratory: Dry Density (pcf) | Field: Core Recovery (%) | Field: RQD (%) | Field: Penetration Resistance (blows/foot) | Field: Pocket Pen. (tsf) | Depth (feet) | Sample | Graphic | USCS | Description (Continued from previous plate) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | 53 | 69 | | | 23 | | | | | SW-SM | |
| | 21 | | | | 35 | | 75 | | | GM | Grayish brown with orange-brown seams **SILTY GRAVEL (BASALTIC)**, medium dense to dense |
| | 48 | 77 | | | 64 | | 80 | | | | |
| | | | | | | | 85 | | | | Boring terminated at 81.5 feet<br>* Elevations estimated from Topographic Map provided by ParEn, Inc. dated May 25, 2005. |
| | | | | | | | 90 | | | | |
| | | | | | | | 95 | | | | |
| | | | | | | | 100 | | | | |
| | | | | | | | 105 | | | | |

| | | | | |
|---|---|---|---|---|
| Date Started: | July 9, 2001 | Water Level: ⊻ | 6.3 ft. 7/9/01 1310 HRS | Plate |
| Date Completed: | July 11, 2001 | | | |
| Logged By: | E. Shinsato | Drill Rig: | CONCORE | |
| Total Depth: | 81.5 feet | Drilling Method: | 4" Auger & 4" Casing | A - 1.3 |
| Work Order: | 4515-00 | Driving Energy: | 140 lb. wt., 30 in. drop | |

| GEOLABS, INC. Geotechnical Engineering | CASTLE HILLS ACCESS ROAD DRAINAGE IMPROVEMENTS PROJECT NO. HWY-0-04-98 KANEOHE, OAHU, HAWAII | Log of Boring 2 |
|---|---|---|

Approximate Ground Surface Elevation (feet MSL): 173 *

| Other Tests (Laboratory) | Moisture Content (%) (Laboratory) | Dry Density (pcf) (Laboratory) | Core Recovery (%) (Field) | RQD (%) (Field) | Penetration Resistance (blows/foot) (Field) | Pocket Pen. (tsf) (Field) | Depth (feet) | USCS | Description |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | 0 | MH | Brown **CLAYEY SILT** with some gravel and sand, soft, moist |
| | 40 | 75 | | | 8 | 0.8 | | CH | Brown with multi-color mottling **CLAY** with gravel and traces of roots, slight organic odor, soft to medium stiff, very moist |
| LL=99 PI=63 | 51 | | | | 8 | | | | |
| | 46 | 68 | | | 20 | 1.0 | 5 | | |
| | 123 | | | | 3 | | 10 | SM | Dark grayish brown **SILTY SAND** with gravel and organic matter, very loose (peat) |
| | 42 | 68 | | | 5 | | 15 | GM | Dark grayish brown **SILTY GRAVEL AND SAND** with some organic matter, very loose |
| | 67 | | | | 1 | | 20 | SM | Grayish brown **SILTY SAND AND GRAVEL** with clay seams and traces of organic matter, very loose (alluvium) |
| | 46 | 72 | | | 15 | | 25 | | grades to orange-grayish brown, medium dense |
| | 38 | | | | 25 | | 30 | | |
| | | | | | | | 35 | | |

| | | | |
|---|---|---|---|
| Date Started: | July 12, 2001 | Water Level: ⊻ | Not Available |
| Date Completed: | July 16, 2001 | | |
| Logged By: | E. Shinsato | Drill Rig: | CONCORE |
| Total Depth: | 70.3 feet | Drilling Method: | 4" Auger & 4" Casing |
| Work Order: | 4515-00 | Driving Energy: | 140 lb. wt., 30 in. drop |

Plate A - 2.1

BORING_LOG 4515-00.GPJ GEOLABS.GDT 4/20/09

| GEOLABS, INC. Geotechnical Engineering | CASTLE HILLS ACCESS ROAD DRAINAGE IMPROVEMENTS PROJECT NO. HWY-0-04-98 KANEOHE, OAHU, HAWAII | Log of Boring 2 |
|---|---|---|

(Continued from previous plate)

| Laboratory | | | Field | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Other Tests | Moisture Content (%) | Dry Density (pcf) | Core Recovery (%) | RQD (%) | Penetration Resistance (blows/foot) | Pocket Pen. (tsf) | Depth (feet) | Sample | Graphic | USCS | Description |
| | 19 | 97 | | | 81 | | | | | SM | grades to dense |
| | 41 | | | | 35 | | 40 | | | | |
| LL=69 PI=37 | 69 | 57 | | | 10 | | 45 | | | CH | Grayish brown with multi-color mottling **CLAY** with sand and gravel, soft to medium stiff (saprolite) |
| | 103 | | | | 7 | | 50 | | | SM | Orange-grayish brown **SILTY SAND** with some sub-rounded gravel and traces of clay seams, loose |
| | 76 | 54 | | | 34 | | 55 | | | | grades to medium dense |
| | 59 | | | | 67 | | 60 | | | | grades to dense to very dense<br>grades with some cobbles and boulders, very dense |
| | 38 | | | | 60/5" Ref. | | 65 | | | | |
| | | | | | | | 70 | | | | |

| | | | |
|---|---|---|---|
| Date Started: | July 12, 2001 | Water Level: ⊻ | Not Available |
| Date Completed: | July 16, 2001 | | |
| Logged By: | E. Shinsato | Drill Rig: | CONCORE |
| Total Depth: | 70.3 feet | Drilling Method: | 4" Auger & 4" Casing |
| Work Order: | 4515-00 | Driving Energy: | 140 lb. wt., 30 in. drop |

Plate A - 2.2

BORING_LOG 4515-00.GPJ GEOLABS.GDT 4/20/09

| GEOLABS, INC. Geotechnical Engineering | CASTLE HILLS ACCESS ROAD DRAINAGE IMPROVEMENTS PROJECT NO. HWY-0-04-98 KANEOHE, OAHU, HAWAII | Log of Boring 2 |
|---|---|---|

| Laboratory | | | Field | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Other Tests | Moisture Content (%) | Dry Density (pcf) | Core Recovery (%) | RQD (%) | Penetration Resistance (blows/foot) | Pocket Pen. (tsf) | Depth (feet) | Sample | Graphic | USCS | (Continued from previous plate) Description |
| | | | | | 50/3" Ref. | | | | | | Boring terminated at 70.3 feet |
| | | | | | | | 75 | | | | |
| | | | | | | | 80 | | | | |
| | | | | | | | 85 | | | | |
| | | | | | | | 90 | | | | |
| | | | | | | | 95 | | | | |
| | | | | | | | 100 | | | | |
| | | | | | | | 105 | | | | |

| | | | | |
|---|---|---|---|---|
| Date Started: | July 12, 2001 | Water Level: ∇ | Not Available | Plate A - 2.3 |
| Date Completed: | July 16, 2001 | | | |
| Logged By: | E. Shinsato | Drill Rig: | CONCORE | |
| Total Depth: | 70.3 feet | Drilling Method: | 4" Auger & 4" Casing | |
| Work Order: | 4515-00 | Driving Energy: | 140 lb. wt., 30 in. drop | |

BORING_LOG 4515-00.GPJ GEOLABS.GDT 4/20/09

# APPENDIX B

# APPENDIX B

Geotechnical Laboratory Testing

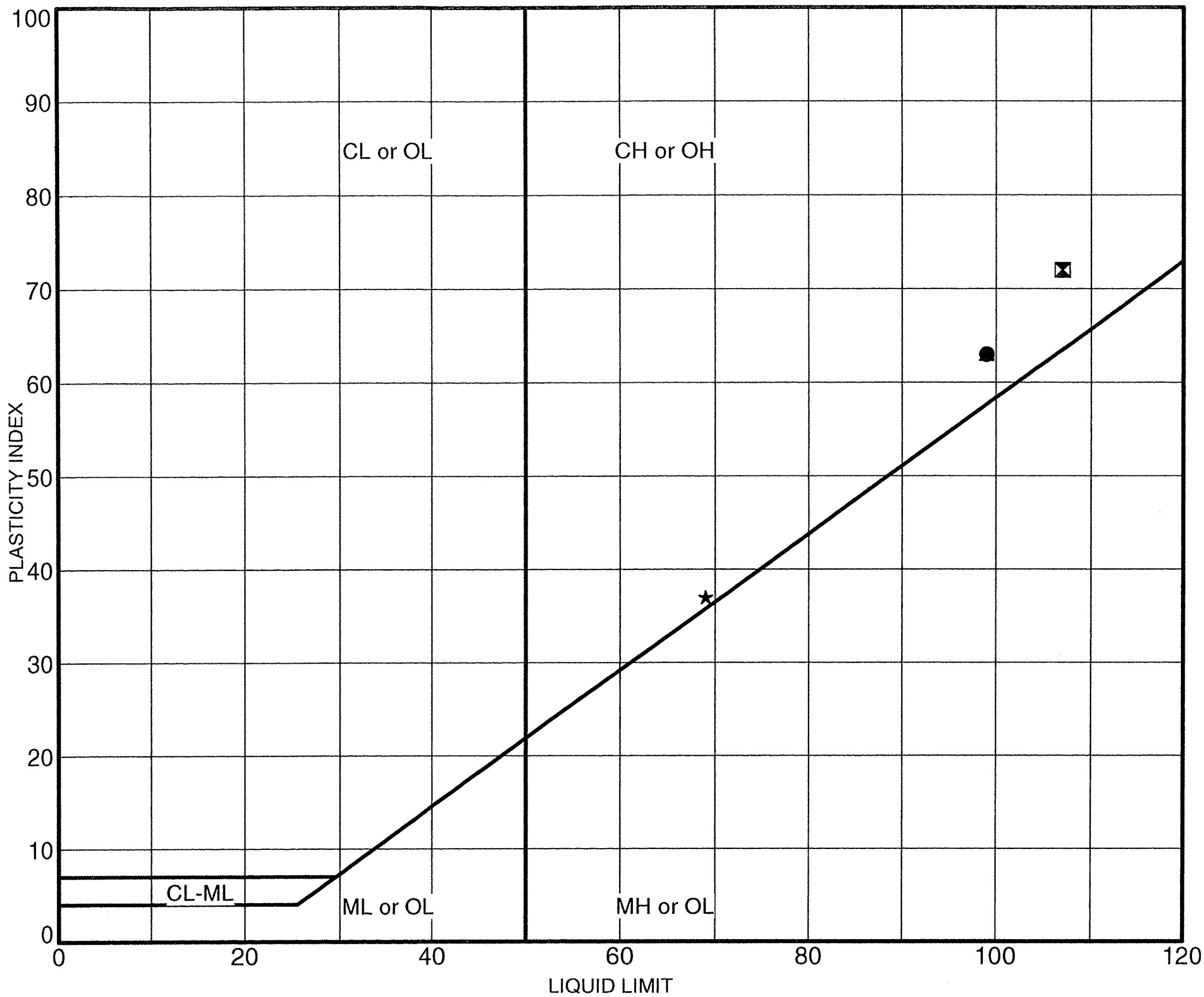
Moisture Content (ASTM D 2216) and Unit Weight (ASTM D 2937) determinations were performed on selected soil samples as an aid in the classification and evaluation of soil properties. The test results are presented on the Logs of Borings at the appropriate sample depths.

Four Atterberg Limits tests (ASTM D 4318) were performed on selected soil samples to evaluate the liquid and plastic limits and to aid in soil classification. The test results are summarized on the Logs of Borings at the appropriate sample depths. Graphic presentation of the test results is provided on Plate B-1.

Three Sieve Analysis tests (ASTM C 117 & C 136) were performed on selected soil samples to evaluate the gradation characteristics of the soils and to aid in soil classification. Graphic presentations of the grain size distribution are presented on Plate B-2.

One Consolidation test (ASTM D 2435) was performed on a sample of the soft compressible soils to evaluate the compressibility characteristics of the materials encountered. Consolidation test results are presented on Plate C-3.

One Unconsolidated Undrained Triaxial Compression (TXUU) test (ASTM D 2850) was performed on a selected soil sample to evaluate the undrained shear strength of the silty and clayey soils encountered. The approximate in-situ effective overburden pressure was used as the applied confining pressure for the relatively "undisturbed" soil sample. The test results and the stress-strain curve are presented on Plate B-4.




| | Sample | Depth (ft) | LL | PL | PI | Description |
|---|---|---|---|---|---|---|
| ● | B-1 | 5.0-6.5 | 99 | 36 | 63 | Dark gray clay (OH) |
| ⧗ | B-1 | 50.0-51.5 | 107 | 35 | 72 | Gray clay (CH) |
| ▲ | B-2 | 2.5-4.0 | 99 | 36 | 63 | Grayish brown clay (CH) |
| ★ | B-2 | 45.0-46.5 | 69 | 32 | 37 | Gray clay (CH) |

**GEOLABS, INC.**
GEOTECHNICAL ENGINEERING

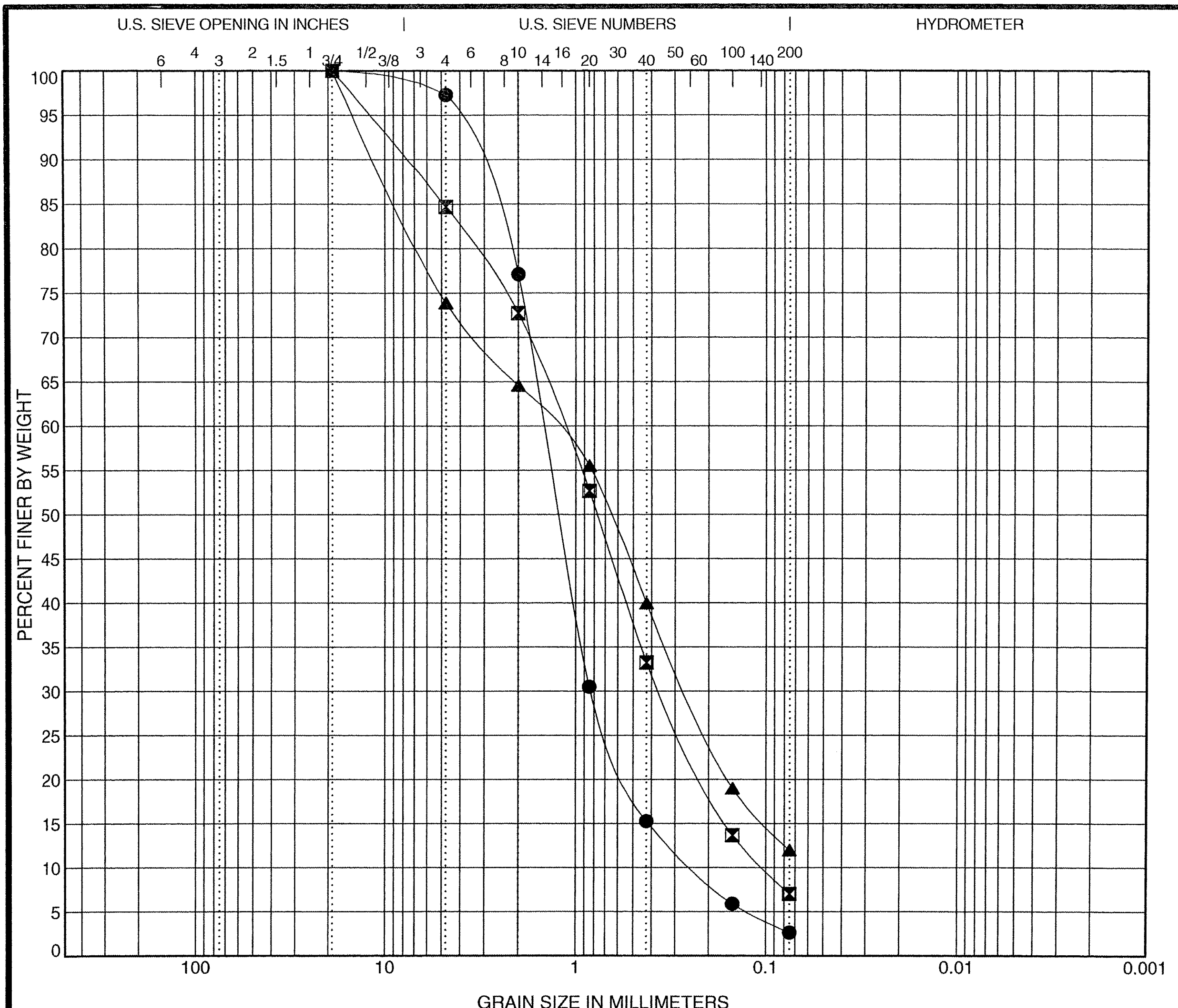
W.O. 4515-00

**ATTERBERG LIMITS TEST RESULTS - ASTM D 4318**

CASTLE HILLS ACCESS ROAD
DRAINAGE IMPROVEMENTS
PROJECT NO. HWY-0-04-98
KANEOHE, OAHU, HAWAII

Plate
B - 1

G_ATTERBERG 4515-00.GPJ GEOLABS.GDT 4/20/09




| COBBLES | GRAVEL | | SAND | | | SILT OR CLAY |
|---|---|---|---|---|---|---|
| | coarse | fine | coarse | medium | fine | |

| | Sample | Depth (ft) | Description | LL | PL | PI | Cc | Cu |
|---|---|---|---|---|---|---|---|---|
| ● | B-1 | 30.0-31.5 | Brown well-graded sand (SW) | | | | 2.0 | 6.2 |
| ⊠ | B-1 | 70.0-71.5 | Brown well-graded sand (SW-SM) w/ silt & gravel | | | | 1.1 | 11.3 |
| ▲ | B-2 | 20.0-21.5 | Gray silty sand (SM) w/ gravel | | | | 0.8 | 21.0 |

| | Sample | Depth (ft) | D100 (mm) | D60 (mm) | D30 (mm) | D10 (mm) | %Gravel | %Sand | %Fine |
|---|---|---|---|---|---|---|---|---|---|
| ● | B-1 | 30.0-31.5 | 19 | 1.461 | 0.831 | 0.237 | 2.7 | 94.7 | 2.6 |
| ⊠ | B-1 | 70.0-71.5 | 19 | 1.162 | 0.358 | 0.103 | 15.3 | 77.7 | 7.0 |
| ▲ | B-2 | 20.0-21.5 | 19 | 1.3 | 0.259 | | 26.1 | 61.9 | 12.0 |

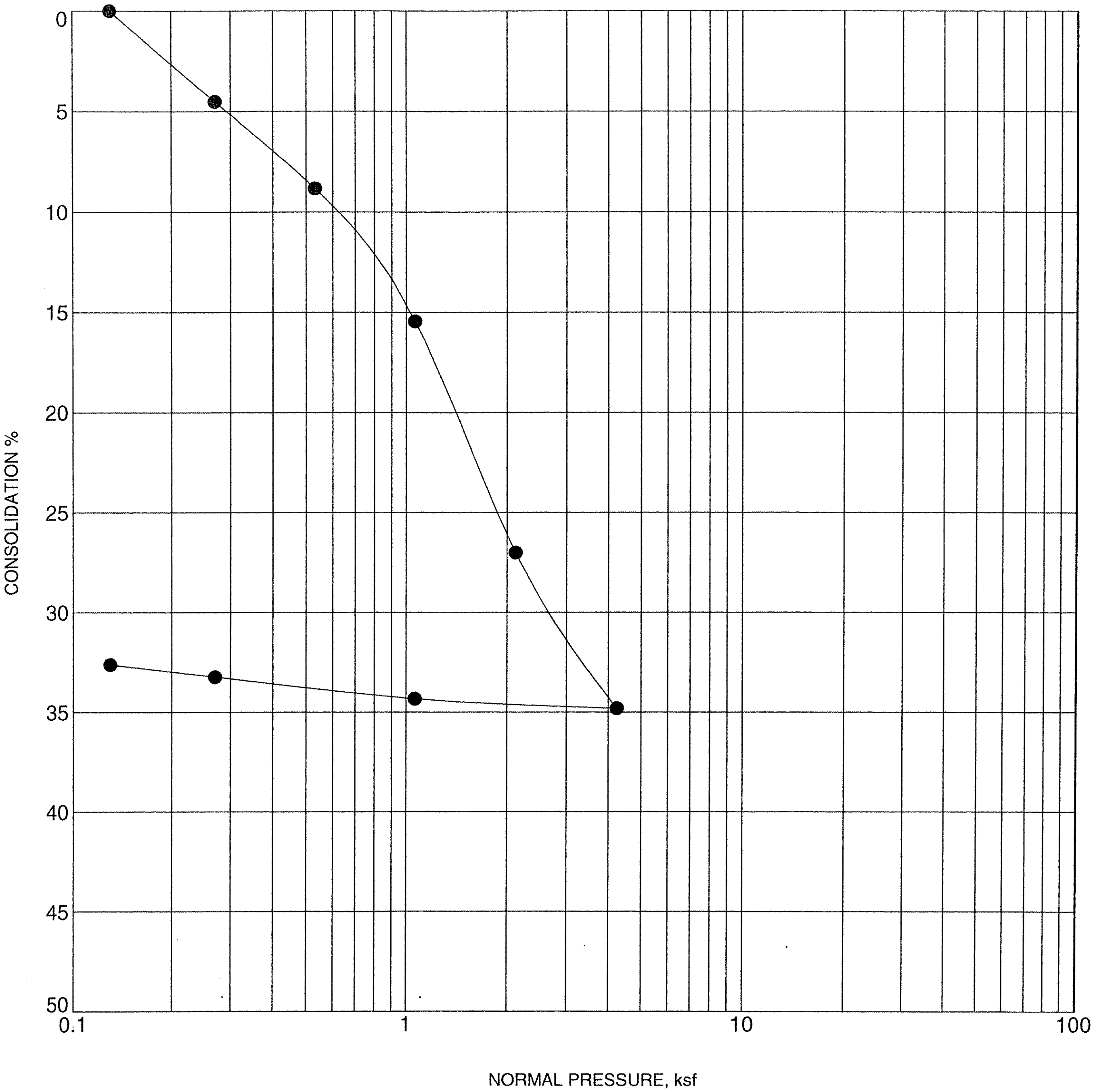
**GEOLABS, INC.**
GEOTECHNICAL ENGINEERING
W.O. 4515-00

**GRAIN SIZE DISTRIBUTION - ASTM C 117 & C 136**

CASTLE HILLS ACCESS ROAD
DRAINAGE IMPROVEMENTS
PROJECT NO. HWY-0-04-98
KANEOHE, OAHU, HAWAII

Plate
B - 2

G_GRAIN SIZE 4515-00.GPJ GEOLABS.GDT 5/1/09




Sample: B-1
Depth: 15.0 - 16.5 feet
Description: Dark brown clay w/ traces of roots and organic matter

Liquid Limit = N/A Plasticity Index = N/A

| | Initial | Final |
|---|---|---|
| Water Content, % | 22.6 | 59.1 |
| Dry Density, pcf: | 54.5 | 80.8 |
| Void Ratio | 5.336 | 3.269 |
| Degree of Saturation, % | 23.4 | 100.0 |
| Sample Height, inches | 1.0000 | 0.6140 |

G_CONSOL 4515-00.GPJ GEOLABS.GDT 5/1/09

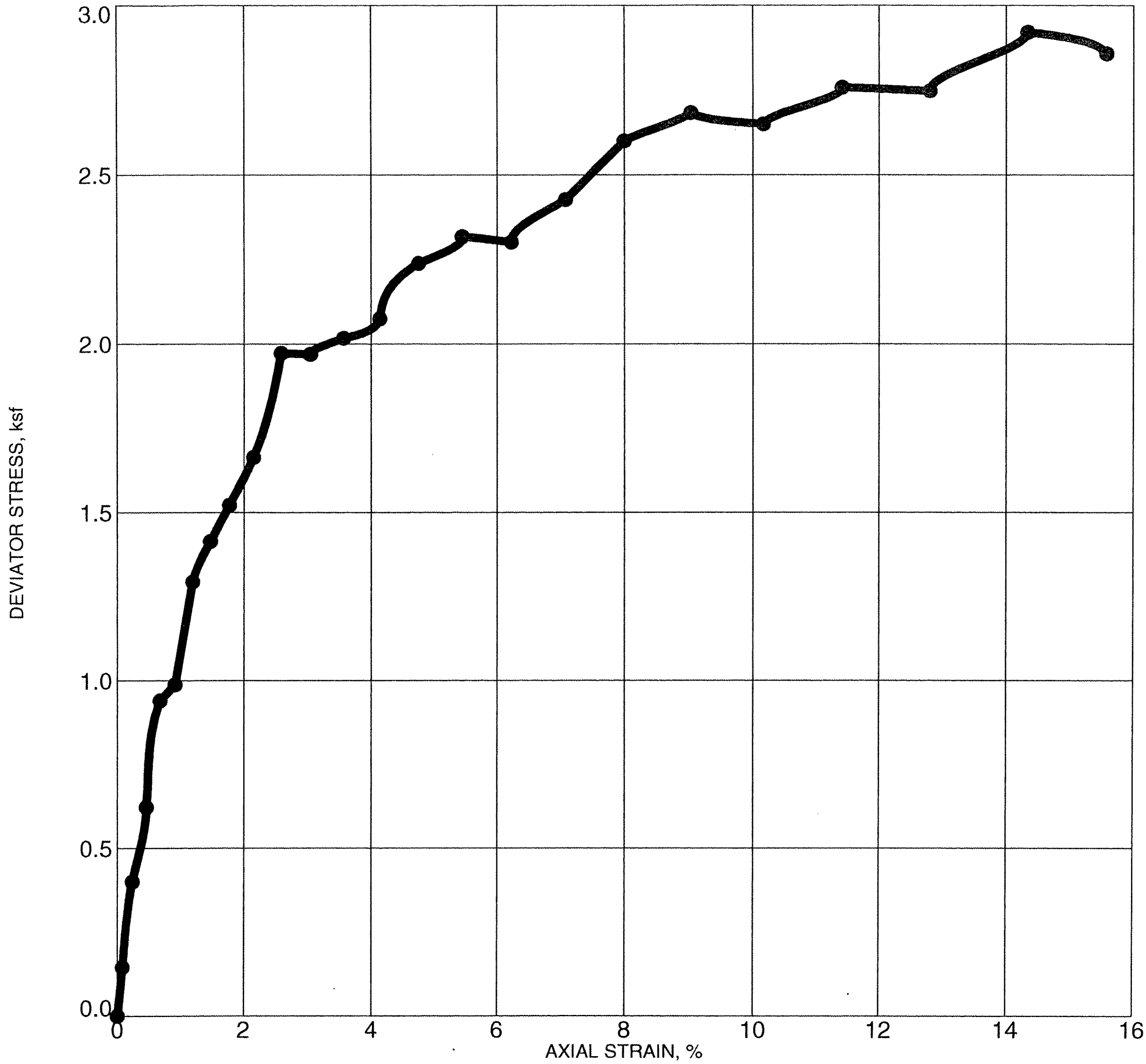
GEOLABS, INC.
GEOTECHNICAL ENGINEERING

W.O. 4515-00

**CONSOLIDATION TEST - ASTM D 2435**

CASTLE HILLS ACCESS ROAD
DRAINAGE IMPROVEMENTS
PROJECT NO. HWY-0-04-98
KANEOHE, OAHU, HAWAII

Plate
B - 3




| | |
|---|---|
| Max. Deviator Stress (ksf): | 2.9 |
| Confining Stress (ksf): | 0.6 |

Location: B-2
Depth: 5.0 - 6.5 feet
Description: Dark gray silty clay
Test Date: 4/20/2009

| | | | |
|---|---|---|---|
| Dry Density (pcf) | 66.7 | Sample Diameter (inches) | 2.411 |
| Moisture (%) | 50.8 | Sample Height (inches) | 5.029 |
| Axial Strain at Failure (%) | 14.3 | Strain Rate (% / minute) | 0.62 |

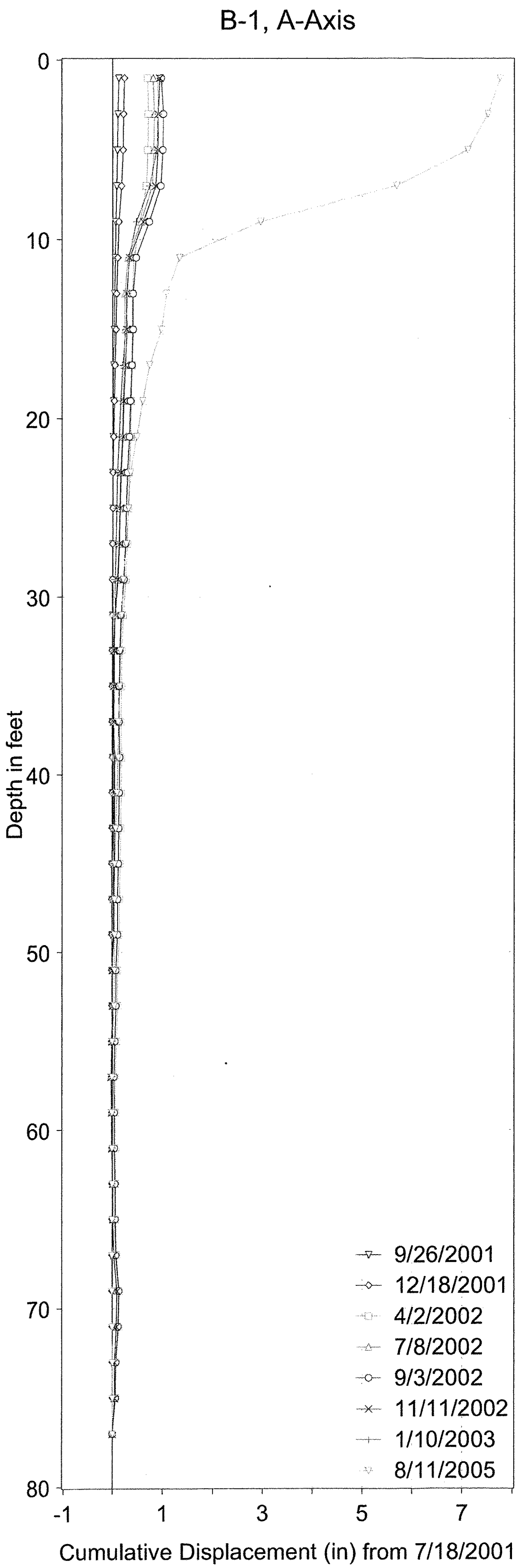
**GEOLABS, INC.**
GEOTECHNICAL ENGINEERING
W.O. 4515-00

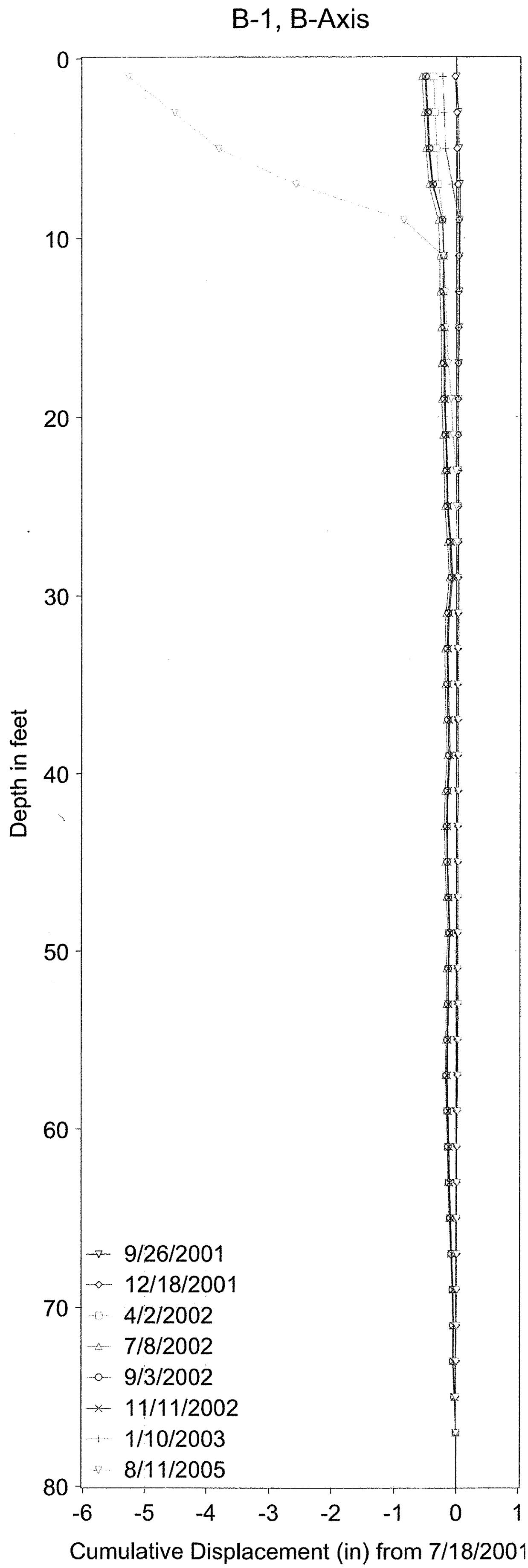
**TRIAXIAL UU COMPRESSION TEST - ASTM D 2850**

CASTLE HILLS ACCESS ROAD
DRAINAGE IMPROVEMENTS
PROJECT NO. HWY-0-04-98
KANEOHE, OAHU, HAWAII

Plate
B - 4

# APPENDIX C


B-1, A-Axis
Depth in feet
Cumulative Displacement (in) from 7/18/2001
9/26/2001
12/18/2001
4/2/2002
7/8/2002
9/3/2002
11/11/2002
1/10/2003
8/11/2005


B-1, B-Axis
Depth in feet
Cumulative Displacement (in) from 7/18/2001
9/26/2001
12/18/2001
4/2/2002
7/8/2002
9/3/2002
11/11/2002
1/10/2003
8/11/2005