

**GEOTECHNICAL ENGINEERING EXPLORATION  
FOREST CITY MARINE BASE HOUSING OFFICE  
KANEOHE BAY, OAHU, HAWAII**

**NOVEMBER 10, 2009**

*Prepared for*  
**FOREST CITY MILITARY COMMUNITIES**



**GEOLABS, INC.**

*Geotechnical Engineering and Drilling Services*

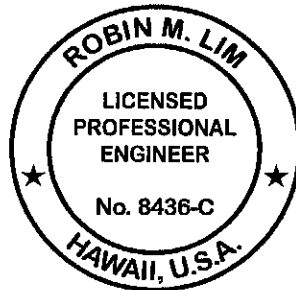
W.O. 5884-00(D)

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THIS WORK WAS PREPARED BY  
ME OR UNDER MY SUPERVISION.

A handwritten signature in black ink, appearing to read "Robin M. Lim".

SIGNATURE

4-30-10  
EXPIRATION DATE  
OF THE LICENSE



**GEOLABS, INC.**  
Geotechnical Engineering and Drilling Services  
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Hawaii • California



## **GEOLABS, INC.**

*Geotechnical Engineering and Drilling Services*

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November 10, 2009  
W.O. 5884-00(D)

**Mr. Kenneth Rappolt**  
**Forest City Military Communities Hawaii**  
5173 Nimitz Road  
Honolulu, HI 96818

Dear **Mr. Rappolt**:

Geolabs, Inc. is pleased to submit our report entitled "Geotechnical Engineering Exploration, Forest City Marine Base Housing Office, Kaneohe Bay, Oahu, Hawaii" prepared in support of the design of the new housing office project.

Our work was performed in general accordance with the scope of services outlined in our revised fee proposal dated July 27, 2009.

Please note that the soil samples recovered during our field exploration (remaining after testing) will be stored for a period of two months from the date of this report. The samples will be discarded after that date unless arrangements are made for a longer sample storage period. Please contact our office for alternative sample storage requirements, if appropriate.

Detailed discussion and specific design recommendations 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.**

**Robin M. Lim, P.E.**  
Vice President

RML:TK:mj

**GEOTECHNICAL ENGINEERING EXPLORATION  
FOREST CITY MARINE BASE HOUSING OFFICE  
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W.O. 5884-00(D) NOVEMBER 10, 2009**

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**GEOTECHNICAL ENGINEERING EXPLORATION  
FOREST CITY MARINE BASE HOUSING OFFICE  
KANEHOE BAY, OAHU, HAWAII  
W.O. 5884-00(D) NOVEMBER 10, 2009**

**SUMMARY OF FINDINGS AND RECOMMENDATIONS**

Based on the borings drilled, the majority of the project site generally is underlain by dense to very dense silty gravel fills overlying soft silty clays extending to depths of about 10 feet below the existing ground surface. The compressible soil deposits were underlain by loose to dense gravelly sands overlying very stiff to hard clayey/sandy silts extending to the maximum depth explored of approximately 26.5 feet below the existing ground surface. Our field exploration toward the eastern portion of the project site generally encountered hard coral formation at shallow depths overlying hard to very hard clayey silts extending to the maximum depth explored of about 16 feet below the existing ground surface. We encountered groundwater in the borings at depths ranging from about 5.5 to 7 feet below the existing ground surface.

Based on the subsurface conditions encountered and the structural loads provided for the project, we believe slabs-on-grade with thickened-edge footings may be used to support the new housing office building planned. In general, we recommend using an allowable bearing pressure of up to about 2,500 psf for the design of shallow foundations bearing on the recompacted on-site soils. The perimeter thickened-edges of the slab should be embedded a minimum of 18 inches (and a maximum of 30 inches due to the presence of soft compressible soils underlying the project site) below the lowest adjacent exterior grade. We also recommend the concrete slab be at least 5 inches thick and lightly reinforced with No. 3 reinforcing bars spaced at 12 inches in each direction, in lieu of the standard welded wire fabric.

Our laboratory tests indicate the near-surface soils at the project site exhibit a slight to low shrink/swell potential when subjected to fluctuations in the soil moisture content. In addition, our experience in the project vicinity suggests that expansive soils could be present in localized areas within the premises. To reduce the potential for future distress to the lightly loaded slabs-on-grade resulting from shrinking and swelling of the expansive clayey soils, we recommend properly preparing and capping subgrade soils with a 12-inch thick layer of non-expansive select granular fill materials.

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

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END OF SUMMARY OF FINDINGS AND RECOMMENDATIONS

## SECTION 1. GENERAL

### 1.1 Introduction

This report presents the results of our geotechnical engineering exploration performed in support of the *Forest City Marine Base Housing Office* project within the Marine Corps Base Hawaii (MCBH) at Kaneohe Bay 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 design of foundations and slabs, retaining structures, site grading, and pavements only. The findings and recommendations presented herein are subject to the limitations noted at the end of this report.

### 1.2 Project Considerations

Based on the information provided, the proposed project involves construction of a new housing office within the MCBH at Kaneohe Bay on the Island of Oahu, Hawaii. The project site is presently occupied by the housing units designated as 2511 and 2512 on Lawrence Road at the time of our field exploration.

The new housing office will be a two-story structure providing a total floor area of approximately 8,000 square feet. We envision the new housing office will be a pre-engineered rigid steel framed building supported by concrete slabs-on-grade with thickened-edge footings. Based on the structural loading information provided, the maximum dead and live loads at the column locations are estimated to be on the order of about 32 and 60 kips, respectively.

The finished floor elevation of the new housing office building is set at about +8.63 feet Mean Sea Level (MSL). Based on the topographic survey, the existing ground surface elevation within the housing office building footprint is between about +6.7 and +7.9 feet MSL. Therefore, we anticipate some site grading work consisting of fills on the order of about 1 to 2 feet will be required to achieve the design finished grades for the housing office structure.

### **1.3 Purpose and Scope**

The purpose of our geotechnical engineering exploration was to obtain an overview of the surface and subsurface conditions to develop a soil and/or rock data set to formulate geotechnical recommendations for design of the *Forest City Marine Base Housing Office* project. In order to accomplish this, we conducted an exploration program consisting of the following tasks and work efforts:

1. Review of available in-house soil and geologic information from the project vicinity.
2. Application of a Digging Work Clearance Request prior to our field exploration work.
3. Preparation and submittal of an accident prevention plan with activity hazard analysis in support of our field exploration activities.
4. Mobilization and demobilization of truck-mounted drill rig, portable drilling equipment, and two operators to the project site and back.
5. Drilling and sampling of five borings extending to depths ranging from about 11.5 to 26.5 feet below the existing ground surface for a total of approximately 96.8 lineal feet of exploration.
6. Coordination of the field exploration and logging of the borings by our field engineer.
7. Laboratory testing of selected soil samples obtained during the field exploration as an aid in classifying the materials and evaluating their engineering properties.
8. Analyses of the field and laboratory data to formulate geotechnical recommendations for design of foundations and slabs, retaining structures, site grading, and pavements.
9. Preparation of this report summarizing our work on the project and presenting our findings and recommendations.
10. Coordination of our overall work on the project by our senior engineer.
11. Quality assurance of our work and client/design team consultation by our principal engineer.
12. Miscellaneous work efforts such as drafting, word processing, and clerical support.

SECTION 1. GENERAL

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Detailed descriptions of our field exploration methodology and the Logs of Borings are presented in Appendix A. Results of the laboratory tests performed on selected soil samples are presented in Appendix B.

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END OF GENERAL

## SECTION 2. SITE CHARACTERIZATION

### 2.1 Regional Geology

The Island of Oahu was built by the extrusion of basaltic lavas from the Waianae and Koolau Shield Volcanoes. The older Waianae Volcano is estimated to be middle to late Pliocene in age and forms the bulk of the western one-third of the island. The younger Koolau Volcano is estimated to be late Pliocene to early Pleistocene (Ice Age) in age and forms the majority of the eastern two-thirds of the island. Waianae Volcano became extinct while Koolau Volcano was still active, and its eastern flank was partially buried below Koolau lavas banking against its eastern flank. The Koolau Volcano reached the end of its main shield-building phase about 2 million years ago.

The project site is on the southern flank of the Koolau Volcano and is composed of basaltic rock built by the extrusion of lavas of the Koolau Volcanic Series. These rocks are generally characterized by flows of jointed dense vesicular basalt with interbedded thin clinker layers. In-situ chemical weathering of the Koolau lava flows has occurred for the last 1 to 2 million years. The weathering process has formed a surface mantle of residual soils, which grade to saprolite with depth. In general, saprolite is composed mainly of silty material and is typical of the tropical weathering of volcanic rocks. Saprolite grades to basaltic rock with increasing depth.

### 2.2 Site Description

The project site is generally in the central portion of the Marine Corps Base Hawaii at Kaneohe Bay on the Island of Oahu, Hawaii. It is approximately bounded by Lawrence Road to the west, an existing stream and Mokapu Elementary School to the east, and residential dwellings to the south.

The project site is a triangular-shaped parcel occupied by the housing units designated as 2511 and 2512 Lawrence Road at the time of our field exploration. Based on the topographic survey map provided, the existing ground surface elevations within the housing office building footprint are between about +6.7 and +7.9 feet MSL.

### **2.3 Subsurface Conditions**

We explored the subsurface conditions by drilling and sampling five borings, designated as Boring Nos. 1 through 5, extending to depths ranging from about 11.5 to 26.5 feet below the existing ground surface. In addition, two bulk samples of the near-surface soils, designated as Bulk-1 and Bulk-2, were collected for laboratory CBR tests to evaluate the pavement support characteristics of the near-surface soils. The approximate boring and bulk sample locations are shown on the Site Plan, Plate 2.

Based on four of the borings drilled (Boring Nos. 2 through 5), the majority of the project site generally is underlain by a thin layer of top soil overlying dense to very dense silty gravel fill materials, on the order of about 4 to 5 feet thick, followed by soft silty clays extending to depths of about 10 feet below the existing ground surface. The compressible soil deposits were underlain by loose to dense gravelly sands overlying very stiff to hard clayey/sandy silts extending to the maximum depth explored of approximately 26.5 feet below the existing ground surface.

In Boring No. 1 (drilled toward the eastern portion of the project site), our field exploration generally encountered hard coral formation at shallow depths below the ground level extending to a depth of about 10 feet below the existing ground surface. Hard to very hard clayey silts were then encountered extending to the maximum depth explored of about 16 feet below the existing ground surface. The compressible soil deposits encountered in the other four borings were not encountered at this boring location.

We encountered groundwater in the borings at depths ranging from about 5.5 to 7 feet below the existing ground surface at the time of our field exploration. The groundwater levels measured during our field exploration correspond to approximately Elevations -1.1 to +1.4 feet MSL. Due to the proximity of the project site to the Pacific Ocean, water levels are expected to vary with tidal fluctuations and storm surge events. It should be noted that water levels also may vary with seasonal rainfall, time of the year, and other factors.

Descriptions and graphic representations of the materials encountered in the borings are presented on the Logs of Borings in Appendix A. Laboratory tests were performed on selected soil samples encountered, and the test results are presented in Appendix B.

#### **2.4 Earthquakes and Seismicity**

In general, earthquakes throughout the world are caused by shifts in the tectonic plates. In contrast, earthquake activity in Hawaii is linked primarily to volcanic activity; therefore, earthquake activity in Hawaii generally occurs before or during volcanic eruptions. In addition, earthquakes may result from the underground movement of magma that comes close to the surface but does not erupt. The Island of Hawaii experiences thousands of earthquakes each year, but most are so small that they can be detected only by sensitive instruments. However, some of the earthquakes are strong enough to be felt, and a few cause minor to moderate damage.

In general, earthquakes associated with volcanic activity are most common on the Island of Hawaii. Earthquakes that are directly associated with the movement of magma are concentrated beneath the active Kilauea and Mauna Loa Volcanoes on the Island of Hawaii. Because the majority of the earthquakes in Hawaii (over 90 percent) are related to volcanic activity, the risk of seismic activity and degree of ground shaking diminishes with increased distance from the Island of Hawaii. The Island of Hawaii has experienced numerous earthquakes greater than Magnitude 5 (M5+); however, earthquakes are not confined only to the Island of Hawaii.

To a lesser degree, the Island of Maui has experienced several earthquakes greater than Magnitude 5. Therefore, moderate to strong earthquakes have occurred in the County of Maui. The effects of earthquakes occurring on the Islands of Hawaii and Maui may be felt on the Island of Oahu. For example, small landslides occurred on the Island of Oahu as a result of the Maui Earthquake of 1938 (M6.8). Some houses on the Island of Oahu were reportedly damaged as a result of the Lanai Earthquake of 1871 (M7+).

In the last 150 years of recorded history, we are not aware of earthquakes greater than Magnitude 6 that have occurred on the Island of Oahu. An earthquake of Magnitude 4.8 to 5.0 occurred along the Diamond Head Fault in 1948 on the Island of Oahu. The moderate tremor resulted in broken store windows, ruptured building walls, and broken underground water mains.

## **2.5 Soil Profile Type for Seismic Design**

Our field exploration generally encountered dense silty gravel or hard coral formation overlying very stiff clayey silts extending to the maximum depth explored of approximately 26.5 feet below the existing ground surface. Based on the average penetration resistance (N-values) of the subsurface materials encountered below the foundation subgrade level and the geology of the area, we believe the project site may be classified as a "Stiff Soil Profile." Therefore, we believe the seismic analysis of the building structure may be designed based on a Site Class D soil profile based on the International Building Code (Table No. 1615.1.1), 2006 Edition.

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END OF SITE CHARACTERIZATION

### **SECTION 3. DISCUSSION AND RECOMMENDATIONS**

Based on the borings drilled, the majority of the project site generally is underlain by dense to very dense silty gravel fill overlying soft silty clays extending to depths of about 10 feet below the existing ground surface. The compressible soil deposits were underlain by loose to dense gravelly sands overlying very stiff to hard clayey/sandy silts extending to the maximum depth explored of approximately 26.5 feet below the existing ground surface. Our field exploration toward the eastern portion of the project site generally encountered hard coral formation at shallow depths overlying hard to very hard clayey silts extending to the maximum depth explored of about 16 feet below the existing ground surface. The compressible soil deposits encountered in the other four borings were not encountered at this boring location. We encountered groundwater in the borings at depths ranging from about 5.5 to 7 feet below the existing ground surface.

Based on the subsurface conditions encountered and the structural loads provided for the project, we recommend utilizing slabs-on-grade with thickened-edge footings to support the new office building planned. The perimeter thickened-edges of the slabs should be embedded a minimum of 18 inches (and a maximum of 30 inches due to the presence of soft compressible soils underlying the project site) below the lowest adjacent exterior grade. In addition, the base of the thickened edge should have a minimum width of 12 inches. We also recommend the concrete slab be at least 5 inches thick and lightly reinforced with No. 3 reinforcing bars spaced at 12 inches in each direction, in lieu of the standard welded wire fabric.

Our laboratory tests indicate the near-surface soils at the project site exhibit a slight to low shrink/swell potential when subjected to fluctuations in the soil moisture content. In addition, our experience in the project vicinity suggests that expansive soils could be present in localized areas within the premises. To reduce the potential for future distress to the lightly loaded slabs-on-grade resulting from shrinking and swelling of the expansive clayey soils, we recommend properly preparing and capping the subgrade soils with a 12-inch thick layer of non-expansive select granular fill materials. The non-expansive select granular fill should extend a minimum of 2 feet beyond the

edges of the structure. Prior to placement of the non-expansive select granular fill, the subgrade soils should be scarified to a depth of about 8 inches, moisture-conditioned to about 2 percent above the optimum moisture content, and compacted to a minimum of 90 percent relative compaction. The non-expansive select granular fill also should be compacted to a minimum of 90 percent relative compaction.

We understand flexible pavements will be utilized at the project site to serve the new housing office. A flexible pavement structural section consisting of 2 inches of asphaltic concrete placed on 6 inches of aggregate base course over 6 inches of aggregate subbase may be used for design of the parking lot area. Detailed discussion of these items and our geotechnical recommendations are presented in the following sections herein.

### **3.1 Slab-On-Grade Foundations**

Based on the subsurface conditions encountered and the structural loads provided for the project, we recommend using slabs-on-grade with thickened-edge footings to support the new office building planned for the project. The perimeter thickened-edges of the slab should be embedded a minimum of 18 inches (and a maximum of 30 inches due to the presence of soft compressible soils underlying the project site) below the lowest adjacent exterior grade. In addition, the base of the thickened edge footing should have a minimum width of 12 inches. We also recommend the concrete slab be at least 5 inches thick and lightly reinforced with No. 3 reinforcing bars spaced at 12 inches in each direction, in lieu of the standard welded wire fabric.

In general, we recommend utilizing an allowable bearing pressure of up to about 2,500 pounds per square foot (psf) for the design of shallow foundations bearing on the recompacted on-site soils. This bearing value is for supporting dead-plus-live loads and may be increased by one-third (1/3) when considering transient loads, such as those caused by wind or seismic forces.

Lateral loads acting on the structure may be resisted by friction between the bottom of the foundation and the bearing soil and by passive earth pressure acting against the near-vertical faces of the foundation system. A coefficient of friction of about

0.35 may be used for footings bearing on the recompacted on-site soils. Resistance due to passive earth pressure may be estimated using an equivalent fluid pressure of about 350 pounds per square foot per foot of depth (pcf). This assumes that the soils around the footings are well compacted (minimum of 90 percent relative compaction). Unless covered by pavements or slabs, the passive pressure resistance in the upper 12 inches of soil should be neglected.

In general, the bottom of footings should bear on the stiff or dense near-surface soils. The bottom of the footing excavations should be recompacted to at least 95 percent relative compaction to provide a relatively firm and smooth bearing surface prior to the placement of reinforcing steel or concrete. Soft and/or loose materials encountered at the bottom of footing excavations should be over-excavated to expose firm materials. The over-excavation should be backfilled with the approved on-site soils compacted to a minimum of 95 percent relative compaction, or the footing bottom should be extended down to bear directly on the underlying competent materials. Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same soil established in accordance with ASTM D 1557. Optimum moisture is the water content (percentage by dry weight) corresponding to the maximum dry density.

Foundations located next to utility trenches or easements should be embedded below a 45-degree imaginary plane extending upward from the bottom edge of the utility trench or extended to a depth as deep as the inverts of the utility lines. This requirement is necessary to avoid surcharging adjacent below-grade structures with additional structural loads and to reduce the potential for appreciable foundation settlement.

If the shallow foundations are designed and constructed in strict accordance with our recommendations, we estimate the total settlements of footings to be on the order of about 1 inch. Differential settlements between adjacent footings supported on similar materials should be on the order of about 0.5 inches.

Our laboratory tests indicate the near-surface soils at the project site exhibit a slight to low shrink/swell potential when subjected to fluctuations in the soil moisture

content. In addition, our experience in the project vicinity suggests that expansive soils could be present in localized areas within the premises. To reduce the potential for future distress to the lightly loaded slabs-on-grade resulting from shrinking and swelling of the expansive clayey soils, we recommend properly preparing and capping the subgrade soils with a 12-inch thick layer of non-expansive select granular fill materials. The non-expansive select granular fill should extend a minimum of 2 feet beyond edge of the structure. Prior to placement of the non-expansive select granular fill, the subgrade soils should be scarified to a depth of about 8 inches, moisture-conditioned to about 2 percent above the optimum moisture content, and compacted to a minimum of 90 percent relative compaction. The non-expansive select granular fill also should be compacted to a minimum of 90 percent relative compaction.

For interior building slabs, which will not be subjected to vehicular traffic, we recommend providing a minimum 4-inch thick layer of cushion fill below the slab for uniform support. The cushion fill should consist of open-graded gravel (ASTM C 33, No. 67 gradation) and would serve as a capillary moisture break. To reduce the potential for future moisture infiltration and subsequent damage to floor coverings, an impervious moisture barrier is recommended on top of the gravel cushion layer.

In addition, we envision that exterior concrete walkways and exterior flatwork likely will be required for the proposed project. In general, we recommend providing a minimum 12-inch thick layer of select granular fill material below the exterior concrete slabs. The select granular fill material should be moisture-conditioned and compacted to a minimum of 90 percent relative compaction. The subgrade soils below the select granular fill should be scarified to a depth of about 8 inches, moisture-conditioned to about 2 percent above the optimum moisture content, and compacted to a minimum of 90 percent relative compaction. To reduce the potential for substantial shrinkage cracks in the slabs, crack control joints should be provided at intervals equal to the width of the walkways (or slabs) with expansion joints at right-angle intersections.

A Geolabs representative should observe subgrade preparation and footing excavations prior to the placement of reinforcing steel or concrete to confirm the foundation bearing conditions and the required embedment depths. Proper

moisture-conditioning and preparation of the subgrade soils is critical to the performance of the foundation and slab system.

### **3.2 Retaining Structures**

Based on the information provided, we understand a basement level is not planned for the proposed housing office building project. However, retaining structures, such as the elevator pit, should be designed to resist the lateral earth pressures due to the adjacent soils and surcharge effects.

#### **3.2.1 Elevator Pit**

Based on the information provided, we understand the new elevator pit will have concrete walls with a concrete mat foundation. An allowable bearing pressure of up to 1,000 psf may be used to design the concrete mat bearing on a stabilization layer as described below. This allowable bearing pressure is for dead-plus-live loads and may be increased by one-third ( $1/3$ ) for transient loads, such as temporary wind and/or seismic forces.

Based on the information provided, we anticipate the bottom of the new elevator pit will be embedded about 5 feet below the design finished floor elevation. Due to the presence of soft subsurface soils near the foundation subgrade level of the underground elevator pit, we recommend providing a stabilization layer consisting of 2 feet of No. 2 Rock (ASTM C 33, No. 4 gradation) wrapped in a woven geotextile fabric, such as Mirafi 500X or equivalent, below the concrete mat for uniform bearing support. The woven geotextile fabric would serve as a separation layer and provide additional strength to the underlying soft subgrade. In addition, the stabilization layer would also serve as a working platform during construction. In order to reduce the potential for significant settlement of the elevator pit structure, we recommend sewing (in lieu of overlapping) the woven geotextile fabric and utilizing light compaction equipment during installation of the elevator pit. In addition, the 2-foot thick stabilization layer should extend a minimum of 2 feet beyond the edges of the concrete mat.

Due to the shallow groundwater levels anticipated at the project site, the proposed elevator pit may be subjected to buoyant uplift forces. Resistance to uplift loads may be mobilized by the dead weight of the underground structure, the concrete mat, and the backfill material above the concrete mat. Contribution of dead weight from the backfill may be estimated using a unit weight of 110 pounds per cubic foot (pcf) above the groundwater table and 46 pcf below the groundwater table. For side shear between the structure and the backfill material, a nominal value of 100 psf may be used in design. For evaluating uplift forces due to hydrostatic pressures, a high groundwater level of Elevation +3 feet MSL may be used for design to take into consideration tidal fluctuations.

Settlements of the new elevator pit may result from compression of the soft soil layer due to the placement of the backfill material and/or disturbance of the soft subsurface soils during construction. We estimate the total settlement of the elevator pit structure to be on the order of about 1 to 1.5 inches. We believe that most (more than one-half) of the settlements should occur during the construction period.

### 3.2.2 Lateral Earth Pressures

The lateral earth pressures acting on the elevator pit walls will depend on the type of backfill used, the extent of backfill, and the compactive effort on the backfill material around the structure. The underground elevator pit should be designed to resist the lateral earth pressures due to the adjacent soil and surcharge effects. Based on the subsurface conditions encountered at the project site, we recommend using the following lateral earth pressures, expressed in equivalent fluid pressures of pounds per square foot per foot of depth (pcf), in the design of the elevator pit structure.

<b>LATERAL EARTH PRESSURES LEVEL GROUND CONDITIONS</b>		
<b><u>Groundwater Conditions</u></b>	<b><u>At-Rest</u> (pcf)</b>	<b><u>Passive</u> (pcf)</b>
Above Groundwater	55	350
Below Groundwater	89	200

The values provided above assume that the on-site granular soils or imported non-expansive, select granular fill materials will be used to backfill around the structure. It is assumed that the backfill around the elevator pit will be compacted to between 90 and 95 percent relative compaction. Over-compaction of the elevator pit backfill should be avoided.

Due to the proximity of the project site to the Pacific Ocean, we recommend using a static groundwater level of +3 feet MSL to design the retaining structures. Retaining structures that extend below Elevation +3 feet MSL should be designed based on the lateral earth pressures for the below groundwater condition presented in the table above.

Surcharge stresses due to areal surcharges, traffic loads, line loads, and point loads within a horizontal distance equal to the depth of the structure should be considered in the design. For uniform surcharge stresses imposed on the loaded side of the elevator pit, a rectangular distribution with uniform pressure equal to 46 percent of the vertical surcharge pressure acting over the entire depth of the elevator pit structure may be used in design. Additional analyses during design may be needed to evaluate the surcharge effects of point loads and line loads.

### **3.3 Site Grading**

In general, we envision the majority of the site grading will consist of minor cuts and fills on the order of up to about 1 to 2 feet thick to achieve the design finished grades for the building structure. The following grading items are addressed in the following subsections:

- Site Preparation
- Fills and Backfills
- Fill Placement and Compaction Requirements
- Excavation

A Geolabs representative should monitor the earthwork operations during construction to observe whether undesirable materials are encountered during the excavation process and to confirm whether the exposed soil and/or rock conditions are similar to those encountered in our field exploration.

### 3.3.1 Site Preparation

At the on-set of earthwork, areas within the contract grading limits should be cleared and grubbed thoroughly. Based on the exposed surface conditions at the project site, the existing structures and asphaltic concrete pavements within the contract grading limits should be demolished and removed at the on-set of earthwork. Deleterious and other unsuitable materials should be removed and disposed properly offsite to reduce the potential for contamination of the excavated materials.

After clearing and demolishing the project area, finished subgrades in cut areas and areas designated to receive fills should be scarified to a depth of about 8 inches, moisture-conditioned to at least 2 percent above the optimum moisture content, and compacted to a minimum of 90 percent relative compaction. Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same soil established in accordance with ASTM D 1557. Optimum moisture is the water content (percentage by dry weight) corresponding to the maximum dry density.

Where yielding areas or soft and/or loose areas are disclosed during clearing and grubbing operations or during the scarification process, the soft and/or loose materials should be over-excavated to expose firm and/or dense ground conditions, and the resulting excavation should be backfilled with well-compacted fill materials. Therefore, contract documents should include additive and deductive unit prices for over-excavation and compacted fill placement to account for variations in the over-excavation quantities.

### 3.3.2 Fills and Backfills

In general, the excavated on-site soils may be re-used as a source of general fill materials provided the materials are free of vegetation, deleterious materials, and clay lumps and rock fragments greater than 3 inches in maximum dimension. The on-site soils, if accepted for use as general fills, may be placed below the select granular fill capping layer below the building structure, underneath the pavement structural section, as part of the trench backfill, or in landscape areas.

Imported general fill materials should consist of soil materials with a maximum particle size of 3 inches or less with sufficient fines (between 10 and 60 percent particles passing the No. 200 sieve) to prevent the occurrence of voids in the compacted mass. In addition, general fill materials should have a CBR value of 8 or greater and a swell of 2 percent or less when tested in accordance with ASTM D 1883.

Select granular fills (required under the building footprint) should consist of select granular fill materials, such as crushed coralline or basaltic materials. The materials should be well graded from coarse to fine with particles no larger than 3 inches in largest dimension and should contain between 10 and 30 percent particles passing the No. 200 sieve. The materials should have a laboratory CBR value of 20 or more and should have a maximum swell of 1 percent or less.

Imported fill materials should be tested for conformance with these recommendations prior to delivery to the project site for the intended use. The excavated on-site coralline materials may be re-used as a source of select granular fills provided that they are processed to meet the above requirements either by earthwork equipment on the building pad or by crusher/screening equipment off the building pad.

We understand it may be desired to recycle some of the existing concrete slabs and/or asphaltic concrete (AC) pavements at the project site. In general, we believe the existing concrete slabs and AC pavements may be recycled and used as a

source of general fill materials and/or non-expansive, select granular fill from a technical perspective, provided that several provisions are followed.

Existing AC pavements intended for recycled use should be removed by cold-planing to a maximum particle size of 1 inch or less in lieu of conventional excavation methods to provide a relatively fine product. Existing concrete slabs and other concrete pieces should be crushed to less than 3 inches in maximum particle size for reuse as fill materials. Protruding reinforcing bars (rebars) and other materials besides concrete should be removed prior to use as fill materials. The concrete slabs and AC pavements should be processed to a well-graded material meeting the following gradation requirements.

SIZE	PERCENT PASSING
3-Inch	100
#4 Sieve	30 to 60
#200 Sieve	10 to 30

In addition, the processed materials should have a laboratory CBR value of 20 or more and should have a maximum swell of 1 percent or less.

We wish to point out that use of recycled concrete materials may have chemical and/or environmental impacts on the proposed development. We have experienced a “problem” in a couple of our prior projects that used recycled concrete materials under new asphaltic concrete pavements. In some cases, gases form from the deterioration of metallic objects within the recycled base, resulting in cracks and openings at the finished pavement surface. Although the exact cause of the pavement damage was not determined, the owner should be aware of the potential impact of using recycled materials as base or subbase materials under asphaltic concrete pavements.

Because AC pavements contain hydrocarbons or petroleum by-products, we believe that the owners and users of the proposed project should be made aware of

the potential for environmental and/or health issues if recycled AC pavement materials are used directly below the building.

Our experience suggests that recycled crushed concrete may not produce sufficient amount of fine-grained materials (materials with particle size less than the No. 4 sieve). Therefore, the crushed concrete slab materials may need to be capped with at least one lift (8 inches) of select granular fills (imported materials) to reduce the potential for appreciable water infiltration into the subgrade soils when used as select granular fill materials.

Because demolished concrete slabs generally are considered construction debris, we believe that the owner and users of the proposed project should be conferred with on the use of crushed concrete for fill materials in the building area.

### 3.3.3 Fill Placement and Compaction Requirements

General fill materials (including the crushed concrete and asphaltic concrete pavements) should be placed in level lifts not exceeding 8 inches in loose thickness, moisture-conditioned to at least 2 percent above the optimum moisture, and compacted to at least 90 percent relative compaction. Select granular fill 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. The compaction requirement for the last lift of the fill (finished subgrade) below areas subjected to vehicular traffic should be increased to at least 95 percent relative compaction. Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same soil established in accordance with ASTM D 1557. Optimum moisture is the water content (percentage by dry weight) corresponding to the maximum dry density. Compaction of fill materials should be accomplished using sheepsfoot rollers, vibratory rollers, or other types of acceptable compaction equipment.

#### 3.3.4 Excavation

Based on our field exploration, the project site generally is underlain by surface fills near the existing ground surface. It is anticipated that the near-surface materials may be excavated with normal heavy excavation equipment, such as excavating with a backhoe excavator. However, it should be noted that coral formation was encountered below the surface soil horizon in one of the borings drilled at the project site. Therefore, it should be noted that the contractors may encounter some localized hard excavation conditions, requiring the use of hoerams or chipping.

The above discussions regarding the excavation of the materials are based on our visual observation of the existing site conditions and field data from the borings drilled at the project site. Contractors proposing to bid on this project should be encouraged to examine the site conditions and the boring data to make their own interpretation.

### 3.4 Pavement Design

We understand flexible pavements will be used for the new driveways and parking areas planned at the project site. In general, we anticipate the vehicle loading for the proposed driveways and parking areas for this project would consist of primarily passenger vehicles with some light trucks and occasional heavy trucks only. Therefore, we have assumed generally light traffic loading conditions for pavement design purposes.

We have assumed the pavement subgrade soils will be similar to the sandy silts and silty gravel soils generally encountered during our field exploration. On this basis, we recommend utilizing the following preliminary pavement design for this project.

#### Flexible Pavement Section

2.0-Inch Asphaltic Concrete

6.0-Inch Aggregate Base Course (95 Percent Relative Compaction)

6.0-Inch Aggregate Subbase Course (95 Percent Relative Compaction)

14.0-Inch Total Pavement Thickness on Moist Compacted Subgrade

The subgrade soils under the pavement areas should be scarified to a depth of about 8 inches (where possible), moisture-conditioned to above the optimum

moisture, and compacted to at least 95 percent relative compaction. CBR tests and/or field observations should be performed on the actual subgrade soils during construction to confirm that the above design section is adequate. The aggregate base and subbase course should consist of crushed basaltic aggregates compacted to a minimum of 95 percent relative compaction.

In general, paved areas should be sloped, and drainage gradients should be maintained to carry surface water off the pavements. Surface water ponding should not be allowed on-site during or after construction. Where concrete curbs are used to isolate landscaping in or adjacent to the pavement areas, we recommend the curbs extend a minimum of 2 inches into the subgrade soil to reduce the potential for migration of excessive landscape water into the pavement section.

### **3.5 Underground Utility Lines**

We envision new utility lines (i.e., water, sewer, and drain lines and utility line connections) likely will be required for the project development. In general, we recommend providing granular bedding consisting of 6 inches of open-graded gravel (ASTM C 33, No. 67 gradation) below the pipes for uniform support.

Due to the anticipated depths of the excavations for the utility lines, some of the utility line trenches likely will extend below the groundwater table and likely will encounter soft and/or loose soils at the bottom of the trenches. Where soft and/or loose soils are encountered at or near the invert of the pipes, an additional 18 to 24 inches of open-graded gravel wrapped in a non-woven filter fabric (Mirafi 180N or equivalent) should be provided below the bedding layer for more uniform support. Free-draining granular materials, such as open-graded gravel, also should be used for the initial trench backfill up to about 12 inches above the pipes or about 12 inches above the groundwater level to provide adequate support around the pipes. It is critical that the free-draining materials be used to reduce the potential for formation of voids below the haunches of pipes and to provide adequate support around the sides of the pipes.

The upper portion of the trench backfill from a level 12 inches above the pipes to the top of the subgrade may consist of the excavated on-site soils, if they are free of

deleterious materials. The backfill should be moisture-conditioned to above the optimum moisture, placed in maximum 8-inch level loose lifts, and mechanically compacted to a minimum of 90 percent relative compaction to reduce the potential for appreciable future ground subsidence. Where trenches will be below areas subject to vehicular traffic (paved areas), the upper 3 feet of the trench backfill below the pavement grade should be compacted to a minimum of 95 percent relative compaction.

### **3.6 Drainage**

The finished grades outside the housing office structure should be sloped adequately to shed water away from the foundations and slabs and to reduce the potential for ponding at all times. In addition, it is advised to install gutter systems around the structure and to divert the discharge away from the foundation and slab areas. Excessive landscape watering near the foundations and slabs also should be avoided. Planters next to foundations (within 3 feet of the edge of the slab) should be avoided or have concrete bottoms and drains to reduce the potential for excessive water infiltration into the subsurface.

These drainage requirements are essential for the proper performance of the above foundation recommendations because ponded water could cause subsurface soil saturation and subsequent loss of strength. The foundation excavations should be properly backfilled against the walls or slab edges immediately after setting of the concrete to reduce the potential for excessive water infiltration into the subsurface. In addition, drainage swales should be provided as soon as possible, and positive gradients should be maintained at all times to drain surface water runoff away from the foundations and slabs.

### **3.7 Design Review**

Preliminary and final drawings and specifications for the project should be forwarded to Geolabs for review and written comments prior to bid solicitation. This review is necessary to evaluate general conformance of the plans and specifications with the intent of the foundation and earthwork recommendations provided herein. If this review is not made, Geolabs cannot be responsible for misinterpretation of our recommendations.

### **3.8 Construction Monitoring**

Geolabs should be retained to provide geotechnical engineering services during construction. The following are critical items of construction monitoring that require "Special Inspection":

- Subgrade preparation
- Fill placement and compaction
- Foundation excavation and preparation

A Geolabs representative also should observe the other aspects of the earthwork construction to confirm compliance with 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. Geolabs should be accorded the opportunity to provide construction observation services to confirm our assumptions in providing the recommendations presented herein.

If the actual exposed subsurface conditions encountered during construction differ from those assumed or considered in this report, Geolabs should be contacted to review and/or revise the geotechnical recommendations presented herein.

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END OF DISCUSSION AND RECOMMENDATIONS

## SECTION 4. LIMITATIONS

The analyses and recommendations submitted herein are based, in part, upon information obtained from literature research, field borings, and bulk samples. Variations of subsurface conditions between and beyond the borings and bulk samples 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 herein are approximate, having been taped from features shown on the Topographic Survey dated August 10, 2009 transmitted by Forest City Military Communities Hawaii. Elevations of the borings were estimated based on interpolation between the elevation points 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 the soil and/or 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 herein. 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 Forest City Military Communities Hawaii for specific application to the design of the proposed *Forest City Marine Base Housing Office* project at Marine Corps Base Hawaii 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 architects and engineers in the preparation of design drawings related to the development of the housing office project. Therefore, this report may not contain sufficient data, or the proper information, to serve as a basis for construction cost estimates. A contractor

#### SECTION 4. LIMITATIONS

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

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END OF LIMITATIONS

**CLOSURE**

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

Project Location Map .....Plate 1  
Site Plan .....Plate 2  
Appendix A: Field Exploration.....Page A-1  
Appendix B: Laboratory Tests.....Page B-2

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Respectfully submitted,

**GEOLABS, INC.**

By Teddy S.T. Kwok  
**Teddy S.T. Kwok, P.E.**  
Senior Project Engineer

By Robin M. Lim  
**Robin M. Lim, P.E.**  
Vice President

RML:TK:mj

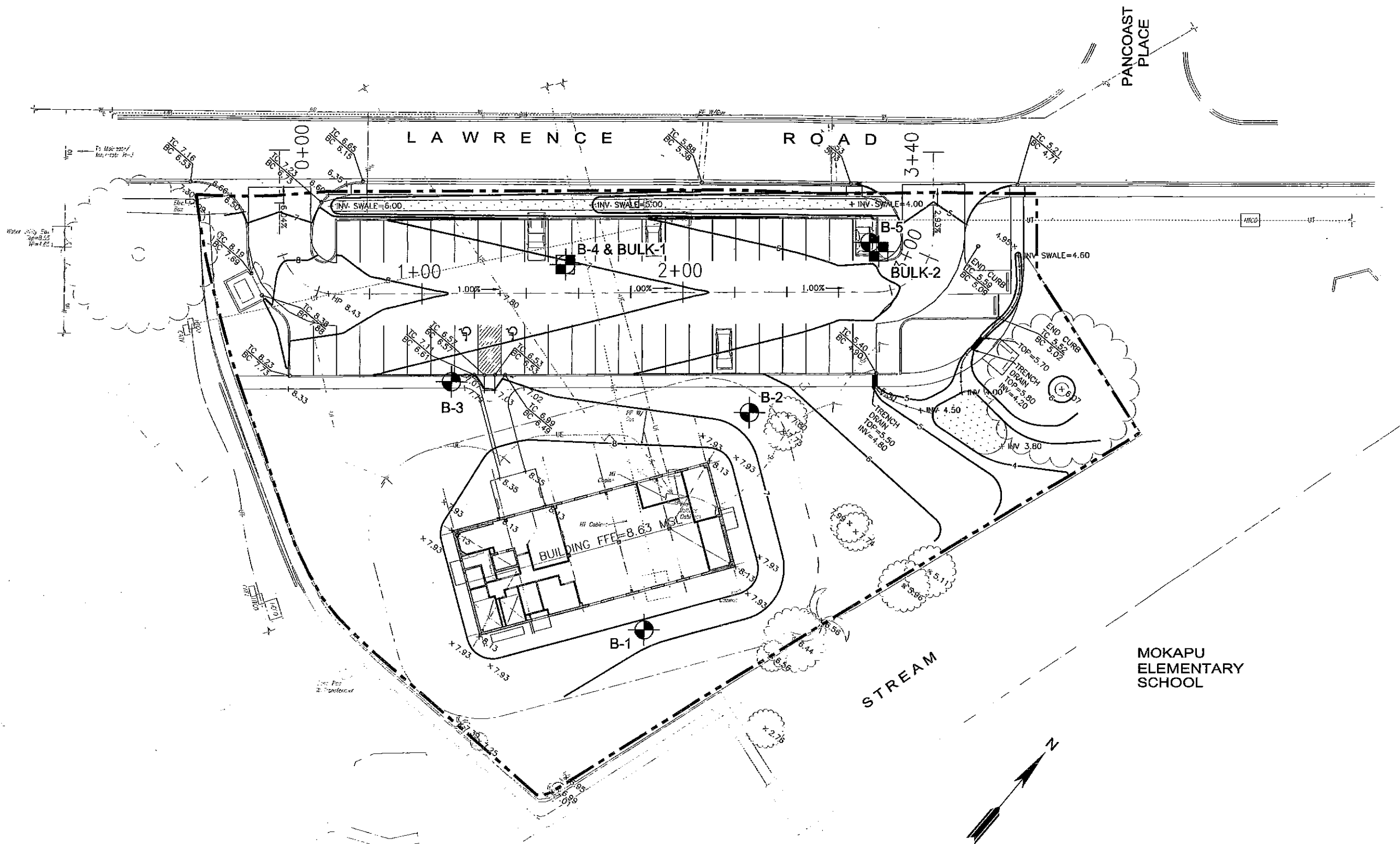
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**PLATES**

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 File: T:\Drafting\_990\AW\k\15884\_00(D)\ForestCity\Office\15884\_00(D)\SitePlan.dwg\SitePlan  
 Plotter: DocuColor 240-250 PS V1.2; PlotStyle: GEO.LABS-No-Differentiating.ctb

- LEGEND:**
-  APPROXIMATE BORING LOCATION
  -  APPROXIMATE BULK SAMPLE LOCATION

**REFERENCE:**  
 SITE PLAN TRANSMITTED BY RIM ARCHITECTS ON OCTOBER 8, 2009.



MOKAPU  
 ELEMENTARY  
 SCHOOL

**SITE PLAN**  
 FOREST CITY MARINE BASE HOUSING OFFICE  
 KANEOHE BAY, OAHU, HAWAII

<b>GEOLABS, INC.</b> Geotechnical Engineering		
DATE OCTOBER 2009	DRAWN BY JRP	PLATE
SCALE 1" = 40'	W.O. 5884-00(D)	<b>2</b>



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**APPENDIX A**

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## **APPENDIX A**

### **Field Exploration**

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We explored the subsurface conditions at the project site by drilling and sampling five borings, designated as Boring Nos. 1 through 5, extending to depths of approximately 11.5 to 26.5 feet below the existing ground surface. The approximate boring locations are shown on the Site Plan, Plate 2. The borings were drilled using both a truck-mounted drill rig and portable drilling equipment equipped with continuous flight augers.

Our field engineer classified the materials encountered in the borings by visual and textural examination in the field in general accordance with ASTM D 2488, Standard Practice for Description and Identification of Soils, and monitored the drilling operations on a near-continuous (full-time) basis. These classifications were further reviewed visually and by testing in the laboratory. Soils were classified in general accordance with ASTM D 2487, Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System), as shown on the Soil Log Legend, Plate A. Graphic representations of the materials encountered are presented on the Logs of Borings, Plates A-1 through A-5.

Relatively "undisturbed" soil samples were obtained 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 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. The penetration resistance shown on the logs of borings indicates the number of blows required for the specific sampler type used. The blow counts may need to be factored to obtain the Standard Penetration Test (SPT) blow counts.

Pocket penetrometer tests were performed on selected cohesive soil samples retrieved in the field. The pocket penetrometer test provides an indication of the unconfined compressive strength of the soil sample. Pocket penetrometer test results are presented on the Logs of Borings at the appropriate sample depths.



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Geotechnical Engineering

**Soil Log Legend**

**UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)**

MAJOR DIVISIONS			USCS	TYPICAL DESCRIPTIONS
COARSE-GRAINED SOILS  MORE THAN 50% OF MATERIAL RETAINED ON NO. 200 SIEVE	GRAVELS  MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS		<b>GW</b> WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
		LESS THAN 5% FINES		<b>GP</b> POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES		<b>GM</b> SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES
		MORE THAN 12% FINES		<b>GC</b> CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
	SANDS  50% OR MORE OF COARSE FRACTION PASSING THROUGH NO. 4 SIEVE	CLEAN SANDS		<b>SW</b> WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
		LESS THAN 5% FINES		<b>SP</b> POORLY-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
		SANDS WITH FINES		<b>SM</b> SILTY SANDS, SAND-SILT MIXTURES
		MORE THAN 12% FINES		<b>SC</b> CLAYEY SANDS, SAND-CLAY MIXTURES
FINE-GRAINED SOILS  50% OR MORE OF MATERIAL PASSING THROUGH NO. 200 SIEVE	SILTS AND CLAYS  LIQUID LIMIT LESS THAN 50			<b>ML</b> INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				<b>CL</b> INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				<b>OL</b> ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS  LIQUID LIMIT 50 OR MORE			<b>MH</b> INORGANIC SILT, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				<b>CH</b> INORGANIC CLAYS OF HIGH PLASTICITY
				<b>OH</b> ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				<b>PT</b> 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)



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FOREST CITY MARINE BASE HOUSING OFFICE  
KANEHOE BAY, OAHU, HAWAII

Log of Boring

1

Laboratory			Field				Depth (feet)	Sample	Graphic	USCS	Approximate Ground Surface Elevation (feet MSL): 6.5 *
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)					Description
	15	82			52/4" Ref.		0	0	ML	Brown <b>SANDY SILT</b> , medium stiff to stiff, dry (topsoil)	
					20/2" Ref.		0	0	GM	Brown <b>SILTY GRAVEL</b> with sand, very dense, dry (fill)	
	12	106			50/3" Ref.		5	5		White <b>SANDY GRAVEL (CORALLINE)</b> , very dense, damp (coral formation)	
	42				32		10	10	MH	Yellowish brown <b>CLAYEY SILT</b> , hard	
	33	86			50/3" Ref.	2.8	15	15		grades to dark grayish brown, very hard	
Boring terminated at 15.8 feet											
* Elevations estimated from Topographic Survey dated August 10, 2009 transmitted by Forest City Military Communities Hawaii.											

BORING LOG 5884-00(D).GPJ GEOLABS.GDT 11/10/09

Date Started: September 11, 2009	Water Level: ∇ 5.5 ft.	Plate <b>A - 1</b>
Date Completed: September 11, 2009		
Logged By: E. Aczon	Drill Rig: MOBILE MINUTEMAN	
Total Depth: 15.8 feet	Drilling Method: 4" Auger	
Work Order: 5884-00(D)	Driving Energy: 140 lb. wt., 30 in. drop	



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FOREST CITY MARINE BASE HOUSING OFFICE  
KANEHOE BAY, OAHU, HAWAII

Log of Boring

2

Laboratory			Field				Depth (feet)	Sample	Graphic	USCS	Approximate Ground Surface Elevation (feet MSL): 7.5 *
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)					Description
LL=54 PI=33 Consol.	6	95			68		0 - 1.5	ML GM	Brown <b>SANDY SILT</b> , medium stiff to stiff, dry (topsoil)		
	9				16		1.5 - 4.5		Yellowish brown <b>SILTY GRAVEL</b> with sand, very dense, damp (fill) grades to medium dense		
	77	56			3		4.5 - 5.5	CH	Yellowish brown <b>SILTY CLAY</b> , soft, wet		
	15				58		5.5 - 10	SW	Gray <b>GRAVELLY SAND</b> , very dense		
	66	59			36		10 - 15		grades to grayish brown, dense		
	54				26		15 - 20		grades to medium dense		
	52	73			33	>4.5	20 - 26.5	MH	Brownish gray <b>CLAYEY SILT</b> , hard to very hard		
Boring terminated at 26.5 feet											

BORING LOG 5884-00(D).GPJ GEOLABS.GDT 11/10/09

Date Started: September 10, 2009	Water Level: ∇ 6.0 ft. 09/10/2009 1110 HRS	Plate <b>A - 2</b>
Date Completed: September 10, 2009		
Logged By: E. Aczon	Drill Rig: CME-75	
Total Depth: 26.5 feet	Drilling Method: 4" Auger	
Work Order: 5884-00(D)	Driving Energy: 140 lb. wt., 30 in. drop	



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Geotechnical Engineering

FOREST CITY MARINE BASE HOUSING OFFICE  
KANEOHE BAY, OAHU, HAWAII

Log of Boring

3

Laboratory			Field				Depth (feet)	Sample	Graphic	USCS	Approximate Ground Surface Elevation (feet MSL): 7.7 *
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)					Description
Sieve	15	108			53		0-1.5	ML	Yellowish brown <b>SANDY SILT</b> , medium stiff to stiff, dry (topsoil)		
	8				17		1.5-3.0	SM	Brown <b>SILTY SAND</b> with gravel, very dense, damp (fill)		
							3.0-4.5	GM	Brown-white <b>SILTY GRAVEL (CORALLINE)</b> , medium dense, damp to moist (fill)		
LL=51 PI=25	15	89			12		4.5-5.5	CH	Brown <b>SILTY CLAY</b> with gravel, stiff, wet		
							5.5-10.0				
Sieve	28				5		10.0-15.0	SP-SM	Gray-brown poorly graded <b>SAND</b> with silt and gravel, loose		
							15.0-20.0				
	32	94			33		20.0-25.0	MH	Yellow-brown <b>CLAYEY SILT</b> , hard		
							25.0-30.0				
	60				16		30.0-35.0	ML	Yellowish brown <b>SANDY SILT</b> , very stiff		
							35.0-40.0				
	60	66			33	3.8	40.0-45.0		grades to hard		
							45.0-50.0				
							50.0-55.0				
							55.0-60.0				
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KANE OHE BAY, OAHU, HAWAII

Log of Boring

4

Laboratory			Field				Depth (feet)	Sample	Graphic	USCS	Description
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)					
	8	98			59		0		MI GM	Brown <b>SANDY SILT</b> , medium stiff to stiff, dry (topsoil)	
	10				54		1			Brown <b>SILTY GRAVEL</b> with sand, very dense, dry to moist (fill)	
	68	65			4		5		CH	Yellowish brown <b>SILTY CLAY</b> , soft, wet	
	17				13		10		SW	Gray <b>GRAVELLY SAND</b> with traces of shells, medium dense	
	35	89			10		15			grades to grayish brown	
							16.5			Boring terminated at 16.5 feet	

BORING LOG 5884-00(D).GPJ GEOLABS.GDT 11/10/09

Date Started: September 10, 2009	Water Level: ∇ Not Measured	Plate  A - 4
Date Completed: September 10, 2009		
Logged By: E. Aczon	Drill Rig: CME-75	
Total Depth: 16.5 feet	Drilling Method: 4" Auger	
Work Order: 5884-00(D)	Driving Energy: 140 lb. wt., 30 in. drop	



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Log of Boring

5

Laboratory			Field				Depth (feet)	Sample	Graphic	USCS	Approximate Ground Surface Elevation (feet MSL): 6.5 *
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)					Description
	18	82			29		0		ML	Brown <b>SANDY SILT</b> , medium stiff to stiff, dry (topsoil)	
	16				13		1		GM	Gray-brown <b>SILTY GRAVEL</b> with sand, medium dense, dry to damp (fill)	
	43	71			4		5		CH	Yellowish brown <b>SILTY CLAY</b> , soft, wet	
	19				6		10		SW	Gray <b>GRAVELLY SAND</b> , loose	
Boring terminated at 11.5 feet											

BORING\_LOG\_5884-00(D).GPJ GEOLABS.GDT 11/10/09

Date Started: September 10, 2009	Water Level: $\nabla$ 7.0 ft.	Plate  <b>A - 5</b>
Date Completed: September 10, 2009		
Logged By: E. Aczon	Drill Rig: CME-75	
Total Depth: 11.5 feet	Drilling Method: 4" Auger	
Work Order: 5884-00(D)	Driving Energy: 140 lb. wt., 30 in. drop	

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**APPENDIX B**

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## **APPENDIX B**

### Laboratory Tests

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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.

Two one-inch Ring Swell tests were performed on selected soil samples to evaluate the swelling potential of the near-surface soils. The test results are summarized on Plate B-1.

Two 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, and the test results are presented graphically on Plate B-2.

Two 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 distributions are provided on Plate B-3.

One Consolidation test with time rates (ASTM D 2435) was performed on a sample of the potentially compressible soils to evaluate the compressibility characteristics of the materials encountered. The consolidation test results are presented on Plate B-4.

Two laboratory California Bearing Ratio tests (ASTM D 1883) were performed on bulk samples of the near-surface soils to evaluate the pavement support characteristics of the soils. The samples were remolded to near the optimum moisture content and saturated. The test results are presented on Plates B-5 and B-6.

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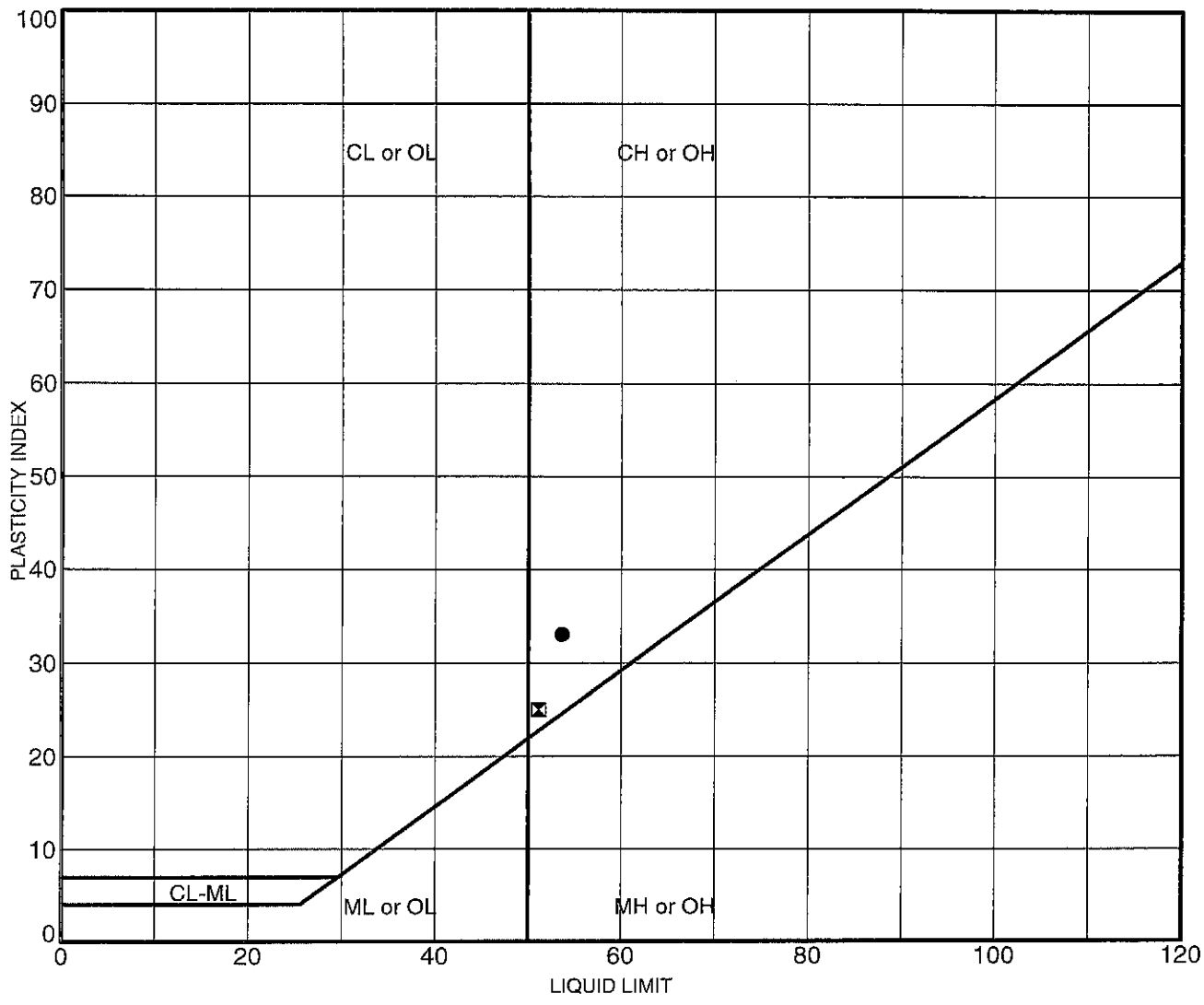
**SUMMARY OF RING SWELL TESTS**

Forest City Marine Base Housing Office  
Kaneohe Bay, Oahu, Hawaii

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<u>Location</u>	<u>Depth</u> (feet)	<u>Soil</u> <u>Description</u>	<u>Dry</u> <u>Density</u> (pcf)	<u>Moisture Contents</u>			<u>Ring</u> <u>Swell</u> (%)
				<u>Initial</u> (%)	<u>Air-Dried</u> (%)	<u>Final</u> (%)	
B-1	1.0 – 1.8	Brown Sandy Silt	78	19	16	42	2.3
B-3	1.0 – 2.5	Yellowish Brown Sandy Silt	89	20	15	37	1.4

NOTE: Samples tested were relatively undisturbed in 2.4-inch diameter by 1-inch high rings. They were air-dried overnight and then saturated for 24 hours under a surcharge pressure of 55 psf.



Sample	Depth (ft)	LL	PL	PI	Description
● B-2	5.0-6.5	54	21	33	Yellowish brown silty clay (CH)
☒ B-3	5.0-6.5	51	26	25	Brown silty clay (CH) with gravel

G. ATTERBERG 5884-00(D).GRP.J GEOLABS.GDT 10/16/09

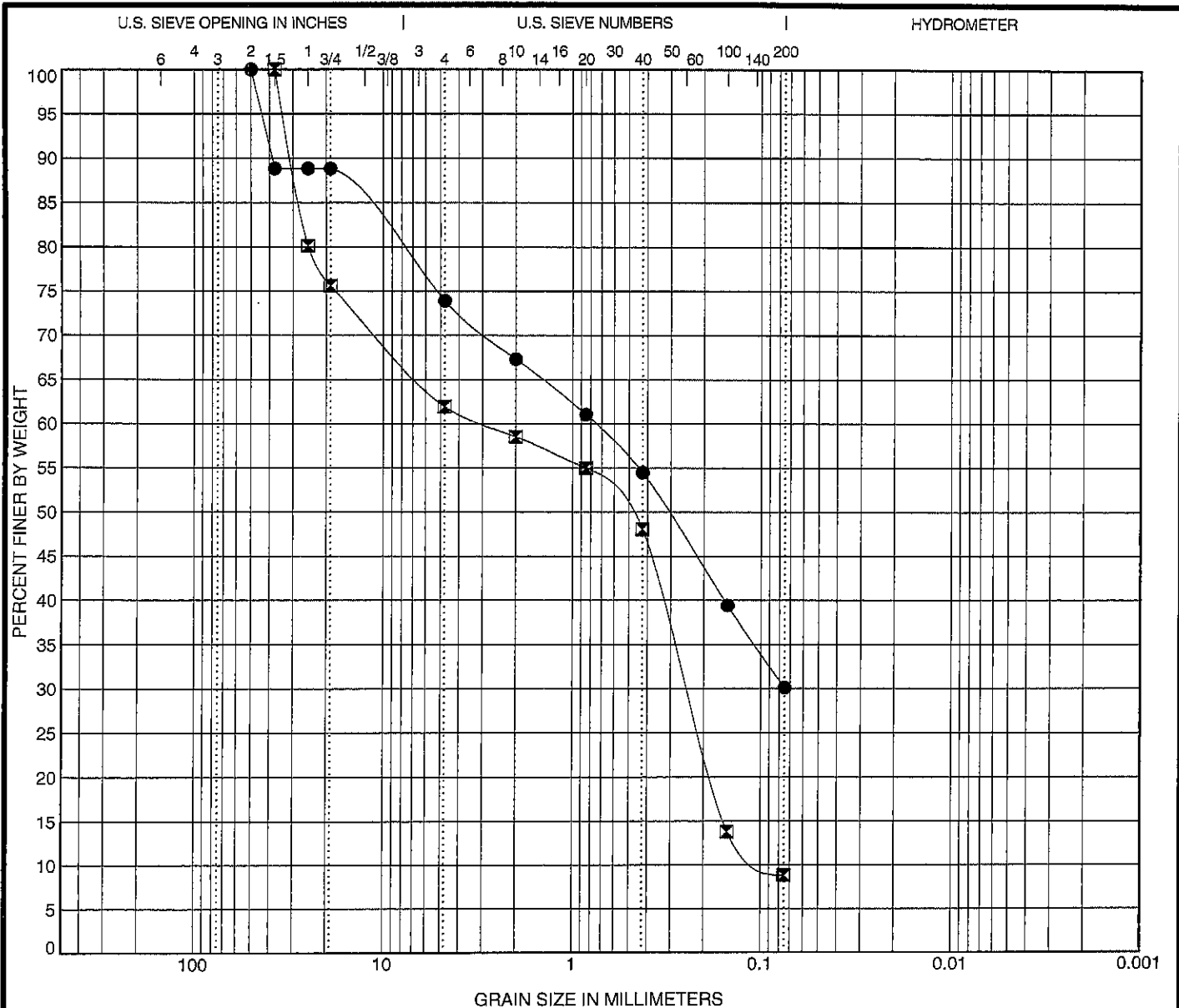


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 W.O. 5884-00(D)

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

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Plate  
**B - 2**



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Sample	Depth (ft)	Description	LL	PL	PI	Cc	Cu
● B-3	1.0-2.5	Brown silty sand (SM) with gravel					
☒ B-3	10.0-11.5	Gray-brown poorly graded sand (SP-SM) with silt and gravel				0.2	32.9

Sample	Depth (ft)	D100 (mm)	D60 (mm)	D30 (mm)	D10 (mm)	%Gravel	%Sand	%Fine
● B-3	1.0-2.5	50	0.761			26.1	43.8	30.1
☒ B-3	10.0-11.5	37.5	2.913	0.245	0.088	38.1	53.1	8.8

G\_GRAIN\_SIZE\_5884-00(D).GPJ GEOLABS.GDT 10/14/09

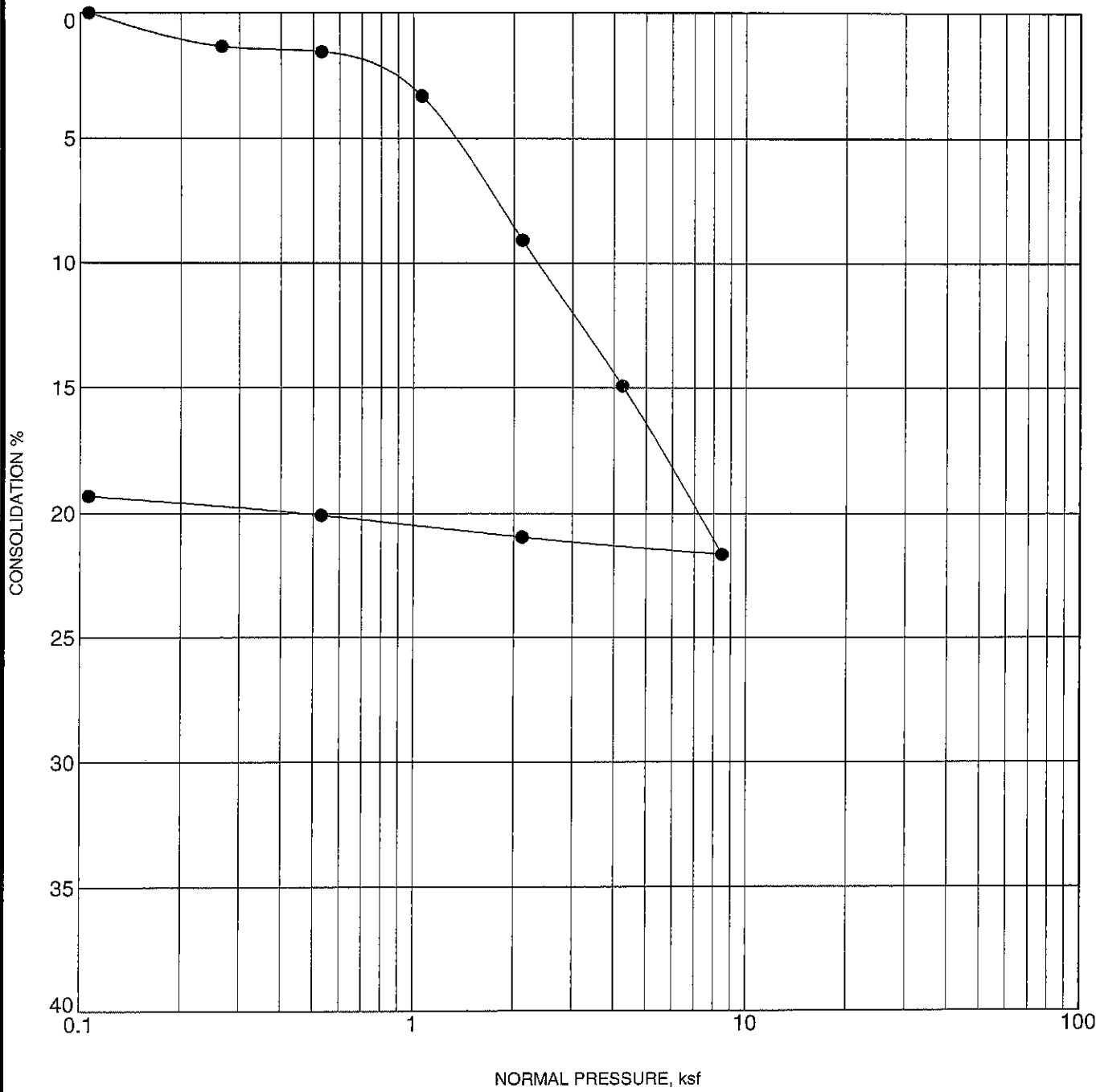


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**GRAIN SIZE DISTRIBUTION - ASTM C 117 & C 136**

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 KANEOHE BAY, OAHU, HAWAII

Plate  
**B - 3**



Sample: B-2  
 Depth: 5.0 - 6.5 feet  
 Description: Yellowish brown silty clay (CH)

Liquid Limit = 54      Plasticity Index = 33

	Initial	Final
Water Content, %	76.9	56.4
Dry Density, pcf:	55.5	68.8
Void Ratio	2.277	1.644
Degree of Saturation, %	98.4	100.0
Sample Height, inches	1.0000	0.7871

3 CONSOL 5884-00(D).GPJ GEOLABS.GDT 11/10/09

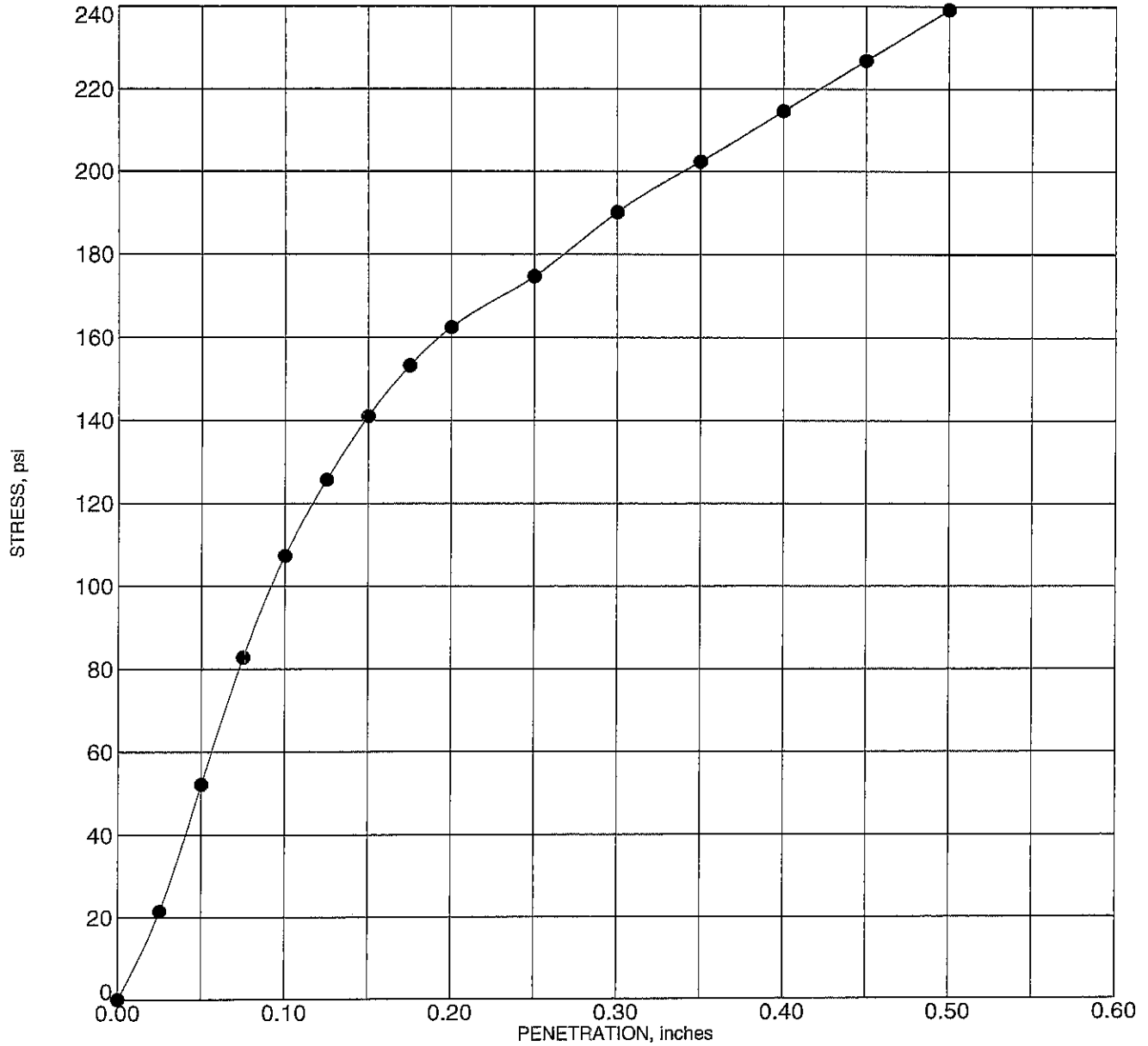


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**CONSOLIDATION TEST - ASTM D 2435**

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Plate  
**B - 4**



Corr. CBR @ 0.1"	11.3
Swell (%)	1.99

Sample: Bulk-1  
 Depth: 0.0 - 1.0 feet  
 Description: Brown silty clay with some sand and gravel

Molding Dry Density (pcf)	102.5	Hammer Wt. (lbs)	10
Molding Moisture (%)	20.8	Hammer Drop (inches)	18
Days Soaked	4	No. of Blows	56
Aggregate	3/4 inch minus	No. of Layers	5

G. CBR 5884-00(D).G.FJ GEOLABS.GDT 10/14/09

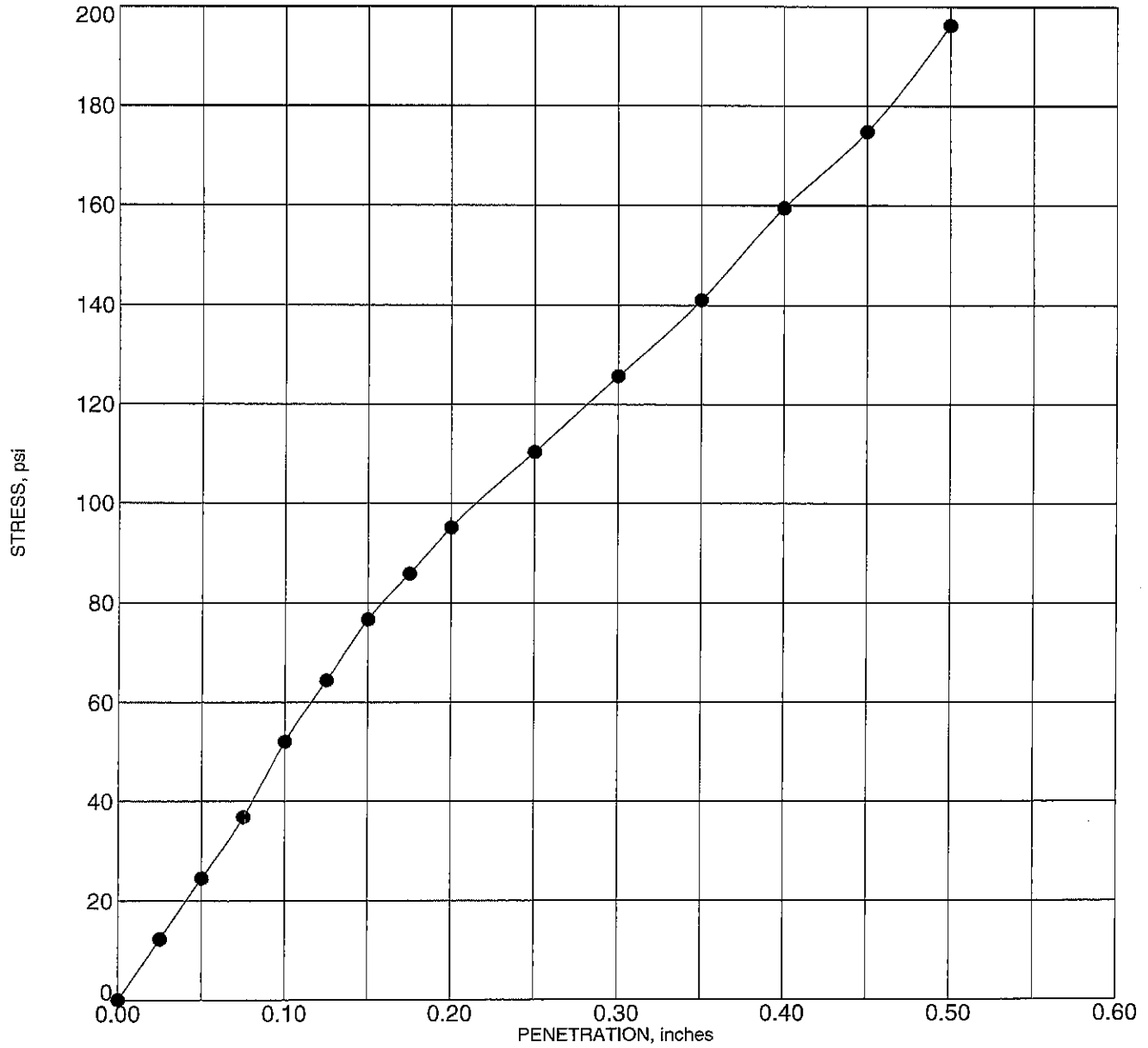


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 W.O. 5884-00(D)

**CALIFORNIA BEARING RATIO - ASTM D 1883**

FOREST CITY MARINE BASE HOUSING OFFICE  
 KANEOHE BAY, OAHU, HAWAII

Plate  
**B - 5**



Corr. CBR @ 0.1"	8.8
Swell (%)	3.03

Sample: Bulk-2  
 Depth: 0.0 - 1.0 feet  
 Description: Brown silty clay with some gravel (coralline and basaltic)

Molding Dry Density (pcf)	105.6	Hammer Wt. (lbs)	10
Molding Moisture (%)	19.5	Hammer Drop (inches)	18
Days Soaked	4	No. of Blows	56
Aggregate	3/4 inch minus	No. of Layers	5

G. CBR 5884-00(D).GFI GEOLABS.GDT 10/16/09



**GEOLABS, INC.**  
 GEOTECHNICAL ENGINEERING  
 W.O. 5884-00(D)

**CALIFORNIA BEARING RATIO - ASTM D 1883**

FOREST CITY MARINE BASE HOUSING OFFICE  
 KANEOHE BAY, OAHU, HAWAII

Plate  
**B - 6**

