# GEOTECHNICAL ENGINEERING EXPLORATION
# SEISMIC RETROFIT OF KAHOLO BRIDGE
# HAWAII BELT ROAD
# PROJECT NO. BR-019-2(072)
# DISTRICT OF HAMAKUA, ISLAND OF HAWAII

**W.O. 8063-00    JULY 12, 2024**

Prepared for

**KSF, INC.**

and

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


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

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GERALD Y. SEKI
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THIS WORK WAS PREPARED BY
ME OR UNDER MY SUPERVISION.

4-30-26

SIGNATURE EXPIRATION DATE OF THE LICENSE


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Hawaii • California

GEOLABS, INC.
*Geotechnical Engineering and Drilling Services*

July 12, 2024
W.O. 8063-00

**Mr. Calvin Miyahara**
**KSF, Inc.**
615 Piikoi Street, Suite 300
Honolulu, HI 96814-4554

Dear **Mr. Miyahara:**

Geolabs, Inc. is pleased to submit our report entitled “Geotechnical Engineering Exploration, Seismic Retrofit of Kaholo Bridge, Hawaii Belt Road, Project No. BR-19-2(072), District of Hamakua, Island of Hawaii,” prepared in support of the design of the bridge retrofit project.

Our work was performed in general accordance with the scope of services outlined in our revised fee proposal dated November 27, 2019.

Please note that the soil and rock 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 the report. If there is any point that is not clear, please contact our office.

Very truly yours,

**GEOLABS, INC**

**Gerald Y. Seki, P.E.**
Vice President

GS:GB:lf

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**GEOTECHNICAL ENGINEERING EXPLORATION**
**SEISMIC RETROFIT OF KAHOLO BRIDGE**
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## SUMMARY OF FINDINGS AND RECOMMENDATIONS


The subsurface conditions encountered below the surface AC pavement consists of fill comprised of loose to medium dense gravel and sand and stiff to hard clayey silt extending to depths of about 1 to 5 feet below the existing ground surface. The fills were underlain by medium stiff to very stiff residual soils, saprolite, and medium dense weathered basalt. The residual soils, saprolite and weathered basalt extended to a depth of about 28 feet below the existing ground surface at the Hilo side abutment, and the saprolite extended to depths of about 36 to 43 feet below the existing ground surface at the Honokaa side abutment. Below the residual soils, saprolite and weathered basalt, as well as highly to moderately weathered basalt rock, were encountered to depths of about 49 to 63 feet below the existing ground surface. The basalt rock graded to moderately to slightly weathered and medium hard to hard down to the maximum depth explored of about 102.5 feet below the existing ground surface. We did not encounter groundwater at the time of our field exploration.

Based on the seismic evaluation of the bridge structure by the project structural engineer, we understand that appreciable lateral deflections of the bridge structure would occur during a seismic event. The lateral deflection of the bridge structure in the longitudinal direction would be reduced by the passive pressure resistance of the shallow bridge foundations and the stiffness of the abutment fills. A group of battered micropiles would be installed to provide resistance to the transverse lateral load and reduce the amount of transverse lateral deflection of the bridge structure.

In general, a 7.625-inch diameter cased micropile system with a minimum grout bulb diameter of 7.625 inches should be used for the battered micropiles to provide lateral load resistance in the transverse direction during a seismic event. The uplift and lateral supporting capacities of the micropile would be derived primarily from skin friction between the micropile bonded zone and the surrounding saprolite soils and highly to moderately weathered soft to medium hard basalt formation. The bonded zone of the micropile should be embedded a minimum of 30 and 45 feet with micropile embedment lengths of 40 and 55 feet for the Hilo and Honokaa abutments, respectively. The text of this report should be referred to for detailed discussion and specific design recommendations.


END OF SUMMARY OF FINDINGS AND RECOMMENDATIONS

# 1. GENERAL

This report presents the results of our geotechnical engineering exploration and engineering analyses performed in support of the design of the *Seismic Retrofit of Kaholo Bridge* project in the District of Hamakua on the Island of Hawaii. The project location and general vicinity are shown on the Project Location Map, Plate 1.

This report summarizes our findings and presents our geotechnical recommendations based on our field exploration, laboratory testing, and engineering analyses. The recommendations presented herein are intended for the design of the bridge seismic retrofit only. The findings and recommendations presented herein are subject to the limitations noted at the end of this report.

## 1.1 Project Considerations

The Kaholo Bridge project is located along Hawaii Belt Road (Route 19) in the District of Hamakua on the Island of Hawaii. The project location and vicinity are shown on the Project Location Map, Plate 1. The existing Kaholo Bridge structure traverses Kaholo Stream.

The existing bridge structure is a three-span bridge supported by two intermediate piers and abutments at both ends. Based on the available plans, the existing bridge structure is supported on shallow foundations. The bridge structure is approximately 220 feet in length and is approximately 29.7 feet in width.

The project consists of the seismic retrofit of the Kaholo Bridge. Based on the seismic evaluation performed by KSF, Inc., we understand that appreciable lateral deflections of the bridge structure would occur during a seismic event. The lateral deflection of the bridge structure in the longitudinal direction would be reduced by the passive pressure resistance of the shallow bridge foundations and the stiffness of the abutment fills. To reduce the amount of transverse lateral deflection of the bridge structure, a group of battered micropiles would be installed to provide resistance to the transverse lateral load. During a seismic event, the battered micropiles would engage their lateral resistance resulting in a reduction of the amount of transverse lateral deflection of the bridge structure to an acceptable limit.

## 1.2 Purpose and Scope

The purpose of our exploration program was to obtain an overview of the surface and subsurface soil conditions at the project site to develop a generalized soil/rock data set to formulate geotechnical recommendations for the design of the seismic retrofit of the existing bridge structure. Our work was performed in general accordance with the scope of services outlined in our revised fee proposal dated November 27, 2019. The scope of our work for this exploration included the following tasks and work efforts:

1. Review of available in-house and geologic information for areas near the bridge.
2. Application of permits from the State of Hawaii – Department of Transportation, Highways Division prior to our drill crew mobilization.
3. Coordination of the utility toning with various utility companies and clearance of the proposed boring locations by our geologist.
4. Traffic control at the boring locations during our field exploration.
5. Mobilization and demobilization of a truck-mounted drill rig and two operators from Honolulu to and from the project site.
6. Drilling and sampling of four borings extending to depths of about 76 to 102.5 feet below the existing ground surface.
7. Performance of a shear wave velocity test of one of the borings to a depth of about 101.7 feet below the existing ground surface.
8. Coordination of the field exploration and logging of the borings by our geologist.
9. Laboratory testing of selected samples obtained during our field exploration as an aid to classify the materials and evaluate their engineering properties.
10. Analyses of the field and laboratory data to formulate geotechnical engineering recommendations for the design of the seismic retrofit of the existing bridge structure.
11. Preparation of this geotechnical engineering report summarizing our work on the project and presenting our findings and recommendations.
12. Coordination of our overall work on the project by our project engineer.

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

14. 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 shear wave velocity profiling are presented in Appendix B.  Results of the laboratory tests performed on selected soil/rock samples obtained from our field exploration are presented in Appendix C. Photographs of core samples are provided in Appendix D.

END OF GENERAL

# 2. SITE CHARACTERIZATION

## 2.1 Regional Geology

Hawaii, the largest island of the Hawaiian Archipelago, covers an area of approximately 4,000 square miles. This island was formed by the activity of the following five shield volcanoes: Kohala (long extinct), Mauna Kea (activity during recent geologic time), Hualalai (last erupted in 1801), and Mauna Loa and Kilauea (both still active).

The project site is located on the northeastern flank of the Mauna Kea Shield Volcano, which had been built up the successive accumulation of basaltic lava flows and pyroclastic materials. The most recent lava flows that comprise the ground surface and shallow subsurface at the project site are believed to be Pliocene to Pleistocene in age (approximately 1.2 million years to 2 million years before present time) and belong to the upper member of the Hamakua Volcanic Series.

The lava flows were subsequently covered by volcanic ash deposits, locally referred to as Pahala Ash. The Pahala Ash is an air-laid deposit of volcanic ash from the Hamakua Volcanic Series of Mauna Loa Mountain during the Pleistocene Epoch. The Pahala Ash in certain locations has high in-situ moisture contents and low in-situ densities. It generally has low shear strength, and when the moisture content is high enough, it becomes thixotropic, i.e., it loses strength when remolded. Therefore, this type of Pahala Ash may be potentially liquefiable during seismic events.

The lava formation appears to be of pahoehoe flow that is characterized by a smooth, rope-like or billowy surface and an internal structure of vesicular (porous) rock. Cavities are commonly encountered in pahoehoe lavas. Cavities are formed when the lava is still in a molten state and represent both lava tubes (intra-flow cavities) and interflow cavities (blisters and pockets). Lava tubes are formed when molten lava drains from the cooling flow, leaving a hollow tube-like structure that extends for a large longitudinal distance along the flow. Inter-flow cavities are generally smaller in horizontal extent.

## 2.2 Site Description

The Kaholo Bridge is located along Hawaii Belt Road (Route 19) in the District of Hamakua on the Island of Hawaii. The existing Kaholo Bridge structure traverses Kaholo Stream. The bridge site is shown on the Site Plan, Plate 2.

The existing bridge structure is a three-span bridge supported by two intermediate piers and abutments at both ends. Based on the available plans, the existing bridge structure is supported on shallow foundations. The bridge structure is approximately 220 feet in length and is approximately 29.7 feet in width. Abutment Nos. 1 (Hilo side) and 2 (Honokaa side) are located at the east and west ends, respectively. Pier No. 1 is located on the east side of Kaholo Stream Bank, and Pier No. 2 is located on the west side of Kaholo Stream Bank.

The existing ground surface elevations where the seismic retrofit improvements are planned range from about +715 to +724 feet Mean Sea Level (MSL) around the Hilo side abutment and about +738 to +744 feet MSL around the Honokaa side abutment.

The existing bridge is generally aligned in an east-west direction. However, the stream flows below the existing bridge in a southwest to northeast direction. At the time of our field exploration, the streambed was generally dry.

## 2.3 Subsurface Conditions

We explored the subsurface conditions by drilling and sampling four borings, designated as Boring Nos. 1 through 4, extending to depths ranging from about 76 to 102.5 feet below the existing ground surface. The approximate boring locations are shown on the Site Plan, Plate 2.

Boring Nos. 1 and 2 were drilled at the Hilo side abutment. In general, subsurface conditions encountered at the Hilo side abutment consist of 8 to 9 inches of asphaltic concrete over loose to medium dense gravel and sand fill extending to depths of about 1 to 3.5 feet below the existing ground surface. The fill was underlain by residual soils, saprolite and weathered basalt consisting of medium stiff to hard clayey silt and medium dense silty gravel extending to a depth of about 28 feet below the existing ground surface. The residual soils, saprolite and weathered basalt were underlain by highly to moderately weathered, soft to medium hard basalt rock interbedded with layers of clayey silt. The highly to moderately weathered basalt rock extended to depths of about 52 to 63 feet below the existing ground surface. The basalt rock was graded moderately to slightly weathered and medium hard to hard down to the maximum depth explored of about 102.5 feet below the existing ground surface.

Boring Nos. 3 and 4 were drilled at the Honokaa side abutment. In general, subsurface conditions encountered at the Honokaa side abutment consist of 4 to 8 inches of asphaltic concrete over medium dense silty gravel and stiff to hard clayey silt fills extending to depths of about 3.5 to 5 feet below the existing ground surface. The fills were underlain by saprolite and weathered basalt consisting of medium stiff to hard clayey silt and very dense to dense silty gravel extending to depths of about 36 to 43 feet below the existing ground surface. Highly to moderately weathered and soft to medium hard basalt rock was encountered below the saprolite and weathered basalt and extended to depths of about 49 to 50 feet below the existing ground surface. The basalt rock was graded moderately to slightly weathered and medium hard to hard down to the maximum depth explored of about 91 feet below the existing ground surface.

We did not encounter groundwater at the time of our field exploration. However, it should be noted that water levels may vary with stream flow conditions, seasonal rainfall, time of year, and other environmental factors.

Detailed descriptions of our field exploration methodology and the Logs of Borings are presented in Appendix A. Results of the shear wave velocity profiling are presented in Appendix B. Results of the laboratory tests performed on selected soil/rock samples obtained from our field exploration are presented in Appendix C. Photographs of core samples are provided in Appendix D.

### 2.4 Seismic Design Considerations

Based on the AASHTO LRFD Bridge Design Specifications, the project site may be subject to seismic activity, and seismic design considerations will need to be addressed. The following subsections provide discussions on the seismicity, the potential for liquefaction at the project site, and the soil profile for seismic design.

### 2.4.1 Earthquakes and Seismicity

Generally, earthquakes that occur 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 only be detected 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 active volcanoes located in the southern portion of the Island of Hawaii.

The Island of Hawaii has experienced numerous earthquakes greater than Magnitude 6 (M6+), including the October 15, 2006 earthquakes. Based on information obtained from the United States Geological Survey (USGS) Bulletin 2006, the following is a list of some destructive earthquakes that occurred on the Island of Hawaii since 1868.

| DATE | LOCATION | MAGNITUDE |
|---|---|---|
| March 28, 1868 | South Hawaii | 7.0 |
| April 2, 1868 | South Hawaii | 7.9 |
| October 5, 1929 | Hualalai | 6.5 |
| August 21, 1951 | Kona | 6.9 |
| April 26, 1973 | North Hilo | 6.2 |
| November 29, 1975 | Kalapana | 7.2 |
| November 16, 1983 | Kaoiki | 6.7 |

| DATE | LOCATION | MAGNITUDE |
|---|---|---|
| June 25, 1989 | Kalapana | 6.2 |
| October 15, 2006 | Kiholo Bay/Hawi | 6.7 / 6.0 |

It should be noted that several of the significant earthquakes on the Island of Hawaii have occurred on the north and west sides in the past 100 years, including two earthquakes greater than Magnitude 6 in 1929 and 1951. In addition, the October 15, 2006 earthquakes occurred in the northwestern portion of the island. Therefore, it may be concluded that the western side of the Island of Hawaii could experience moderate to severe earthquakes and associated ground shaking, depending on the earthquake's origin.

### 2.4.2 Liquefaction Potential

Soil liquefaction is a condition where saturated cohesionless soils located near the ground surface undergo a substantial loss of strength due to the build-up of excess pore water pressures resulting from cyclic stress applications induced by earthquakes. In this process, when the loose saturated sand deposit is subjected to vibration (such as during an earthquake), the soil tends to densify and decrease in volume, causing an increase in pore water pressure. If drainage is unable to occur rapidly enough to dissipate the build-up of pore water pressure, the effective stress (internal strength) of the soil is reduced. Under sustained vibrations, the pore water pressure build-up could equal the overburden pressure, essentially reducing the soil shear strength to zero and causing it to behave as a viscous fluid. During liquefaction, the soil acquires sufficient mobility to permit both horizontal and vertical movements, and if not confined, will result in significant deformations.

Soils most susceptible to liquefaction are loose, uniformly graded, fine-grained sands and loose silts with little cohesion. The major factors affecting the liquefaction characteristics of a soil deposit are as follows:

| FACTORS | LIQUEFACTION SUSCEPTIBILITY |
|---|---|
| Grain Size Distribution | Fine and uniform sands and silts are more susceptible to liquefaction than coarse or well-graded sands. |
| Initial Relative Density | Loose sands and silts are most susceptible to liquefaction. Liquefaction potential is inversely proportional to relative density. |
| Magnitude and Duration of Vibration | Liquefaction potential is directly proportional to the magnitude and duration of the earthquake. |

Based on the subsurface conditions encountered, the phenomenon of soil liquefaction is not a design consideration for this project site. The risk for potential liquefaction is low based on the subsurface conditions encountered (relatively stiff residual soil and saprolite/weathered basalt overlying basalt rock formation within the depths of our borings).

2.4.3 Soil Profile

Seismic shear wave velocity profiling, using seismic piezocone penetration testing (SCPT) equipment, was performed at the project site to more closely analyze the seismic design considerations. Seismic shear wave velocity testing was performed in Boring No. 2 (Hilo side abutment). Based on the seismic shear wave velocity test results, the weighted average shear wave velocity of the materials within the upper 100 feet of the soil profile is on the order of about 1,274 feet per second.

Based on the results of the seismic shear wave velocity test, the project site may be classified from a seismic analysis standpoint as being a “Very Dense Soil and Soft Rock” site corresponding to a Site Class C soil profile type based on AASHTO 2020 LRFD Bridge Design Specifications, 9th Edition.

Based on the AASHTO 2020 LRFD Bridge Design Specifications, the seismic retrofitted bridge structure will need to be designed based on an earthquake return period of 1,000 years. Based on a 1,000-year return period and the anticipated Site Class, the following seismic design parameters were estimated and may be used for the seismic analysis of the bridge structure planned for the project.

| SEISMIC DESIGN PARAMETERS<br>KAHOLO BRIDGE<br>AASHTO 2020 LRFD BRIDGE DESIGN SPECIFICATIONS<br>1,000-YEAR RETURN PERIOD<br>(~7% PROBABILITY OF EXCEEDANCE IN 75 YEARS) | |
|---|---|
| **Parameter** | **Value** |
| Peak Bedrock Acceleration, PBA (Site Class B) | 0.456g |
| Spectral Response Acceleration (Site Class B), $S_S$ | 0.904g |
| Spectral Response Acceleration (Site Class B), $S_1$ | 0.352g |
| Site Class | "C" |
| Site Coefficient, $F_{pga}$ | 1.00 |
| Site Coefficient, $F_a$ | 1.04 |
| Site Coefficient, $F_v$ | 1.45 |
| Design Peak Ground Acceleration, PGA (Site Class C) or $A_s$ | 0.456g |
| Design Spectral Response Acceleration, $S_{DS}$ | 0.939g |
| Design Spectral Response Acceleration, $S_{D1}$ | 0.509g |
| Seismic Design Category | "D" |

END OF SITE CHARACTERIZATION

## 3. DISCUSSION AND RECOMMENDATIONS

Seismic evaluation of Kaholo Bridge was performed using the soil information obtained from our borings. Based on our evaluation, the Hilo shallow abutment foundations appear to be slightly above highly weathered to moderately weathered basalt. The Hilo shallow wingwall foundations appear to be bearing on stiff residual soils consisting of clayey silt. The Honokaa shallow abutment and wingwall foundations appear to be bearing on stiff to very stiff clayey silt. In general, our analyses for the stiffness modeling parameters of the foundations included the following:

- Estimation of the ultimate bearing capacity of the shallow foundations.
- Estimation of the lateral load resistance of the shallow foundations.
- Estimation of the static and dynamic lateral earth pressures acting on the bridge structure.

Based on the seismic evaluation of the bridge structure by the project structural engineer, we understand that appreciable lateral deflections of the bridge structure would occur during a seismic event. The lateral deflection of the bridge structure in the longitudinal direction would be reduced by the passive pressure resistance of the existing shallow bridge foundations and the stiffness of the abutment fills. A group of battered micropiles would be installed to provide resistance to the transverse lateral load and reduce the amount of transverse lateral deflection of the bridge structure.

To provide the lateral load resistance in the transverse direction during a seismic event, we recommend using a 7.625-inch diameter cased micropile system with a minimum grout bulb diameter of 7.625 inches for the battered micropiles. The uplift and lateral supporting capacities of the micropile would be derived primarily from skin friction between the micropile bonded zone and the surrounding saprolite soils and highly to moderately weathered soft to medium hard basalt formation. The bonded zone of the micropile should be embedded a minimum of 30 and 45 feet for the Hilo and Honokaa abutments, respectively. A detailed discussion of these items and our geotechnical recommendations for the design of the bridge seismic retrofit are presented in the following sections of this report.

## 3.1 Stiffness Modeling Analysis

In order to evaluate the lateral load resistance of the existing bridge structure, foundation capacities and stiffness modeling parameters were estimated based on the soil descriptions obtained from our borings and our laboratory test results.

We understand that soil foundation parameters consisting of the foundation bearing pressure, friction resistance, and static and dynamic lateral earth pressures of the existing bridge structure are required; therefore, an evaluation was conducted of the subsurface conditions and available as-built information. Based on our evaluation, Hilo abutment foundations appear to be slightly above highly weathered to moderately weathered basalt. The Hilo wing wall foundations appear to be bearing on stiff residual soils consisting of clayey silt. The Honokaa abutment and wing wall foundations appear to be bearing on stiff to very stiff clayey silt.

Based on the anticipated bearing conditions of the existing bridge abutment and wing wall foundations, the foundation bearing pressures and friction resistance for the extreme event limit state are provided in the following tables.

| HILO ABUTMENTS AND WINGWALLS | | | | | |
|---|---|---|---|---|---|
| Location | | Hilo | Hilo | Hilo | Hilo |
| | | North Abutment | South Abutment | West Wingwall | East Wingwall |
| Size of Footing | | 19' x 10.5' | 11.5' x 10' | 12' x 11' | 10' x 6.5' |
| Bottom of Footing | | +711 | +721 | +719 | +726 |
| Soil Profile Type (Seismic Analysis) | | Site Class C, $A_s$ = 0.456g | | | |
| Estimated Extreme Event Bearing Capacity (ksf) | | 35.2 | 36.9 | 7.9 | 7.6 |
| Extreme Event Sliding Resistance | Friction, $\tan\delta$ | 0.43 | 0.43 | 0.40 | 0.40 |

<table>
<tr><th colspan="6">HONOKAA ABUTMENTS AND WINGWALLS</th></tr>
<tr><td colspan="2" rowspan="2">Location</td><td>Honokaa</td><td>Honokaa</td><td>Honokaa</td><td>Honokaa</td></tr>
<tr><td>North Abutment</td><td>South Abutment</td><td>West Wingwall</td><td>East Wingwall</td></tr>
<tr><td colspan="2">Size of Footing</td><td>11.5' x 10'</td><td>19' x 10'</td><td>10' x 6.5'</td><td>11' x 6'</td></tr>
<tr><td colspan="2">Bottom of Footing</td><td>+735</td><td>+727</td><td>+740</td><td>+730</td></tr>
<tr><td colspan="2">Soil Profile Type (Seismic Analysis)</td><td colspan="4">Site Class C, $A_s$ = 0.456g</td></tr>
<tr><td colspan="2">Estimated Extreme Event Bearing Capacity (ksf)</td><td>7.8</td><td>11.4</td><td>11.6</td><td>11.4</td></tr>
<tr><td>Extreme Event Sliding Resistance</td><td>Friction, $\tan\delta$</td><td>0.40</td><td>0.43</td><td>0.43</td><td>0.43</td></tr>
</table>

The recommended static lateral earth pressures acting on the abutment and wing wall structures are presented in the following table.

| STATIC LATERAL EARTH PRESSURES | |
|---|---|
| **Active** (pcf) | **At-Rest** (pcf) |
| 38 | 58 |

Dynamic lateral earth forces due to seismic loading are based on a seismic loading peak ground acceleration (As) of 0.456g. The table below summarizes the dynamic lateral earth forces acting on the retaining structures in the event of an earthquake versus the estimated wall displacements.

| DYNAMIC LATERAL EARTH FORCES | |
|---|---|
| **Lateral Movement** (inches) | **Dynamic Lateral Earth Forces** ($H^2$ pounds per linear foot) |
| 1 | 47 |
| 2 | 33 |
| 3 | 20 |

Please note that the above table only applies to level backfill conditions, where H is the height of the wall in feet. The resultant force should be assumed to act through the mid-height of the wall. The above dynamic lateral earth forces are in addition to the static lateral earth pressures provided previously.

### 3.2 Micropiles

As mentioned previously, a group of battered micropiles will be installed at each abutment to provide lateral load resistance in the transverse direction during a seismic event. In general, a micropile consists of a small diameter (usually less than 12 inches) drilled and grouted pile with steel reinforcing. A micropile is typically constructed by drilling a borehole, placing reinforcing steel in the hole, and grouting the borehole. Micropiles are desirable because they can be readily installed in access restrictive environments, such as low headroom areas, and in numerous soil types and ground conditions. Micropile equipment is also more compact, making transport on narrow roadways feasible. In addition, the installation of the micropiles generally causes minimal disturbance to adjacent structures, the adjacent soils, and the environment.

Based on our analyses and the availability of equipment, we envision a cased micropile system with a minimum grout bulb diameter of 7.625 inches may be used. The load-supporting capacity (tension) of the micropile would be derived primarily from skin friction between the micropile bonded zone and the surrounding saprolite soils and highly to moderately weathered soft to medium hard basalt rock formation. The micropile capacities and recommendations are summarized in the following tables.

| **AXIAL (TENSION) LOAD CAPACITIES OF MICROPILE** | |
|---|---|
| **Extreme Event Limit State** (kips) | **Strength Limit State** (kips) |
| 255 | 175 |

| **CASED MICROPILE SYSTEM RECOMMENDATIONS** | |
|---|---|
| Micropile Outside Diameter of Casing | 7.625 inches minimum |
| Micropile Casing Thickness | 0.430 inches minimum |
| Micropile Unbonded Length | 10 feet |

| CASED MICROPILE SYSTEM RECOMMENDATIONS | |
|---|---|
| Diameter of Micropile Bonded Length | 7.625 inches minimum |
| Micropile Bonded Length (Hilo Abutment) | 30 feet minimum |
| Micropile Total Length (Hilo Abutment) | ~40 feet from the bottom of the pile cap |
| Micropile Bonded Length (Honokaa Abutment) | 45 feet minimum |
| Micropile Total Length (Honokaa Abutment) | ~55 feet from the bottom of the pile cap |
| Center Reinforcing Bar (Full Depth) | 1.75-Inch Grade 150 ksi Bar |
| Grout Minimum Compressive Strength | 5,000 psi (water-cement ratio of 0.40 or less) |

To facilitate the micropile drilling and ensure the quality of the grouting, we recommend advancing the steel casing to the bottom of the micropile during the drilling operation. The steel casing may be withdrawn during the grouting operation while a minimum of 5 feet of grout head is maintained above the bottom of the casing at all times. The casing should be withdrawn to above the specified permanent casing tip elevation (minimum 3 feet) and plunged back to the design casing tip elevation to ensure proper grout cover around the permanent casing.

#### 3.2.1 Lateral Load Resistance

The lateral load capacity of a battered micropile will depend on the vertical load capacity and the batter angle. In this case, the micropiles will be subjected to a tension load. Based on the extreme event axial capacity and a batter of four horizontal to twelve vertical (4H:12V), a lateral load resistance of up to 120 kips per micropile may be used to resist the lateral load acting on the bridge structure.

#### 3.2.2 Micropile Load Test Program

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 between different contractors and micropile foundation systems. In order to determine whether the contractor's methods of micropile installation are adequate and to determine the ultimate axial load capacity, we recommend performing one pre-production tension load test on a sacrificial micropile at the Hilo abutment and one pre-production tension load test on a sacrificial micropile at the Honokaa abutment location for a total to two pre-production load tests for the project. In general, the purpose of the pre-production load tests on a 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 depths required.
- To confirm or modify the estimated minimum depth of the micropiles by determining the ultimate grout-to-soil bond stress.
- To assess the contractor’s method of drilling and grouting.

In general, the pre-production load tests should be performed in accordance with ASTM D3689, Standard Test Methods for Deep Foundations Under Static Axial Tensile Load. Based on experience, we believe the load test should be conducted no earlier than 7 days after completion of the micropile installation to allow the grout adequate time to cure.

The load test micropile should be loaded gradually to the maximum test load of at least 380 kips. The pre-production load test is an integral part of the design of the micropile foundation system. Therefore, we recommend a Geolabs representative observe the pre-production load tests.

In addition to the pre-production load tests, we also recommend performing pullout tests (proof tests) on selected micropiles during construction to confirm the load-carrying capacity of the installed micropiles. We recommend testing a minimum of 2 production micropiles at each abutment for pullout. The pullout tests should consist of subjecting the micropile to at least 255 kips. The micropile should be loaded in 25-kip load increments, and each load should be held for at least 5 minutes. The maximum test load should be held for a minimum of 10 or 60 minutes, depending on the recorded movements of the tested micropile. Pullout test on the selected micropiles is an integral part of the design of the micropile foundation system. Therefore, we also recommend conducting the pullout tests under the observation of a Geolabs representative.

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, a Geolabs representative (Special Inspector) should be present to observe the geotechnical aspects of the micropile foundation installation and testing.

#### 3.2.3 Micropile Construction Considerations

A specialty contractor experienced in the construction of a micropile foundation system (minimum five projects) should perform the installation of the micropiles. Saprolite and basalt formation were encountered in the borings within the embedment depths of the micropiles. The micropile contractor should anticipate hard drilling conditions during micropile construction.

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.

Due to the specialized nature of the micropile foundation construction, observation, and testing of the micropile foundation system should be designated as a “Special Inspection” item. Therefore, a Geolabs representative (Special Inspector) should be present to observe the geotechnical aspects of the micropile foundation installation and testing.

### 3.3 Soil Nails

Based on the information provided, we understand it is desired to retain the existing soil within the space between the abutment columns at both ends of the bridge. Therefore, we recommend constructing a soil-nailed retaining wall to retain the existing soil between the abutment columns. Construction of the permanent soil-nailed wall system should be performed by a specialty contractor experienced in the construction of soil-nailed walls. Due to the specialized nature of the soil-nailed wall construction, a Geolabs representative should be present to observe the geotechnical aspects of the soil-nailed wall and test the soil nails.

Items pertaining to the permanent soil nailed wall system are addressed in the subsequent subsections and include the following:

1. Soil Nailed Wall
2. Soil Nail Installation
3. Soil Nail Testing
4. Shotcrete Facing
5. Drainage

### 3.3.1 Soil Nailed Wall

The soil-nailed wall system consists of a series of individual reinforcing bars grouted into drilled holes used to stabilize the near vertical slope. Design of the soil-nailed wall system will need to consider both the internal and external stability of the reinforced mass. The design of the internal stability includes establishing the size, spacing, orientation, and length of the grouted reinforcing bars. The external stability includes slope stability of the reinforced mass.

The soil nails should be installed by drilling a minimum 6-inch diameter hole with an inclination of approximately 15 degrees from horizontal. The soil nail bar should consist of ASTM A615 Grade 75 threaded bar with a minimum bar diameter of 1.0 inches. We anticipated that the existing subsoil at the project site may be very corrosive. Therefore, we recommend using a double corrosion protection system for the nails. Galvanized or epoxy coated bars surrounded by neat cement grout or sand-cement mixture with a minimum 28-day compressive strength of 4,000 pounds per square inch (psi) may be considered.

Based on our soil nail analysis, we recommend using a design embedment nail length of 30 feet for the nails extending into the fill and residual/saprolite soils encountered in our field exploration. The soil nails should be spaced 5 feet on-center horizontally and vertically. The first nail should be installed about 2 feet below the bottom of the existing abutment beam.

### 3.3.2 Soil Nail Installation

Potentially difficult drilling conditions may be encountered during the installation of the soil nails due to the potential presence of medium hard to hard rock (in the form of relatively unweathered cobbles and boulders) within the residual and saprolite soils. In addition, utilizing a temporary casing may be required to maintain an open hole for the soil nail installation when encountering zones of very moist, medium stiff soils.

### 3.3.3 Soil Nail Testing

Due to the limited number of soil nails that will be installed, we believe the performance of pre-production verification testing on sacrificial soil nails is not needed. Proof tests should be performed on at least 10 percent of the production soil nails or a minimum of one proof test at each abutment during construction to confirm the bond stresses used in the design.

The proof tests should consist of subjecting the soil nail to at least 133 percent of the design load of 24 kips, and the load should be held for at least 10 minutes (until stable). The proof test nails may be incorporated into the permanent soil-nailed wall, provided the nail satisfies the test criteria. Pullout tests on the soil nails are integral parts of the design of the soil-nailed wall system. Therefore, a Geolabs representative should observe the pullout tests.

### 3.3.4 Shotcrete Facing

Shotcrete placement should be performed by an experienced nozzleman certified as a nozzleman for shotcrete placement by the American Concrete Institute (ACI). Prior to production shotcreting, it is recommended that unreinforced test panels (4-foot by 4-foot size by 4-inch-thick panels) of shotcrete be constructed for inspection.

### 3.3.5 Drainage

The soil-nailed wall should be well-drained to reduce the potential for the build-up of hydrostatic pressures. A drainage system consisting of 2-foot wide strips of a prefabricated drainage composite product should be installed on the face of the slope before the application of the shotcrete facing. The prefabricated drainage composite product should be installed extending from the top of the slope to the base of the slope and be hydraulically connected to weep holes at the base of the wall. In addition, the drainage strips should be spaced a minimum of about 8 feet on-center.

## 3.4 Site Preparation

At the on-set of earthwork, areas within the contract grading limits should be cleared and grubbed thoroughly. Vegetation, debris, deleterious materials, existing structures to be demolished, and other unsuitable materials should be removed and disposed of properly off-site to reduce the potential for contaminating the excavated materials.

Soft and yielding areas encountered during clearing and grubbing below areas designated to receive fill and/or future improvements should be over-excavated to expose firm natural material, and the resulting excavation should be backfilled with well-compacted fill. The excavated soft soils should not be reused as fill materials and should be properly disposed of off-site.

After clearing and grubbing, the exposed subgrades and areas designated to receive fills should be scarified to a depth of about 8 inches, moisture-conditioned to above the optimum moisture content, and recompacted 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 as determined by ASTM D1557. Optimum moisture is the water content (percentage by dry weight) corresponding to the maximum dry density.

In general, the excavated on-site materials should be suitable for use as general fill materials, provided that the maximum particle size is less than 3 inches in the largest dimension. General fills and backfills should be moisture-conditioned to above the optimum moisture content, placed in level lifts not exceeding 8 inches in loose thickness, and compacted to at least 95 percent relative compaction.

Imported materials, if required, should consist of select granular fill such as crushed basalt. The select granular fill should be well-graded from coarse to fine with no particles larger than 3 inches in the largest dimension. The material should have a California Bearing Ratio (CBR) value of 20 or higher and a swell potential of 1 percent or less when tested in accordance with AASHTO T193 (ASTM D1883). The material should also contain between 10 and 30 percent particles passing the No. 200 sieve. Imported fill materials should be tested for conformance with these recommendations prior to delivery to the project site for the intended use. Select granular fills should be moisture-conditioned to above the optimum moisture content, placed in level lifts not exceeding 8 inches in loose thickness, and compacted to at least 95 percent relative compaction. Imported fill materials should be tested and approved prior to delivery to the project site for the intended use.

## 3.5 Cut and Fill Slopes

Based on the subsurface conditions encountered in the borings, it appears that permanent cut slopes near the existing ground surface would expose the soil-like materials (residual soils and saprolite). In general, permanent cut slopes exposing the soil-like materials may be designed with a slope inclination of 2H:1V or flatter. Where cut slopes expose the dense basalt formation, the cut slope may be steepened to a slope inclination as steep as 0.5H:1V, if desired. Cavities that may be exposed on the cut slope face should be backfilled and grouted. We recommend that cut slopes exposing soil-like materials be immediately protected by appropriate slope planting or other means to reduce the potential for erosion of the exposed soils.

Permanent fill slopes constructed with either general fill materials or imported fill materials may be designed with a slope inclination of 2H:1V or flatter. Fills placed on slopes steeper than 5H:1V should be keyed and benched into the existing slope to provide stability for the new fill against sliding. The filling operations should start at the lowest point and continue up in level horizontal compacted layers in accordance with the above fill placement recommendations. Fill slopes should be constructed by overfilling and cutting back to the design slope ratio to obtain a well-compacted slope face. Water should be diverted away from the tops of slopes, and slope planting should be provided as soon as possible to reduce the potential for significant erosion of the finished slopes.

## 3.6 Design Review

Preliminary and final drawings and specifications for the proposed project should be forwarded to Geolabs for review and written comments prior to solicitation for construction bids. This review is necessary to evaluate the 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 assume responsibility for the misinterpretation of the recommendations presented herein.

### 3.7 Post-Design Services/ Services During Construction

It is highly recommended to retain Geolabs for geotechnical engineering support and continued services during construction. The following are critical items of construction monitoring that require "Special Inspections":

1. Review of micropile and soil nail installation submittals
2. Observation of the load test micropiles installation
3. Observation of the micropile load testing
4. Observation of the production micropile installation
5. Observation of the production soil nail installation
6. Observation of the micropile and soil nail proof testing
7. Observation of the subgrade soil preparation
8. Observation of fill placement and compaction

A Geolabs representative should monitor other aspects of the earthwork construction. This is 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 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

# 4. LIMITATIONS

The analyses and recommendations submitted herein are based in part upon information obtained from the field borings. Variations of the subsurface conditions between and beyond the field 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 presented herein.

The locations of the field borings indicated in this report are approximate, having been estimated by taping from visible features at the project site and using a handheld GPS device. Elevations noted on the borings were approximated using the Topographic Survey Map provided by KSF, Inc. on September 30, 2001. The locations and elevations of the field borings should be considered accurate only to the degree implied by the methods used.

The stratification breaks shown on the graphic representations of the borings depict the approximate boundaries between soil and/or rock types and, as such, may denote a gradual transition. Water level data from the borings were measured at times shown on the graphic representations and/or presented in the text of this report. This data has been reviewed, and interpretations made in the formulation of this report. However, it must be noted that fluctuation may occur due to stream flow conditions, tidal fluctuations, seasonal rainfall, time of year, and other environmental factors.

This report has been prepared for the exclusive use of KSF, Inc. and their project consultants for specific application to the *Seismic Retrofit of Kaholo Bridge, Hawaii Belt Road, Project No. BR-019-2(072)* 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 engineers in the design of the bridge seismic retrofit project. Therefore, this report may not contain sufficient data or the proper information to prepare construction cost estimates or contract bidding. A contractor wishing to bid on this project should retain a competent geotechnical engineer to assist in the interpretation of this report and/or performance of site-specific exploration for bid estimating purposes.

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

This geotechnical engineering exploration conducted at the project site was not intended to investigate the potential 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.

END OF LIMITATIONS

## CLOSURE

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

Project Location Map........................................................................................ Plate 1

Site Plan ............................................................................................................... Plate 2

Field Exploration ........................................................................................ Appendix A

Shear Wave Velocity Test .......................................................................... Appendix B

Laboratory Tests .......................................................................................... Appendix C

Photographs of Core Samples .................................................................... Appendix D

-ΩΩΩΩΩΩΩΩΩ-

Respectfully submitted,

**GEOLABS, INC.**

By ______________________

**Gerald Y. Seki, P.E.**
Vice-President

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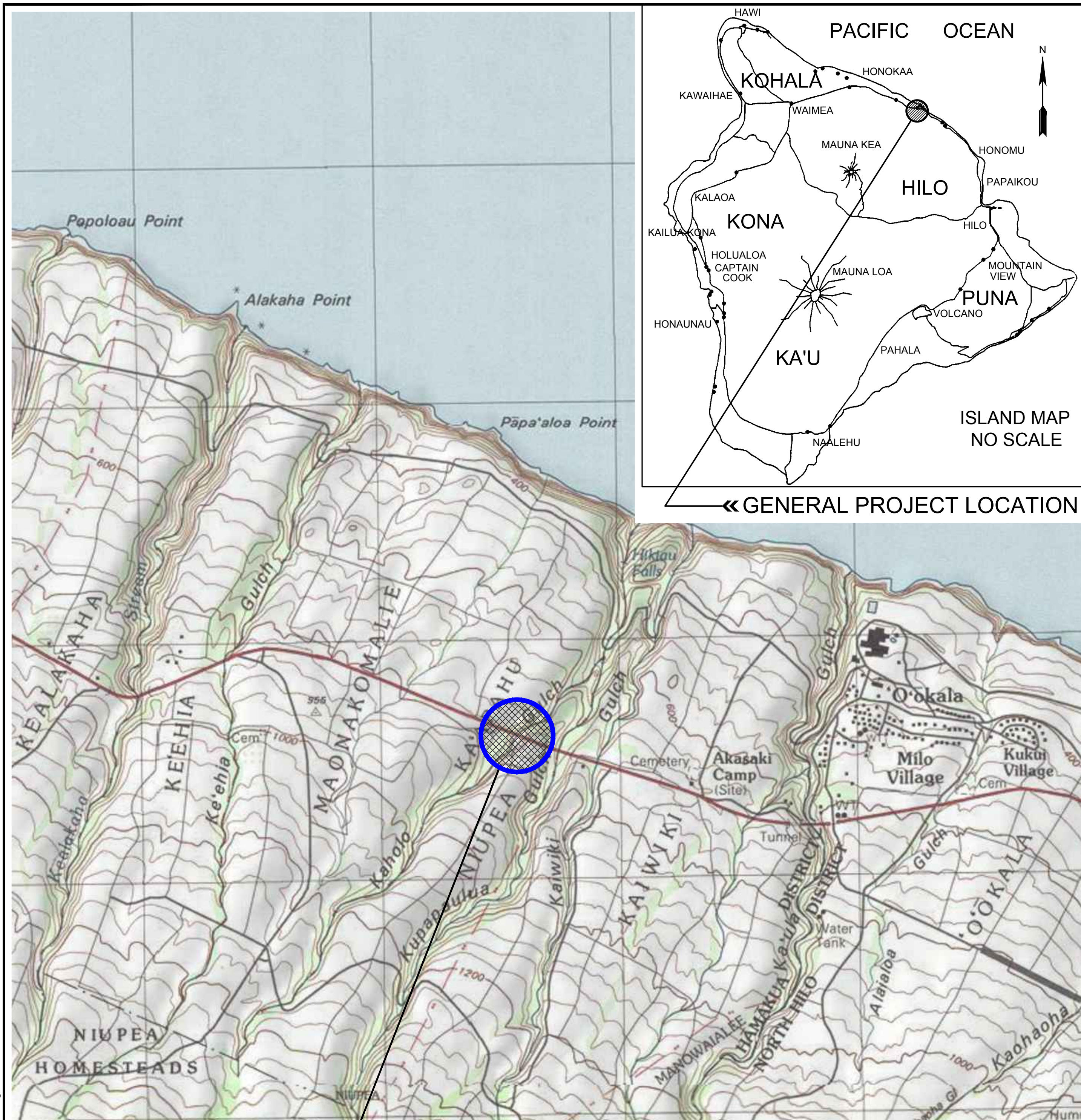
# PLATES




PROJECT LOCATION»

# PROJECT LOCATION MAP

SEISMIC RETROFIT OF KAHOLO BRIDGE
HAWAII BELT ROAD, PROJECT NO. BR-019-2(072)
DISTRICT OF HAMAKUA, ISLAND OF HAWAII

2000 1000 0 2000 FT.

GRAPHIC SCALE

N

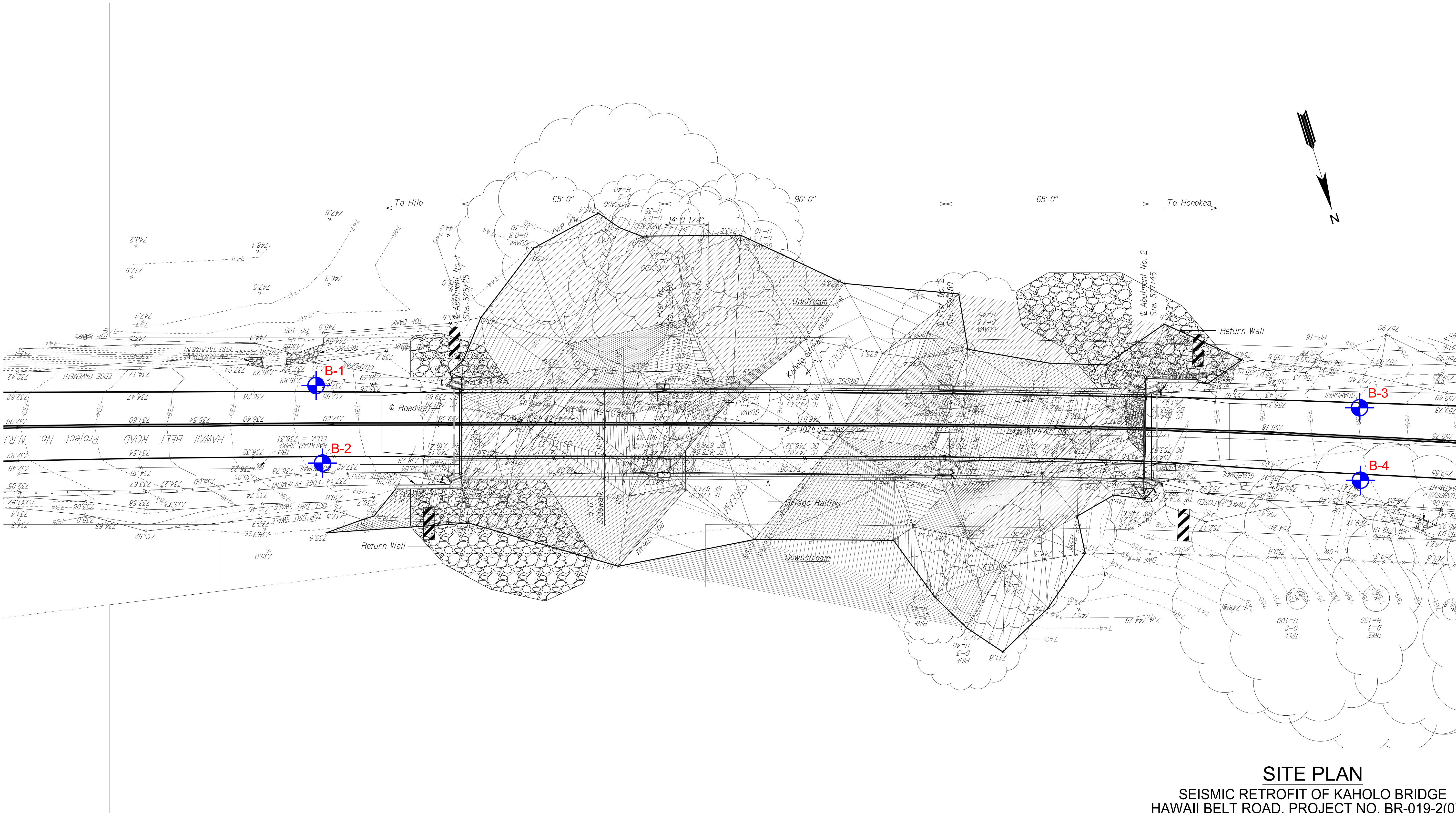
REFERENCE: MAP CREATED WITH TOPO!® ©2010 NATIONAL GEOGRAPHIC; ©2007 TELE ATLAS, REL. 1/2007.

GEOLABS, INC.

*Geotechnical Engineering*

| DATE | DRAWN BY | PLATE |
|---|---|---|
| AUGUST 2021 | HYC | |
| SCALE | W.O. | |
| 1" = 2,000' | 8063-00 | 1 |

CAD User: HENRY File Last Updated: August 04, 2021 11:00:16pm Plot Date: August 04, 2021 - 11:08:36pm
File: A:\Drafting\Drafting\Working\8063-00_Kaholo_Bridge\8063-00PLM.dwg\PLM
Plotter: DWG To PDF-GEO.pc3 Plotstyle: GEO-No-Dithering-Blue-Boring.ctb




LEGEND:

APPROXIMATE BORING LOCATION

REFERENCE: TOPOGRAPHIC SURVEY MAP PROVIDED BY KSF, INC. ON SEPTEMBER 30, 2021.

0 10 20 30 60 90 FT.

GRAPHIC SCALE

# SITE PLAN

SEISMIC RETROFIT OF KAHOLO BRIDGE
HAWAII BELT ROAD, PROJECT NO. BR-019-2(072)
DISTRICT OF HAMAKUA, ISLAND OF HAWAII

GEOLABS, INC.
*Geotechnical Engineering*

| DATE | DRAWN BY | PLATE |
|---|---|---|
| OCTOBER 2021 | HYC | 2 |
| SCALE | W.O. | |
| 1" = 30' | 8063-00 | |

# APPENDIX A

# APPENDIX A

Field Exploration

We explored the subsurface conditions at the project site by drilling and sampling four borings, designated as Boring Nos. 1 through 4, extending to depths of about 76 to 102.5 feet below the existing ground surface. The approximate boring locations are shown on the Site Plan, Plate 2. The borings were drilled using a truck-mounted drill rig equipped with continuous flight augers and coring tools.

Our engineers classified the materials encountered in the borings by visual and textural examination in the field in general accordance with ASTM D2488, 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 D2487, Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System), as shown on the Soil Log Legend, Plate A-0.1. Deviations made to the soil classification in accordance with ASTM D2487 are described on the Soil Classification Log Key, Plate A-0.2. Graphic representations of the materials encountered are presented on the Logs of Borings, Plates A-1.1 through A-4.3.

Relatively "undisturbed" soil samples were obtained in general accordance with ASTM D3550, 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 drilled borings in general accordance with ASTM D1586, 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.

Core samples of the rock materials encountered at the project site were obtained by using diamond core drilling techniques in general accordance with ASTM D2113, Diamond Core Drilling for Site Investigation. Core drilling is a rotary drilling method that uses a hollow bit to cut into the rock formation. The rock material left in the hollow core of the bit is mechanically recovered for examination and description. Rock cores were described in general accordance with the Rock Description System, as shown on the Rock Log Legend, Plate A-0.3. The Rock Description System is based on the publication "Suggested Methods for the Quantitative Description of Discontinuities in Rock Masses" by the International Society for Rock Mechanics (March 1977).

Recovery (REC) may be used as a subjective guide to the interpretation of the relative quality of rock masses (including intermediate geo-materials), where appropriate. Recovery is defined as the actual length of material recovered from a coring attempt versus the length of the core attempt. For example, if 3.7 feet of material is recovered from a 5.0-foot core run, the recovery would be 74 percent and would be shown on the Logs of Borings as REC = 74%.

The Rock Quality Designation (RQD) is also a subjective guide to the relative quality of rock masses. RQD is defined as the percentage of the core run in rock that is sound material in excess of 4 inches in length without any discontinuities, discounting any drilling, mechanical, and handling-induced fractures or breaks. If 2.5 feet of sound material is recovered from a 5.0-foot core run in rock, the RQD would be 50 percent and would be shown on the Logs of Borings as RQD = 50%. Generally, the following is used to describe the relative quality of the rock based on the "Practical Handbook of Physical Properties of Rocks and Minerals" by Robert S. Carmichael (1989).

| **Rock Quality** | **RQD** (%) |
|---|---|
| Very Poor | 0 – 25 |
| Poor | 25 – 50 |
| Fair | 50 – 75 |
| Good | 75 – 90 |
| Excellent | 90 – 100 |

The excavation characteristic of a rock mass is a function of the relative hardness of the rock, its relative quality, brittleness, and fissile characteristics. A dense rock formation with a high RQD value would be very difficult to excavate and probably would require more arduous methods of excavation.

# GEOLABS, INC.

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 AT TIME OF DRILLING
- WATER LEVEL OBSERVED IN BORING AFTER DRILLING
- WATER LEVEL OBSERVED IN BORING OVERNIGHT

| | |
|---|---|
| LL | LIQUID LIMIT (NP=NON-PLASTIC) |
| PI | PLASTICITY INDEX (NP=NON-PLASTIC) |
| TV | TORVANE SHEAR (tsf) |
| UC | UNCONFINED COMPRESSION OR UNIAXIAL COMPRESSIVE STRENGTH |
| TXUU | UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION (ksf) |



LOG LEGEND FOR SOIL 8063-00.GPJ GEOLABS.GDT 8/5/21

| | | |
|---|---|---|
| | **GEOLABS, INC.**<br>Geotechnical Engineering | Soil Classification Log Key<br>(with deviations from ASTM D2488) |

## GEOLABS, INC. CLASSIFICATION*

| GRANULAR SOIL (- #200 <50%) | COHESIVE SOIL (- #200 ≥50%) |
|---|---|
| • **PRIMARY** constituents are composed of the largest percent of the soil mass. Primary constituents are capitalized and bold (i.e., **GRAVEL, SAND**) | • **PRIMARY** constituents are based on plasticity. Primary constituents are capitalized and bold (i.e., **CLAY, SILT**) |
| • **SECONDARY** constituents are composed of a percentage less than the primary constituent. If the soil mass consists of 12 percent or more fines content, a cohesive constituent is used (**SILTY** or **CLAYEY**); otherwise, a granular constituent is used (**GRAVELLY** or **SANDY**) provided that the secondary constituent consists of 20 percent or more of the soil mass. Secondary constituents are capitalized and bold (i.e., **SANDY** GRAVEL, **CLAYEY** SAND) and precede the primary constituent. | • **SECONDARY** constituents are composed of a percentage less than the primary constituent, but more than 20 percent of the soil mass. Secondary constituents are capitalized and bold (i.e., **SANDY** CLAY, **SILTY** CLAY, **CLAYEY** SILT) and precede the primary constituent. |
| • **accessory descriptions** compose of the following:<br>with some: >12%<br>with a little: 5 - 12%<br>with traces of: <5%<br>accessory descriptions are lower cased and follow the Primary and Secondary Constituents<br>(i.e., SILTY GRAVEL **with a little sand**) | • **accessory descriptions** compose of the following:<br>with some: >12%<br>with a little: 5 - 12%<br>with traces of: <5%<br>accessory descriptions are lower cased and follow the Primary and Secondary Constituents<br>(i.e., SILTY CLAY **with some sand**) |

**EXAMPLE:** Soil Containing 60% Gravel, 25% Sand, 15% Fines. Described as: **SILTY GRAVEL** with some sand

## RELATIVE DENSITY / CONSISTENCY

| Granular Soils | | | Cohesive Soils | | | |
|---|---|---|---|---|---|---|
| N-Value (Blows/Foot) | | Relative Density | N-Value (Blows/Foot) | | PP Readings (tsf) | Consistency |
| SPT | MCS | | SPT | MCS | | |
| 0 - 4 | 0 - 7 | Very Loose | 0 - 2 | 0 - 4 | | Very Soft |
| 4 - 10 | 7 - 18 | Loose | 2 - 4 | 4 - 7 | < 0.5 | Soft |
| 10 - 30 | 18 - 55 | Medium Dense | 4 - 8 | 7 - 15 | 0.5 - 1.0 | Medium Stiff |
| 30 - 50 | 55 - 91 | Dense | 8 - 15 | 15 - 27 | 1.0 - 2.0 | Stiff |
| > 50 | > 91 | Very Dense | 15 - 30 | 27 - 55 | 2.0 - 4.0 | Very Stiff |
| | | | > 30 | > 55 | > 4.0 | Hard |

## MOISTURE CONTENT DEFINITIONS

Dry: Absence of moisture, dry to the touch

Moist: Damp but no visible water

Wet: Visible free water

## ABBREVIATIONS

WOH: Weight of Hammer

WOR: Weight of Drill Rods

SPT: Standard Penetration Test Split-Spoon Sampler

MCS: Modified California Sampler

PP: Pocket Penetrometer

## GRAIN SIZE DEFINITION

| Description | Sieve Number and / or Size |
|---|---|
| Boulders | > 12 inches (305-mm) |
| Cobbles | 3 to 12 inches (75-mm to 305-mm) |
| Gravel | 3-inch to #4 (75-mm to 4.75-mm) |
| Coarse Gravel | 3-inch to 3/4-inch (75-mm to 19-mm) |
| Fine Gravel | 3/4-inch to #4 (19-mm to 4.75-mm) |
| Sand | #4 to #200 (4.75-mm to 0.075-mm) |
| Coarse Sand | #4 to #10 (4.75-mm to 2-mm) |
| Medium Sand | #10 to #40 (2-mm to 0.425-mm) |
| Fine Sand | #40 to #200 (0.425-mm to 0.075-mm) |

**Soil descriptions are based on ASTM D2488-09a, Visual-Manual Procedure, with the above modifications by Geolabs, Inc. to the Unified Soil Classification System (USCS).*

Plate

A-0.2

SOIL_CLASS_LOG_KEY 8063-00.GPJ GEOLABS.GDT 8/5/21

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

## ROCK DESCRIPTIONS

| | | | |
|---|---|---|---|
| | **BASALT** | | **CONGLOMERATE** |
| | **BOULDERS** | | **LIMESTONE** |
| | **BRECCIA** | | **SANDSTONE** |
| | **CLINKER** | | **SILTSTONE** |
| | **COBBLES** | | **TUFF** |
| | **CORAL** | | **VOID/CAVITY** |

## ROCK DESCRIPTION SYSTEM

### ROCK FRACTURE CHARACTERISTICS

*The following terms describe general fracture spacing of a rock:*

| | |
|---|---|
| **Massive:** | Greater than 24 inches apart |
| **Slightly Fractured:** | 12 to 24 inches apart |
| **Moderately Fractured:** | 6 to 12 inches apart |
| **Closely Fractured:** | 3 to 6 inches apart |
| **Severely Fractured:** | Less than 3 inches apart |

### DEGREE OF WEATHERING

*The following terms describe the chemical weathering of a rock:*

| | |
|---|---|
| **Unweathered:** | Rock shows no sign of discoloration or loss of strength. |
| **Slightly Weathered:** | Slight discoloration inwards from open fractures. |
| **Moderately Weathered:** | Discoloration throughout and noticeably weakened though not able to break by hand. |
| **Highly Weathered:** | Most minerals decomposed with some corestones present in residual soil mass. Can be broken by hand. |
| **Extremely Weathered:** | Saprolite. Mineral residue completely decomposed to soil but fabric and structure preserved. |

### HARDNESS

*The following terms describe the resistance of a rock to indentation or scratching:*

| | |
|---|---|
| **Very Hard:** | Specimen breaks with difficulty after several "pinging" hammer blows.<br>Example: Dense, fine grain volcanic rock |
| **Hard:** | Specimen breaks with some difficulty after several hammer blows.<br>Example: Vesicular, vugular, coarse-grained rock |
| **Medium Hard:** | Specimen can be broked by one hammer blow. Cannot be scraped by knife. SPT may penetrate by ~25 blows per inch with bounce.<br>Example: Porous rock such as clinker, cinder, and coral reef |
| **Soft:** | Can be indented by one hammer blow. Can be scraped or peeled by knife. SPT can penetrate by ~100 blows per foot.<br>Example: Weathered rock, chalk-like coral reef |
| **Very Soft:** | Crumbles under hammer blow. Can be peeled and carved by knife. Can be indented by finger pressure.<br>Example: Saprolite |

**GEOLABS, INC.**

Geotechnical Engineering

SEISMIC RETROFIT OF KAHOLO BRIDGE
HAWAII BELT ROAD, PROJECT NO. BR-019-2(072)
DISTRICT OF HAMAKUA, ISLAND OF HAWAII

# Log of Boring 1

Approximate Ground Surface Elevation (feet ): 737 *

| 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 |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | 0 | | 8-inch **ASPHALTIC CONCRETE** |
| | | | | | | | | GM | Gray with some brown **SILTY GRAVEL (BASALTIC)**, medium dense, moist (fill) |
| | 52 | | | | 14 | | | ML | Orangish brown with gray mottling **CLAYEY SILT** with some sand (basaltic) and a little decomposed gravel, stiff to very stiff, moist (saprolite) |
| LL=43 PI=10 | | | | | 85 | 3.5 | 5 | | grades with cobble sized basalt corestones locally |
| TXUU $S_u$=1.7 ksf | 54 | 68 | | | 41 | 1.0 | 10 | | grades to medium stiff and very moist locally |
| Direct Shear | 65 | 62 | | | 17 | 0.8 | 15 | | |
| | | | | | | | | GM | Gray and brown **SILTY GRAVEL (BASALTIC)** with some sand, medium dense, moist (weathered basalt) |
| Sieve - #200 = 31.2% | 32 | 77 | | | 51 | | 20 | | |
| | | | 29 | | | | | | |
| | | | | | | | 25 | | |
| | 44 | | | | 23 | | | ML | Brown with gray mottling **CLAYEY SILT** with some sand (basaltic) and traces of gravel, very stiff, moist (saprolite) |
| | | | 33 | 0 | | | | | |
| | | | | | | | | | Brownish gray to gray vesicular **BASALT**, severely fractured, moderately to highly weathered, soft to medium hard (pahoehoe basalt) |
| | 39 | | | | 15/6" +25/3" | | 30 | | |
| | | | 47 | 0 | | | | | |
| | | | | | | | | ML | |
| | | | | | | | 35 | | |

| | | | | |
|---|---|---|---|---|
| Date Started: | May 13, 2021 | Water Level: | Not Encountered | Plate |
| Date Completed: | May 14, 2021 | | | |
| Logged By: | S. Latronic | Drill Rig: | MOBILE B-53.1 *(Energy Transfer Ratio = 42.9%)* | A - 1.1 |
| Total Depth: | 80.5 feet | Drilling Method: | 4" Solid-Stem Auger & HQ Coring | |
| Work Order: | 8063-00 | Driving Energy: | 140 lb. wt., 30 in. drop | |

| GEOLABS, INC. Geotechnical Engineering | SEISMIC RETROFIT OF KAHOLO BRIDGE<br>HAWAII BELT ROAD, PROJECT NO. BR-019-2(072)<br>DISTRICT OF HAMAKUA, ISLAND OF HAWAII | Log of Boring<br>1 |
|---|---|---|

| Laboratory | | | Field | | | | | | | | (Continued from previous plate) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Other Tests | Moisture Content (%) | Dry Density (pcf) | Core Recovery (%) | RQD (%) | Penetration Resistance (blows/foot) | Pocket Pen. (tsf) | Depth (feet) | Sample | Graphic | USCS | Description |
| | 35 | | | | 42 | | | | | ML | Brown with gray mottling **CLAYEY SILT** with some sand (basaltic) and traces of gravel, hard, moist (saprolite) |
| | | | 14 | | | | | | | | |
| | 30 | | | | 25/2" | | 40 | | | | |
| | | | 72 | 10 | | | | | | | Brownish gray to gray vesicular **BASALT**, closely fractured, moderately to highly weathered, medium hard (pahoehoe basalt) |
| UC=<br>660 psi | | | | | | | | | | | |
| | | | 55 | 13 | | | 45 | | | | |
| | | | | | | | | | | | grades to hard locally |
| | | | | | | | | | | | grades with small voids |
| | | | 57 | 0 | | | 50 | | | | |
| | | | | | | | | | | | Brownish gray to gray vugular **BASALT**, closely to severely fractured, moderately weathered, medium hard to hard (a'a basalt) |
| | | | | | | | 55 | | | GW | Brownish gray subangular **SANDY GRAVEL (BASALTIC)** with a little silt and cobbles, medium dense, moist (clinker) |
| | | | 35 | 0 | | | | | | | |
| | | | | | | | | | | | Gray vugular **BASALT**, closely to severely fractured, slightly weathered, hard (a'a basalt) |
| | | | 92 | 10 | | | 60 | | | | |
| | | | | | | | | | | | Brownish gray to gray vesicular **BASALT**, moderately to closely fractured, slightly to moderately weathered, medium hard to hard (pahoehoe basalt) |
| | | | | | | | 65 | | | | grades with highly weathered soft zones locally |
| | | | 100 | 50 | | | | | | | |
| UC=<br>4300 psi | | | | | | | | | | | |
| | | | 100 | 60 | | | | | | | |
| | | | | | | | 70 | | | | |

| | | | |
|---|---|---|---|
| Date Started: | May 13, 2021 | Water Level: ⊻ | Not Encountered |
| Date Completed: | May 14, 2021 | | |
| Logged By: | S. Latronic | Drill Rig: | MOBILE B-53.1 *(Energy Transfer Ratio = 42.9%)* |
| Total Depth: | 80.5 feet | Drilling Method: | 4" Solid-Stem Auger & HQ Coring |
| Work Order: | 8063-00 | Driving Energy: | 140 lb. wt., 30 in. drop |

| GEOLABS, INC. Geotechnical Engineering | SEISMIC RETROFIT OF KAHOLO BRIDGE HAWAII BELT ROAD, PROJECT NO. BR-019-2(072) DISTRICT OF HAMAKUA, ISLAND OF HAWAII | Log of Boring 1 |
|---|---|---|

(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 |
| | | | 83 | 25 | | | | | | | |
| | | | 93 | 52 | | | 75 | | | | |
| | | | | | | | | | | | grades to very hard |
| | | | | | | | 80 | | | | |
| | | | | | | | | | | | Boring terminated at 80.5 feet<br><br>* Elevations estimated from Topographic Survey Map provided by KSF, Inc. on September 30, 2021. |
| | | | | | | | 85 | | | | |
| | | | | | | | 90 | | | | |
| | | | | | | | 95 | | | | |
| | | | | | | | 100 | | | | |
| | | | | | | | 105 | | | | |

| | | | | |
|---|---|---|---|---|
| Date Started: | May 13, 2021 | Water Level: | Not Encountered | Plate A - 1.3 |
| Date Completed: | May 14, 2021 | | | |
| Logged By: | S. Latronic | Drill Rig: | MOBILE B-53.1 *(Energy Transfer Ratio = 42.9%)* | |
| Total Depth: | 80.5 feet | Drilling Method: | 4" Solid-Stem Auger & HQ Coring | |
| Work Order: | 8063-00 | Driving Energy: | 140 lb. wt., 30 in. drop | |

BORING_LOG 8063-00.GPJ GEOLABS.GDT 12/13/23

| GEOLABS, INC. | SEISMIC RETROFIT OF KAHOLO BRIDGE |  Log of Boring |
|---|---|---|
| Geotechnical Engineering | HAWAII BELT ROAD, PROJECT NO. BR-019-2(072)<br>DISTRICT OF HAMAKUA, ISLAND OF HAWAII | 2 |

Approximate Ground Surface Elevation (feet ): 737 *

| 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 |
| | | | | | | | 0 | | | | 9-inch **ASPHALTIC CONCRETE** |
| | 18 | 100 | | | 24 | | | | | GP | Brownish gray **GRAVEL (BASALTIC)** with some sand, medium dense, moist (fill) |
| | 43 | | | | 8 | | | | | GP | Dark gray **GRAVELLY SAND** with some silt, medium dense to loose, moist (fill) |
| TXUU $S_u$=1.3 ksf | 55 | 66 | | | 11 | | 5 | | | MH | Brown **CLAYEY SILT** with some sand, medium stiff, wet (residual soil) |
| | 66 | | | | 6 | | 10 | | | | grades with white mottling |
| Direct Shear | 71 | 60 | | | 13 | | 15 | | | | |
| | | | 0 | | | | | | | | |
| | | | | | | | 20 | | | | |
| | 18 | | | | 31 | | | | | | grades with some gravel, hard |
| | | | 0 | | | | 25 | | | | |
| | 21 | | | | 45 | | | | | | |
| | | | 43 | 0 | | | 30 | | | | Gray with orangish mottling vesicular **BASALT**, severely fractured, highly to moderately weathered, medium hard to hard (basalt formation) |
| | | | 0 | 0 | 30/1" | | | | | | |
| | | | | | | | 35 | | | | |

| | | | | |
|---|---|---|---|---|
| Date Started: | May 4, 2021 | Water Level: ⊻ | Not Encountered | Plate |
| Date Completed: | May 6, 2021 | | | |
| Logged By: | D. Gremminger | Drill Rig: | MOBILE B-53.1 *(Energy Transfer Ratio = 42.9%)* | A - 2.1 |
| Total Depth: | 102.5 feet | Drilling Method: | 4" Solid-Stem Auger & HQ Coring | |
| Work Order: | 8063-00 | Driving Energy: | 140 lb. wt., 30 in. drop | |

(Continued from previous plate)

| Other Tests (Laboratory) | Moisture Content (%) | Dry Density (pcf) | Core Recovery (%) (Field) | RQD (%) | Penetration Resistance (blows/foot) | Pocket Pen. (tsf) | Depth (feet) | USCS | Description |
|---|---|---|---|---|---|---|---|---|---|
| | 28 | | | | 31 | | | | breaks down to a sandy gravel (basaltic), hard |
| | | | 14 | 0 | | | 40 | ML | Brownish gray **GRAVELLY SILT** with some sand, hard, wet (saprolite) |
| Sieve - #200 = 52.5% | 43 | | | | 32 | | | | |
| | | | 0 | 0 | | | 45 | | |
| | 26 | | | | 77 | | | | |
| | | | 26 | 0 | | | 50 | | Gray vesicular **BASALT**, severely fractured, highly to moderately weathered, medium hard to hard (basalt formation) |
| | | | 22 | 0 | 45/2" | | | | |
| | | | | | | | 55 | | |
| | | | 7 | 0 | 25/1" | | | | |
| | | | | | | | 60 | | |
| | | | 88 | 64 | 50/4" | | | | Gray **BASALT**, severely to moderately fractured, moderately weathered, hard (basalt formation) |
| | | | | | | | 65 | | |
| | | | 80 | 42 | | | | | |
| | | | | | | | 70 | | |

| | |
|---|---|
| Date Started: | May 4, 2021 |
| Date Completed: | May 6, 2021 |
| Logged By: | D. Gremminger |
| Total Depth: | 102.5 feet |
| Work Order: | 8063-00 |

| | |
|---|---|
| Water Level: ⊻ | Not Encountered |
| Drill Rig: | MOBILE B-53.1 *(Energy Transfer Ratio = 42.9%)* |
| Drilling Method: | 4" Solid-Stem Auger & HQ Coring |
| Driving Energy: | 140 lb. wt., 30 in. drop |

(Continued from previous plate)

| Other Tests | Moisture Content (%) | Dry Density (pcf) | Core Recovery (%) | RQD (%) | Penetration Resistance (blows/foot) | Pocket Pen. (tsf) | Depth (feet) | Sample | Graphic | USCS | Description |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | 100 | 62 | | | | | | | |
| UC= 1230 psi | | | | | | | 75 | | | | |
| | | | 100 | 85 | | | | | | | grades to closely to slightly fractured |
| | | | | | | | 80 | | | | |
| | | | 62 | 22 | | | | | | | grades to severely to closely fractured |
| | | | | | | | 85 | | | | |
| | | | 52 | 23 | | | | | | | |
| | | | | | | | 90 | | | GM | Reddish brown **SANDY GRAVEL (BASALTIC)**, dense, wet (clinker) |
| | | | 70 | 33 | | | | | | | |
| UC= 9080 psi | | | | | | | 95 | | | | Gray **BASALT**, severely to moderately fractured, highly to moderately weathered, medium hard to hard (basalt formation) |
| | | | 62 | 22 | | | | | | | |
| | | | | | | | 100 | | | | |
| | | | | | | | | | | | Boring terminated at 102.5 feet |
| | | | | | | | 105 | | | | |

| | | | |
|---|---|---|---|
| Date Started: | May 4, 2021 | Water Level: ⊻ | Not Encountered |
| Date Completed: | May 6, 2021 | | |
| Logged By: | D. Gremminger | Drill Rig: | MOBILE B-53.1 *(Energy Transfer Ratio = 42.9%)* |
| Total Depth: | 102.5 feet | Drilling Method: | 4" Solid-Stem Auger & HQ Coring |
| Work Order: | 8063-00 | Driving Energy: | 140 lb. wt., 30 in. drop |

| GEOLABS, INC. Geotechnical Engineering | SEISMIC RETROFIT OF KAHOLO BRIDGE<br>HAWAII BELT ROAD, PROJECT NO. BR-019-2(072)<br>DISTRICT OF HAMAKUA, ISLAND OF HAWAII | Log of Boring 3 |
|---|---|---|

Approximate Ground Surface Elevation (feet ): 758 *

| Other Tests | Moisture Content (%) | Dry Density (pcf) | Core Recovery (%) | RQD (%) | Penetration Resistance (blows/foot) | Pocket Pen. (tsf) | Depth (feet) | USCS | Description |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | 8-inch **ASPHALTIC CONCRETE** |
| | | | | | | | | GM | Gray and brown **SILTY GRAVEL (BASALTIC)**, medium dense, moist (fill) |
| | 46 | | | | 57 | | | MH | Dark brown **CLAYEY SILT** with a little gravel (basaltic), hard, moist (fill) |
| | 37 | 74 | | | 41 | 2.0 | 5 | MH | Orangish brown with gray mottling **CLAYEY SILT** with some sand (basaltic) and a little decomposed gravel, very stiff, moist (saprolite) |
| LL=77<br>PI=27<br>TXUU<br>$S_u$=2.1 ksf<br>Sieve<br>- #200 =<br>9.0% | 29 | 80 | | | 35 | 3.0 | 10 | | grades with gravel |
| TXUU<br>$S_u$=2.2 ksf | 67 | 56 | | | 13 | 0.8 | 15 | | grades to medium stiff and very moist locally |
| Direct<br>Shear<br>Sieve<br>- #200 =<br>93.9% | 68 | 57 | | | 31 | 2.5 | 20 | | grades with a little sand, very stiff |
| | | | 43 | | | | | | |
| LL=60<br>PI=12 | 83 | | | | 20 | | 25 | | grades to reddish brown locally |
| | | | 36 | | | | | | |
| | 43 | | | | 7 | | 30 | | grades to medium stiff |
| | | | 0 | | | | 35 | | |

| | | | |
|---|---|---|---|
| Date Started: | May 12, 2021 | Water Level: | Not Encountered |
| Date Completed: | May 13, 2021 | | |
| Logged By: | S. Latronic | Drill Rig: | MOBILE B-53.1 *(Energy Transfer Ratio = 42.9%)* |
| Total Depth: | 91 feet | Drilling Method: | 4" Solid-Stem Auger & HQ Coring |
| Work Order: | 8063-00 | Driving Energy: | 140 lb. wt., 30 in. drop |

Plate A - 3.1

| | | | |
|---|---|---|---|
| GEOLABS, INC. Geotechnical Engineering | SEISMIC RETROFIT OF KAHOLO BRIDGE<br>HAWAII BELT ROAD, PROJECT NO. BR-019-2(072)<br>DISTRICT OF HAMAKUA, ISLAND OF HAWAII | Log of Boring | 3 |

(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 |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | MH | |
| | 64 | | | | 23 | | | | grades to very stiff |
| | | | 14 | | | | | | |
| | | | | | | | 40 | | |
| | 40 | | | | 52 | | | | grades to brownish gray, hard |
| | | | 42 | 0 | | | | | |
| | | | | | | | | | Gray vugular **BASALT**, closely to severely fractured, slightly to moderately weathered, medium hard to hard (a'a basalt) |
| | | | | | | | 45 | ML | |
| | 72 | | | | 16 | | | | Brownish gray **CLAYEY SILT** with some sand (basaltic), very stiff, moist (saprolite) |
| | | | 72 | 28 | | | | | |
| | | | | | | | | | Gray vesicular **BASALT**, moderately fractured, slightly to moderately weathered, medium hard to hard (pahoehoe basalt) |
| | | | | | | | 50 | | |
| UC= 600 psi | | | 92 | 12 | | | | | |
| | | | | | | | | | Brownish gray vesicular **BASALT**, closely to severely fractured, highly weathered, soft (pahoehoe basalt) |
| | | | | | | | 55 | | |
| | | | | | | | | | Gray vugular **BASALT**, closely fractured, slightly to moderately weathered, medium hard to hard (a'a basalt) |
| | | | 93 | 37 | | | | | |
| | | | | | | | | ML | Brownish gray **SANDY SILT** with some decomposed gravel, stiff, moist (saprolite) |
| | | | | | | | 60 | | Brownish gray vesicular **BASALT**, moderately fractured, slightly to moderately weathered, medium hard to hard (pahoehoe basalt) |
| UC= 3040 psi | | | 88 | 35 | | | | | |
| | | | | | | | 65 | | |
| | | | 57 | 23 | | | | | |
| | | | | | | | | | grades with severely fractured, highly weathered soft zones locally |
| | | | | | | | 70 | | |

| | | | | |
|---|---|---|---|---|
| Date Started: | May 12, 2021 | Water Level: | Not Encountered | Plate |
| Date Completed: | May 13, 2021 | | | |
| Logged By: | S. Latronic | Drill Rig: | MOBILE B-53.1 *(Energy Transfer Ratio = 42.9%)* | A - 3.2 |
| Total Depth: | 91 feet | Drilling Method: | 4" Solid-Stem Auger & HQ Coring | |
| Work Order: | 8063-00 | Driving Energy: | 140 lb. wt., 30 in. drop | |

| | | |
|---|---|---|
| GEOLABS, INC.<br>Geotechnical Engineering | SEISMIC RETROFIT OF KAHOLO BRIDGE<br>HAWAII BELT ROAD, PROJECT NO. BR-019-2(072)<br>DISTRICT OF HAMAKUA, ISLAND OF HAWAII | Log of Boring<br>3 |

(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 |
|---|---|---|---|---|---|---|---|---|---|
| | | | 72 | 28 | | | | | |
| | | | | | | | 75 | | Gray vugular **BASALT**, moderately fractured, slightly weathered, hard to very hard (a'a basalt) |
| | | | 75 | 42 | | | | GM | Grayish brown **SILTY GRAVEL (BASALTIC)**, medium dense, moist (clinker) |
| | | | | | | | 80 | | Gray vugular **BASALT**, moderately fractured, slightly weathered, very hard (a'a basalt) |
| | | | 72 | 33 | | | | GM | Grayish brown **SILTY GRAVEL (BASALTIC)**, medium dense, moist (clinker) |
| | | | | | | | 85 | | Gray vugular **BASALT**, moderately fractured, slightly weathered, very hard (a'a basalt) |
| | | | 93 | 67 | | | | | grades to vesicular locally |
| | | | | | | | 90 | | |
| | | | | | | | | | Boring terminated at 91 feet |
| | | | | | | | 95 | | |
| | | | | | | | 100 | | |
| | | | | | | | 105 | | |

| | | | | |
|---|---|---|---|---|
| Date Started: | May 12, 2021 | Water Level: | Not Encountered | Plate |
| Date Completed: | May 13, 2021 | | | |
| Logged By: | S. Latronic | Drill Rig: | MOBILE B-53.1 *(Energy Transfer Ratio = 42.9%)* | A - 3.3 |
| Total Depth: | 91 feet | Drilling Method: | 4" Solid-Stem Auger & HQ Coring | |
| Work Order: | 8063-00 | Driving Energy: | 140 lb. wt., 30 in. drop | |

| | | | |
|---|---|---|---|
| **GEOLABS, INC.**<br>Geotechnical Engineering | SEISMIC RETROFIT OF KAHOLO BRIDGE<br>HAWAII BELT ROAD, PROJECT NO. BR-019-2(072)<br>DISTRICT OF HAMAKUA, ISLAND OF HAWAII | Log of Boring | 4 |

Approximate Ground Surface Elevation (feet ): 758 *

| 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 |
| | | | | | | | 0 | | | | 4-inch **ASPHALTIC CONCRETE** |
| | | | | | | | | | | GM | Gray and brown **SILTY GRAVEL (BASALTIC)**, medium dense, moist (fill) |
| | | | | | | | | | | MH | Dark brown **CLAYEY SILT** with a little gravel (basaltic), stiff to very stiff, moist (fill) |
| | 25 | | | | 15 | | | | | | |
| LL=176<br>PI=91 | 76 | 48 | | | 26 | 1.8 | 5 | | | MH | Brown with orange mottling **CLAYEY SILT** with some sand (basaltic) and traces of decomposed gravel, stiff, moist (saprolite) |
| TXUU<br>$S_u$=1.3 ksf | 69 | 56 | | | 11 | 0.8 | 10 | | | | grades to medium stiff and very moist locally |
| LL=32<br>PI=NP | 40 | | | | 46 | | 15 | | | ML | Brown with orange mottling **CLAYEY SILT** with some sand and decomposed cobble corestones locally, hard, very moist |
| Direct Shear | 46 | 83 | | | 50/5" | | 20 | | | | grades with gravel |
| | | | 8 | | | | | | | | |
| | | | | | | | 25 | | | | |
| Sieve - #200 = 41.4% | 39 | | | | 58 | | | | | GM | Brown with orange mottling **SILTY GRAVEL** with some sand, very dense to dense, very moist (basalt formation) |
| | | | 25 | | | | | | | | |
| | | | | | | | 30 | | | | |
| | 36 | | | | 31 | | | | | | |
| | | | 0 | 0 | | | | | | | |
| | | | | | | | 35 | | | | |

| | | | | |
|---|---|---|---|---|
| Date Started: | May 10, 2021 | Water Level: | Not Encountered | Plate |
| Date Completed: | May 11, 2021 | | | |
| Logged By: | S. Latronic | Drill Rig: | MOBILE B-53.1 *(Energy Transfer Ratio = 42.9%)* | A - 4.1 |
| Total Depth: | 76 feet | Drilling Method: | 4" Solid-Stem Auger & HQ Coring | |
| Work Order: | 8063-00 | Driving Energy: | 140 lb. wt., 30 in. drop | |

# GEOLABS, INC.

Geotechnical Engineering

SEISMIC RETROFIT OF KAHOLO BRIDGE
HAWAII BELT ROAD, PROJECT NO. BR-019-2(072)
DISTRICT OF HAMAKUA, ISLAND OF HAWAII

**Log of Boring 4**

| 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 |
| | | | | | | | | | | GM | grades to highly weathered basalt |
| | 15 | | 57 | 7 | 50/2" | | | | | | Brownish gray vesicular **BASALT**, severely fractured, moderately to highly weathered, soft to medium hard (pahoehoe basalt) |
| | | | | | | | | | | | Gray dense **BASALT**, moderately fractured, slightly to moderately weathered, hard to very hard (a'a basalt) |
| | | | | | | | 40 | | | GM | |
| | | | 52 | 13 | | | | | | | Brownish gray **SILTY GRAVEL (BASALTIC)**, medium dense, moist (clinker) |
| | | | | | | | | | | | Grayish brown vesicular **BASALT**, severely fractured, moderately to highly weathered, soft to medium hard (pahoehoe basalt) |
| | | | | | | | 45 | | | | Brownish gray vugular **BASALT**, closely fractured, slightly to moderately weathered, medium hard to hard (a'a basalt) |
| | 29 | | 76 | 33 | 50/2" | | | | | | |
| UC= 420 psi | | | | | | | | | | | Grayish brown vesicular **BASALT**, moderately fractured, moderately to highly weathered, medium hard (pahoehoe basalt) |
| | | | | | | | 50 | | | | |
| | | | 95 | 20 | | | | | | | Gray vugular **BASALT**, moderately fractured, slightly weathered, hard (a'a basalt) |
| | | | | | | | 55 | | | ML | Reddish brown with gray mottling **SANDY SILT** with a little gravel (basaltic), stiff, moist (saprolite) |
| | | | 95 | 50 | | | | | | | |
| | | | | | | | | | | | Gray vugular **BASALT**, moderately fractured, slightly weathered, hard (a'a basalt) |
| UC= 2230 psi | | | | | | | | | | | |
| | | | | | | | 60 | | | | |
| | | | 95 | 58 | | | | | | | Brownish gray vesicular **BASALT**, moderately fractured, slightly to moderately weathered, medium hard to hard (pahoehoe basalt) |
| | | | | | | | 65 | | | | |
| | | | 57 | 35 | | | | | | | grades to dense |
| | | | | | | | | | | | **VOID** |
| | | | | | | | 70 | | | | |

| | | | |
|---|---|---|---|
| Date Started: | May 10, 2021 | Water Level: ⊻ | Not Encountered |
| Date Completed: | May 11, 2021 | | |
| Logged By: | S. Latronic | Drill Rig: | MOBILE B-53.1 *(Energy Transfer Ratio = 42.9%)* |
| Total Depth: | 76 feet | Drilling Method: | 4" Solid-Stem Auger & HQ Coring |
| Work Order: | 8063-00 | Driving Energy: | 140 lb. wt., 30 in. drop |

Plate

A - 4.2

BORING_LOG 8063-00.GPJ GEOLABS.GDT 12/13/23

| GEOLABS, INC. Geotechnical Engineering | SEISMIC RETROFIT OF KAHOLO BRIDGE<br>HAWAII BELT ROAD, PROJECT NO. BR-019-2(072)<br>DISTRICT OF HAMAKUA, ISLAND OF HAWAII | Log of Boring 4 |
|---|---|---|

(Continued from previous plate)

| Other Tests | Moisture Content (%) | Dry Density (pcf) | Core Recovery (%) | RQD (%) | Penetration Resistance (blows/foot) | Pocket Pen. (tsf) | Depth (feet) | Sample | Graphic | USCS | Description |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | 63 | 0 | | | | | | | Gray vugular **BASALT**, moderately fractured, slightly weathered, hard (a'a basalt) |
| | | | | | | | 75 | | | SM | Brown with some gray **SILTY SAND (BASALTIC)** with some gravel, medium dense, moist (saprolite) |
| | | | | | | | | | | | Boring terminated at 76 feet |
| | | | | | | | 80 | | | | |
| | | | | | | | 85 | | | | |
| | | | | | | | 90 | | | | |
| | | | | | | | 95 | | | | |
| | | | | | | | 100 | | | | |
| | | | | | | | 105 | | | | |

| | | | | |
|---|---|---|---|---|
| Date Started: | May 10, 2021 | Water Level: ▼ | Not Encountered | Plate A - 4.3 |
| Date Completed: | May 11, 2021 | | | |
| Logged By: | S. Latronic | Drill Rig: | MOBILE B-53.1 *(Energy Transfer Ratio = 42.9%)* | |
| Total Depth: | 76 feet | Drilling Method: | 4" Solid-Stem Auger & HQ Coring | |
| Work Order: | 8063-00 | Driving Energy: | 140 lb. wt., 30 in. drop | |

# APPENDIX B

# APPENDIX B

Seismic Shear Wave Velocity Test

Seismic shear wave velocity profiling of the subsurface materials at the project site was performed using Seismic Cone Penetration Testing (SCPT) equipment. The purpose of the seismic shear wave velocity profiling of the subsurface materials was to analyze the seismic design considerations more closely for the project. Shear wave velocity testing using seismic cone penetration test equipment was performed at a selected boring location, designated as Boring No. B-2, as shown on the Site Plan (Plate 2). The seismic shear wave velocity profiling was performed at various depths, extending to a depth of about 101.7 feet below the existing ground surface.

In order to conduct the seismic shear wave velocity test in the boring, the test boring was advanced, utilizing rotary coring methods to the maximum depth of the boring. Logs of materials encountered in the boring are presented on the Logs of Borings in Appendix A. After the boring was advanced to the maximum depth of the borehole, the bored hole was backfilled with 0.25-inch diameter coated bentonite pellets. The temporary casing from the coring operations was used as a tremie pipe to place the bentonite pellets, starting from the bottom and advancing upward. When the bentonite pellets are in contact with the groundwater in the borehole, the pellets start to hydrate slowly. As the bentonite pellets hydrate, they swell and soften. The probe was then pushed through the softened bentonite, extending to a depth of about 101.7 feet below the existing ground surface using seismic cone testing equipment (SCPT).

The seismic shear wave velocity test consists of hydraulically pushing a 10-ton steel electronic subtraction cone with an apex angle of 60 degrees and a projected surface area of 1.55 square inches (10 square centimeters) into the bored hole. The cone carries a uniaxial horizontal accelerator geophone to detect the arrival of a shear wave generated and propagated from the ground surface. The seismic measurements were made when the SCPT had stopped and a shear wave was sent into the subsurface. A shear wave was generated at the surface by striking a loaded plank with a switched hammer. The propagation time of the wave from the hammer blow to the cone was measured at each discrete depth interval. The vector difference of these depths divided by the time difference for the shear wave to arrive at the various depths provided the average shear wave velocity over the depth interval.

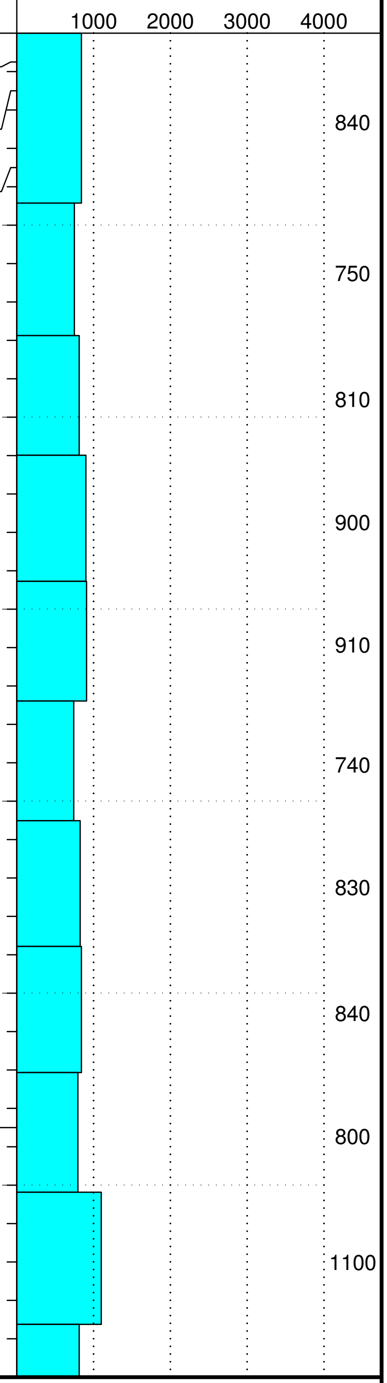
The seismic shear wave velocities measured and the weighted average seismic shear wave velocity calculated for the top 100 feet of the soil and rock profile at the selected boring location are presented on Plate B-1.1 through B-1.4 in Appendix B. The weighted average shear wave velocity was calculated based on the average shear wave velocity method described in Section 20.4.1 of ASCE 7-16.

| GEOLABS, INC. | SEISMIC RETROFIT OF KAHOLO BRIDGE | Data Plot of Boring |
|---|---|---|
| Geotechnical Engineering | HAWAII BELT ROAD, PROJECT NO. BR-019-2(072)<br>DISTRICT OF HAMAKUA, ISLAND OF HAWAII | 2 |

Approximate Ground Surface Elevation :

| Core Recovery (%) | Penetration Resistance (blows/foot) | Pocket Pen. (tsf) | Depth (feet) | USCS | Description |
|---|---|---|---|---|---|
| | | | | | 9-inch **ASPHALTIC CONCRETE** |
| | 24 | | | GP | Brownish gray **GRAVEL (BASALTIC)** with some sand, medium dense, moist (fill) |
| | 8 | | | GP | Dark gray **GRAVELLY SAND** with some silt, loose, moist (fill) |
| | 11 | | 5 | MH | Brown **CLAYEY SILT** with some sand, medium stiff, wet (residual soil) |
| | 6 | | 10 | | grades with white mottling |
| | 13 | | 15 | | |
| 0 | | | | | |
| | | | 20 | | |
| | 31 | | | | grades with some gravel, very stiff |
| | | | 25 | | |
| | 45 | | | | |
| 43 | | | | | Gray with orangish mottling vesicular **BASALT**, severely fractured, highly to moderately weathered, medium hard to hard (basalt formation) |
| | | | 30 | | |
| 0 | 30/1" | | | | |
| | | | 35 | | |

SHEAR WAVE VELOCITY (feet/sec)




| | | | |
|---|---|---|---|
| Date Started: | May 4, 2021 | Water Level: ⊻ | Not Encountered |
| Date Completed: | May 6, 2021 | | |
| Logged By: | D. Gremminger | Drill Rig: | MOBILE B-53.1 |
| Total Depth: | 102.5 feet | Drilling Method: | 4" Solid-Stem Auger & HQ Coring |
| Work Order: | 8063-00 | Driving Energy: | 140 lb. wt., 30 in. drop |

Plate

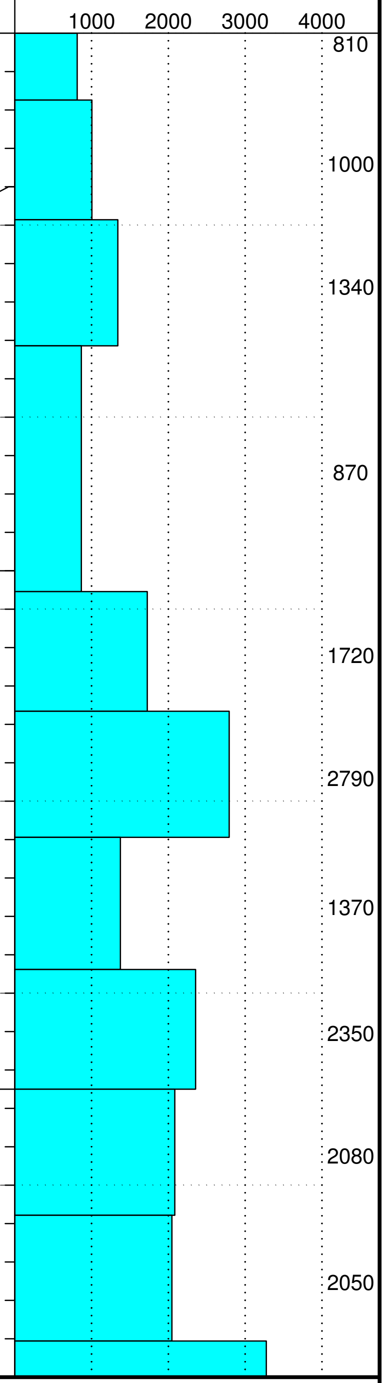
B - 1.1

SHEAR_WAVE_PLOT 8063-00.GPJ GEOLABS.GDT 10/1/21

| GEOLABS, INC. Geotechnical Engineering | SEISMIC RETROFIT OF KAHOLO BRIDGE HAWAII BELT ROAD, PROJECT NO. BR-019-2(072) DISTRICT OF HAMAKUA, ISLAND OF HAWAII | Data Plot of Boring 2 |
|---|---|---|

| Core Recovery (%) | Penetration Resistance (blows/foot) | Pocket Pen. (tsf) | Depth (feet) | Sample | Graphic | USCS | (Continued from previous plate) Description |
|---|---|---|---|---|---|---|---|
| | 31 | | | | | | breaks down to a sandy gravel (basaltic), medium dense |
| 14 | | | 40 | | | ML | Brownish gray **GRAVELLY SILT** with some sand, medium dense, wet (saprolite) |
| | 32 | | | | | | |
| 0 | | | 45 | | | | |
| | 77 | | | | | | grades to very hard |
| 26 | | | 50 | | | | Gray vesicular **BASALT**, severely fractured, highly to moderately weathered, medium hard to hard (basalt formation) |
| 22 | 45/2" | | | | | | |
| | | | 55 | | | | |
| 7 | 25/1" | | | | | | |
| | | | 60 | | | | |
| 88 | 50/4" | | | | | | Gray **BASALT**, severely to moderately fractured, moderately weathered, hard (basalt formation) |
| | | | 65 | | | | |
| 80 | | | | | | | |
| | | | 70 | | | | |

SHEAR WAVE VELOCITY (feet/sec)




| | | | |
|---|---|---|---|
| Date Started: | May 4, 2021 | Water Level: ⊻ | Not Encountered |
| Date Completed: | May 6, 2021 | | |
| Logged By: | D. Gremminger | Drill Rig: | MOBILE B-53.1 |
| Total Depth: | 102.5 feet | Drilling Method: | 4" Solid-Stem Auger & HQ Coring |
| Work Order: | 8063-00 | Driving Energy: | 140 lb. wt., 30 in. drop |

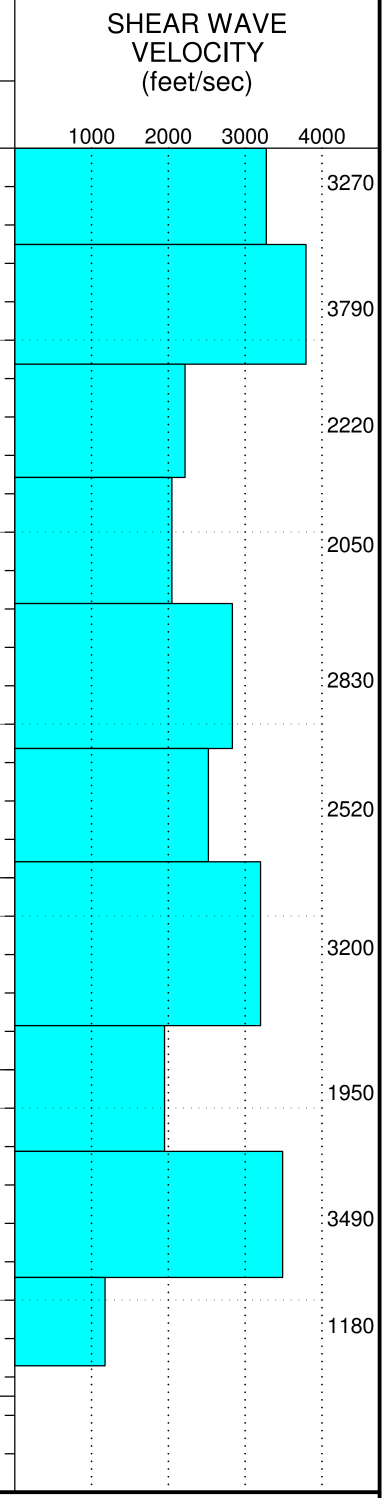
Plate B - 1.2

SHEAR_WAVE_PLOT 8063-00.GPJ GEOLABS.GDT 10/1/21

| GEOLABS, INC. Geotechnical Engineering | SEISMIC RETROFIT OF KAHOLO BRIDGE<br>HAWAII BELT ROAD, PROJECT NO. BR-019-2(072)<br>DISTRICT OF HAMAKUA, ISLAND OF HAWAII | Data Plot of Boring 2 |
|---|---|---|

(Continued from previous plate)

| Core Recovery (%) | Penetration Resistance (blows/foot) | Pocket Pen. (tsf) | Depth (feet) | Sample | Graphic | USCS | Description | Shear Wave Velocity (feet/sec) |
|---|---|---|---|---|---|---|---|---|
| 100 | | | 70–75 | | | | | 3270; 3790 |
| 100 | | | 75–80 | | | | grades to closely to slightly fractured | 2220 |
| 62 | | | 80–85 | | | | grades to severely to closely fractured | 2050; 2830 |
| 52 | | | 85–89 | | | | | 2520 |
| 70 | | | 89–94 | | | GM | Reddish brown **SANDY GRAVEL (BASALTIC)**, dense, wet (clinker) | 3200 |
| 62 | | | 94–102.5 | | | | Gray **BASALT**, severely to moderately fractured, highly to moderately weathered, medium hard to hard (basalt formation) | 1950; 3490; 1180 |
| | | | 102.5 | | | | Boring terminated at 102.5 feet | |




| | | | |
|---|---|---|---|
| Date Started: | May 4, 2021 | Water Level: ⊽ | Not Encountered |
| Date Completed: | May 6, 2021 | | |
| Logged By: | D. Gremminger | Drill Rig: | MOBILE B-53.1 |
| Total Depth: | 102.5 feet | Drilling Method: | 4" Solid-Stem Auger & HQ Coring |
| Work Order: | 8063-00 | Driving Energy: | 140 lb. wt., 30 in. drop |

Plate B - 1.3

**SHEAR WAVE VELOCITY TEST RESULTS**

Seismic Retrofit of Kaholo Bridge
Hawaii Belt Road, Project No. BR-019-2(072)
District of Hamakua, Island of Hawaii

| B-2 | | | | |
|---|---|---|---|---|
| Depth (From) | Depth (To) | Layer Thickness ($d_i$) | Estimated Shear Wave Velocity ($V_{si}$) | Average Travel Time ($d_i/V_{si}$) |
| (feet) | (feet) | (feet) | (feet/second) | (milliseconds) |
| 0.0 | 4.4 | 4.4 | 843 | 5.25 |
| 4.4 | 7.9 | 3.4 | 751 | 4.59 |
| 7.9 | 11.0 | 3.1 | 811 | 3.84 |
| 11.0 | 14.3 | 3.3 | 898 | 3.65 |
| 14.3 | 17.4 | 3.1 | 907 | 3.44 |
| 17.4 | 20.5 | 3.1 | 743 | 4.19 |
| 20.5 | 23.8 | 3.3 | 827 | 3.97 |
| 23.8 | 27.1 | 3.3 | 841 | 3.90 |
| 27.1 | 30.2 | 3.1 | 796 | 3.92 |
| 30.2 | 33.6 | 3.4 | 1,102 | 3.13 |
| 33.6 | 36.7 | 3.1 | 814 | 3.83 |
| 36.7 | 39.9 | 3.1 | 1,004 | 3.10 |
| 39.9 | 43.1 | 3.3 | 1,341 | 2.45 |
| 43.1 | 49.5 | 6.4 | 867 | 7.38 |
| 49.5 | 52.7 | 3.1 | 1,724 | 1.81 |
| 52.7 | 55.9 | 3.3 | 2,793 | 1.17 |
| 55.9 | 59.4 | 3.4 | 1,373 | 2.51 |
| 59.4 | 62.5 | 3.1 | 2,354 | 1.32 |
| 62.5 | 65.8 | 3.3 | 2,084 | 1.57 |
| 65.8 | 69.1 | 3.3 | 2,045 | 1.60 |
| 69.1 | 72.5 | 3.4 | 3,273 | 1.05 |
| 72.5 | 75.6 | 3.1 | 3,793 | 0.82 |
| 75.6 | 78.6 | 3.0 | 2,216 | 1.33 |
| 78.6 | 81.9 | 3.3 | 2,047 | 1.60 |
| 81.9 | 85.6 | 3.8 | 2,832 | 1.33 |
| 85.6 | 88.6 | 3.0 | 2,520 | 1.17 |
| 88.6 | 92.8 | 4.3 | 3,202 | 1.33 |
| 92.8 | 96.1 | 3.3 | 1,950 | 1.68 |
| 96.1 | 99.4 | 3.3 | 3,486 | 0.94 |
| 99.4 | 101.7 | 2.3 | 1,176 | 1.95 |
| | | | | |
| | | | | |
| | | | | |
| TOTAL | | 101.7 | | 75.28 |
| Standard Weighted Average | | | 1,702 | feet/second |
| Computed $V_{s100'}$ Using IBC Formula | | | 1,274 | feet/second |

# APPENDIX C

# **APPENDIX C**

Laboratory Tests

Moisture Content (ASTM D2216) and Unit Weight (ASTM D2937) 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.

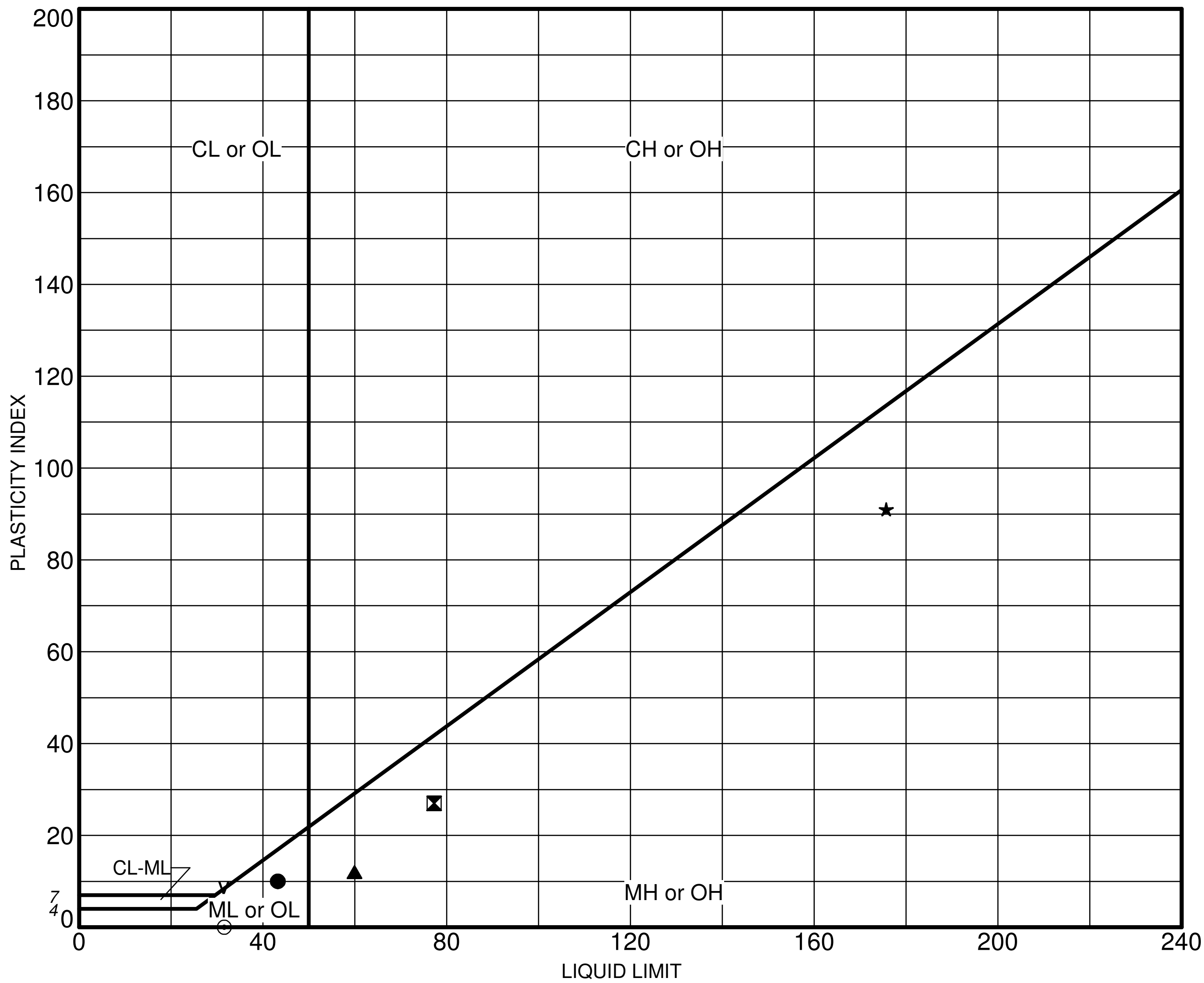
Five Atterberg Limits tests (ASTM D4318) were performed on selected soil samples to evaluate the liquid and plastic limits. The test results are summarized on the Logs of Borings at the appropriate sample depths. A graphic presentation of the test results is provided on Plate C-1.

Five Sieve Analysis tests (ASTM D6913) were performed on selected soil samples to evaluate the gradation characteristics of the soils and to aid in soil classification. Graphic presentations of the test results are provided on Plate C-2.

Eight Unconfined Compression tests (ASTM D7012 Method C) were performed on selected rock core samples to evaluate the uniaxial compressive strength of the rock material encountered. The test results are presented on Plate C-3.

Five Unconsolidated Undrained Triaxial Compression tests (ASTM D2850) were performed on selected soil samples to evaluate the undrained shear strength of the in-situ soils. The approximate in-situ effective overburden pressure was used as the applied confining pressure for each relatively “undisturbed” soil sample. The test results and the stress-strain curves are presented on Plates C-4 through C-8.

Four Direct Shear tests (ASTM D3080) were performed on selected samples to evaluate the shear strength characteristics of the material tested. The test results are presented on Plates C-9 through C-12.




| | Sample | Depth (ft) | LL | PL | PI | Description |
|---|---|---|---|---|---|---|
| ● | B-1 | 5.0-6.5 | 43 | 33 | 10 | Orangish brown with gray mottling clayey silt (ML) with some sand |
| ⧗ | B-3 | 10.0-11.5 | 77 | 50 | 27 | Orangish brown with gray mottling clayey silt (MH) with some sand |
| ▲ | B-3 | 26.0-28.0 | 60 | 48 | 12 | Reddish brown clayey silt (MH) with some sand |
| ★ | B-4 | 5.0-6.5 | 176 | 85 | 91 | Brown with orange mottling clayey silt (MH) with some sand |
| ⊙ | B-4 | 15.0-16.5 | 32 | 43 | NP | Brown with orange mottling clayey silt (ML) with some sand |
| | | | | | | |
| | | | | | | |
| | | | | | | |
| | | | | | | |
| | | | | | | |

NP = NON-PLASTIC

G ATTERBERG PI-200 LL-240 8063-00.GPJ GEOLABS.GDT 10/1/21

**GEOLABS, INC.**

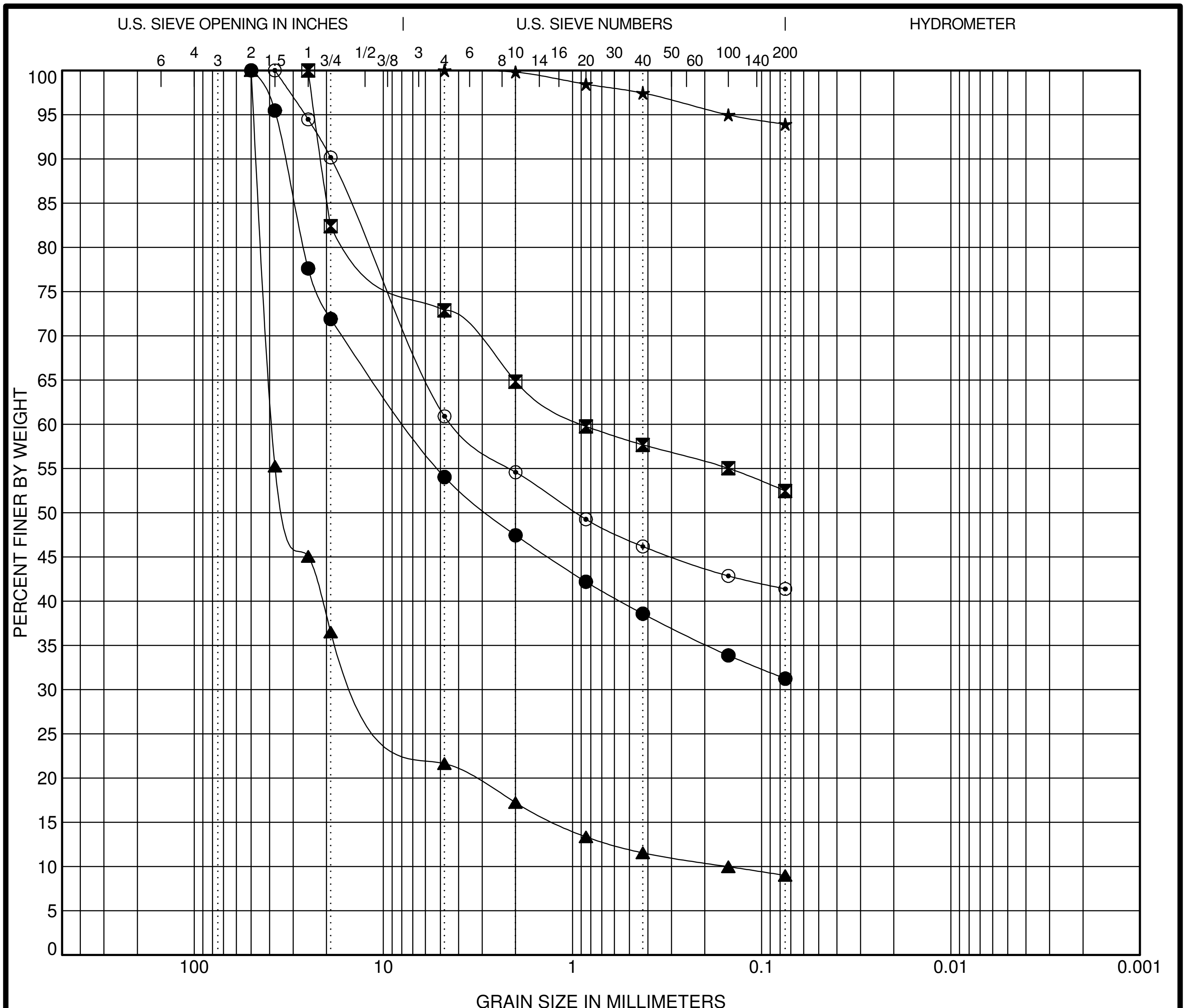
GEOTECHNICAL ENGINEERING

W.O. 8063-00

**ATTERBERG LIMITS TEST RESULTS - ASTM D4318**

SEISMIC RETROFIT OF KAHOLO BRIDGE
HAWAII BELT ROAD, PROJECT NO. BR-019-2(072)
DISTRICT OF HAMAKUA, ISLAND OF HAWAII

Plate
C - 1




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

| | Sample | Depth (ft) | Description | LL | PL | PI | Cc | Cu |
|---|---|---|---|---|---|---|---|---|
| ● | B-1 | 20.0-21.5 | Gray and brown silty gravel (GM) with some sand | | | | | |
| ⧗ | B-2 | 42.5-44.0 | Brownish gray gravelly silt (ML) with some sand | | | | | |
| ▲ | B-3 | 10.0-11.5 | Orangish brown with gray mottling clayey silt (MH) with some sand | 77 | 50 | 27 | 18.2 | 252.9 |
| ★ | B-3 | 20.0-21.5 | Orangish brown with gray mottling clayey silt (MH) with a little sand | | | | | |
| ⊙ | B-4 | 26.0-28.0 | Brown with orange mottling silty gravel (GM) with some sand | | | | | |

| | Sample | Depth (ft) | D100 (mm) | D60 (mm) | D30 (mm) | D10 (mm) | %Gravel | %Sand | %Fine |
|---|---|---|---|---|---|---|---|---|---|
| ● | B-1 | 20.0-21.5 | 50 | 7.55 | | | 46.0 | 22.8 | 31.2 |
| ⧗ | B-2 | 42.5-44.0 | 25 | 0.885 | | | 27.1 | 20.4 | 52.5 |
| ▲ | B-3 | 10.0-11.5 | 50 | 38.662 | 10.368 | 0.153 | 78.4 | 12.6 | 9.0 |
| ★ | B-3 | 20.0-21.5 | 4.75 | | | | 0.0 | 6.1 | 93.9 |
| ⊙ | B-4 | 26.0-28.0 | 37.5 | 4.201 | | | 39.1 | 19.5 | 41.4 |

**GEOLABS, INC.**
GEOTECHNICAL ENGINEERING
W.O. 8063-00

**GRAIN SIZE DISTRIBUTION - ASTM D6913**

SEISMIC RETROFIT OF KAHOLO BRIDGE
HAWAII BELT ROAD, PROJECT NO. BR-019-2(072)
DISTRICT OF HAMAKUA, ISLAND OF HAWAII

Plate
C - 2

G_GRAIN_SIZE_MOD 8063-00.GPJ GEOLABS.GDT 10/1/21

| Location | Depth | Length | Diameter | Length/ Diameter Ratio | Density | Load | Compressive Strength |
|---|---|---|---|---|---|---|---|
| | (feet) | (inches) | (inches) | | (pcf) | (lbs) | (psi) |
| B-1 | 42 - 42.5 | 5.380 | 2.360 | 2.28 | 118.2 | 2,870 | 660 |
| B-1 | 66 - 66.5 | 5.440 | 2.380 | 2.29 | 142.8 | 19,110 | 4,300 |
| B-2 | 75 - 76 | 5.480 | 2.380 | 2.30 | 115.6 | 5,450 | 1,230 |
| B-2 | 95 - 96 | 5.350 | 2.390 | 2.24 | 160.3 | 40,740 | 9,080 |
| B-3 | 50.5 - 51 | 5.520 | 2.390 | 2.31 | 118.8 | 2,700 | 600 |
| B-3 | 61.5 - 62.5 | 5.400 | 2.390 | 2.26 | 137.3 | 13,650 | 3,040 |
| B-4 | 47 - 47.5 | 5.000 | 2.320 | 2.16 | 110.8 | 1,770 | 420 |
| B-4 | 58.5 - 59.5 | 5.300 | 2.380 | 2.23 | 138.5 | 9,930 | 2,230 |

ASTM D7012 (METHOD C)

Note: Samples were not prepared in accordance with ASTM D4543. Therefore, results reported may differ from results obtained from a test speciment that meets the requirements of Practice D4543



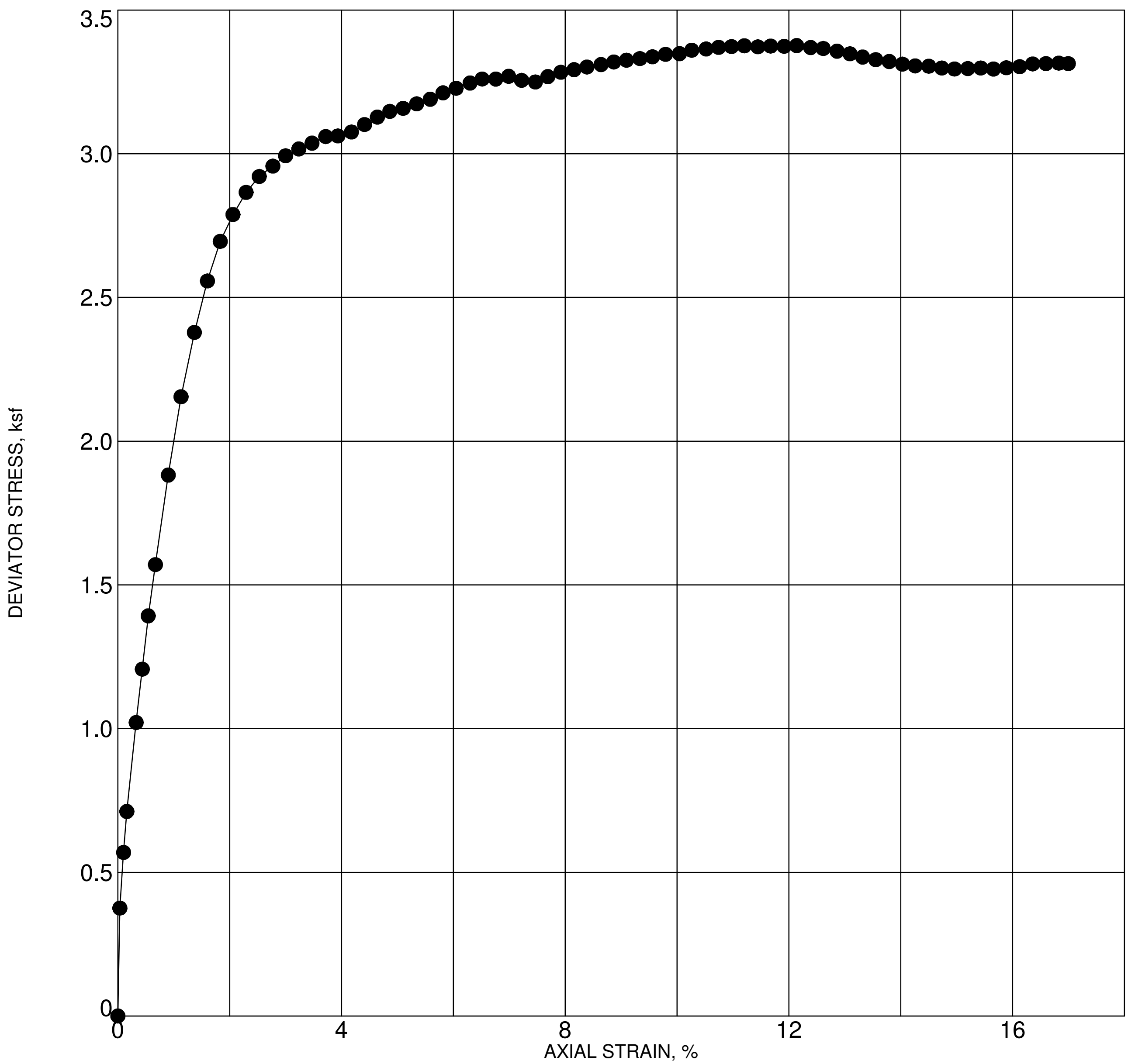
**GEOLABS, INC.**

GEOTECHNICAL ENGINEERING

W.O. 8063-00

**UNIAXIAL COMPRESSIVE STRENGTH TEST**

SEISMIC RETROFIT OF KAHOLO BRIDGE
HAWAII BELT ROAD, PROJECT NO. BR-019-2(072)
DISTRICT OF HAMAKUA, ISLAND OF HAWAII




| | |
|---|---|
| Max. Deviator Stress (ksf): | 3.4 |
| Confining Stress (ksf): | 1.0 |

Location: B-1
Depth: 10.0 - 11.5 feet
Description: Orangish brown with gray mottling clayey silt with some sand
Test Date: 6/4/2021

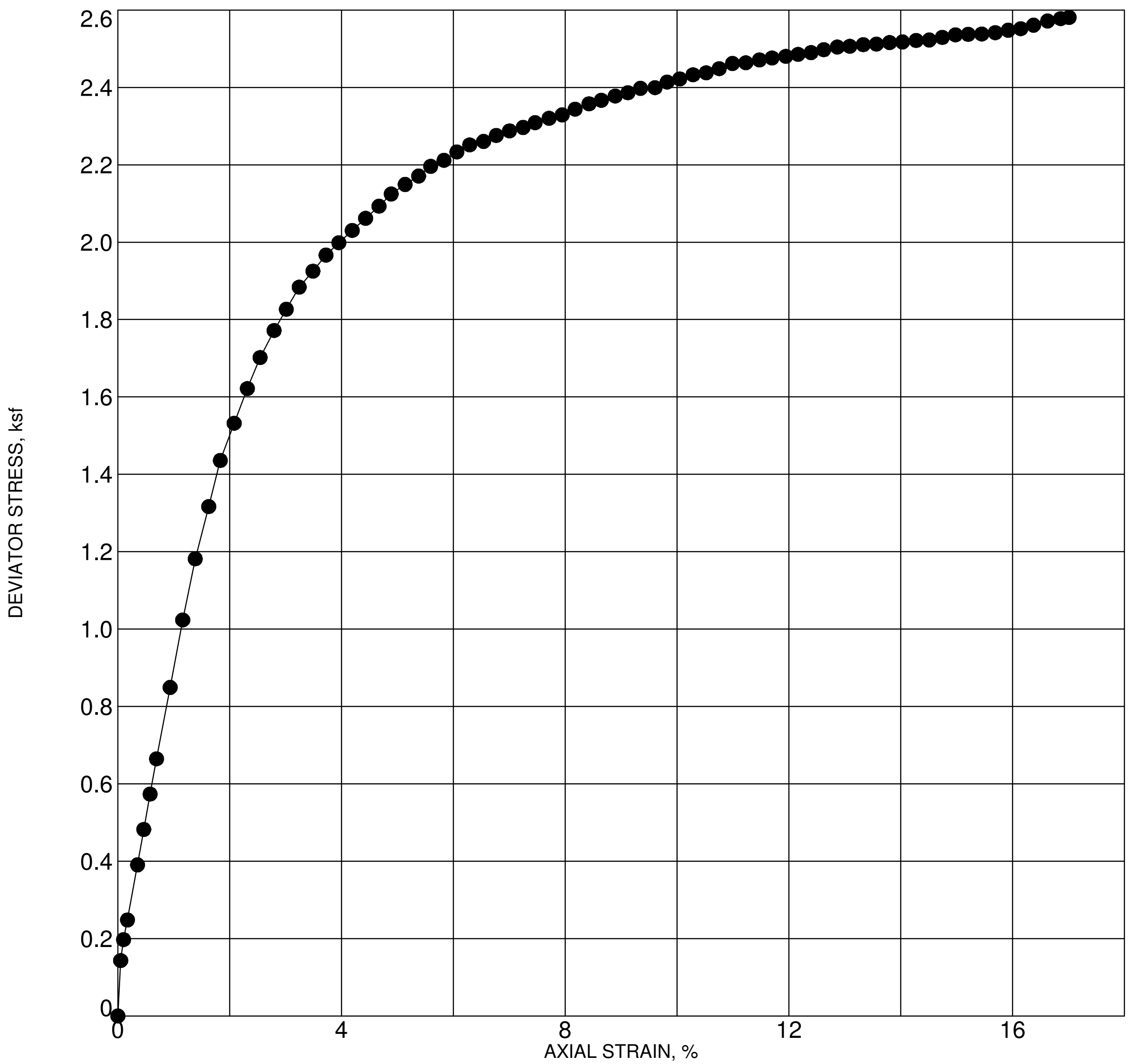
| | | | |
|---|---|---|---|
| Dry Density (pcf) | 64.7 | Sample Diameter (inches) | 2.413 |
| Moisture (%) | 62.3 | Sample Height (inches) | 5.100 |
| Axial Strain at Failure (%) | 12.1 | Strain Rate (% / minute) | 0.70 |

**GEOLABS, INC.**
GEOTECHNICAL ENGINEERING
W.O. 8063-00

**TRIAXIAL UU COMPRESSION TEST - ASTM D2850**

SEISMIC RETROFIT OF KAHOLO BRIDGE
HAWAII BELT ROAD, PROJECT NO. BR-019-2(072)
DISTRICT OF HAMAKUA, ISLAND OF HAWAII

Plate
C - 4




| | |
|---|---|
| Max. Deviator Stress (ksf): | 2.5 |
| Confining Stress (ksf): | 0.5 |

Location: B-2
Depth: 5.0 - 6.5 feet
Description: Brown clayey silt with some sand
Test Date: 6/4/2021

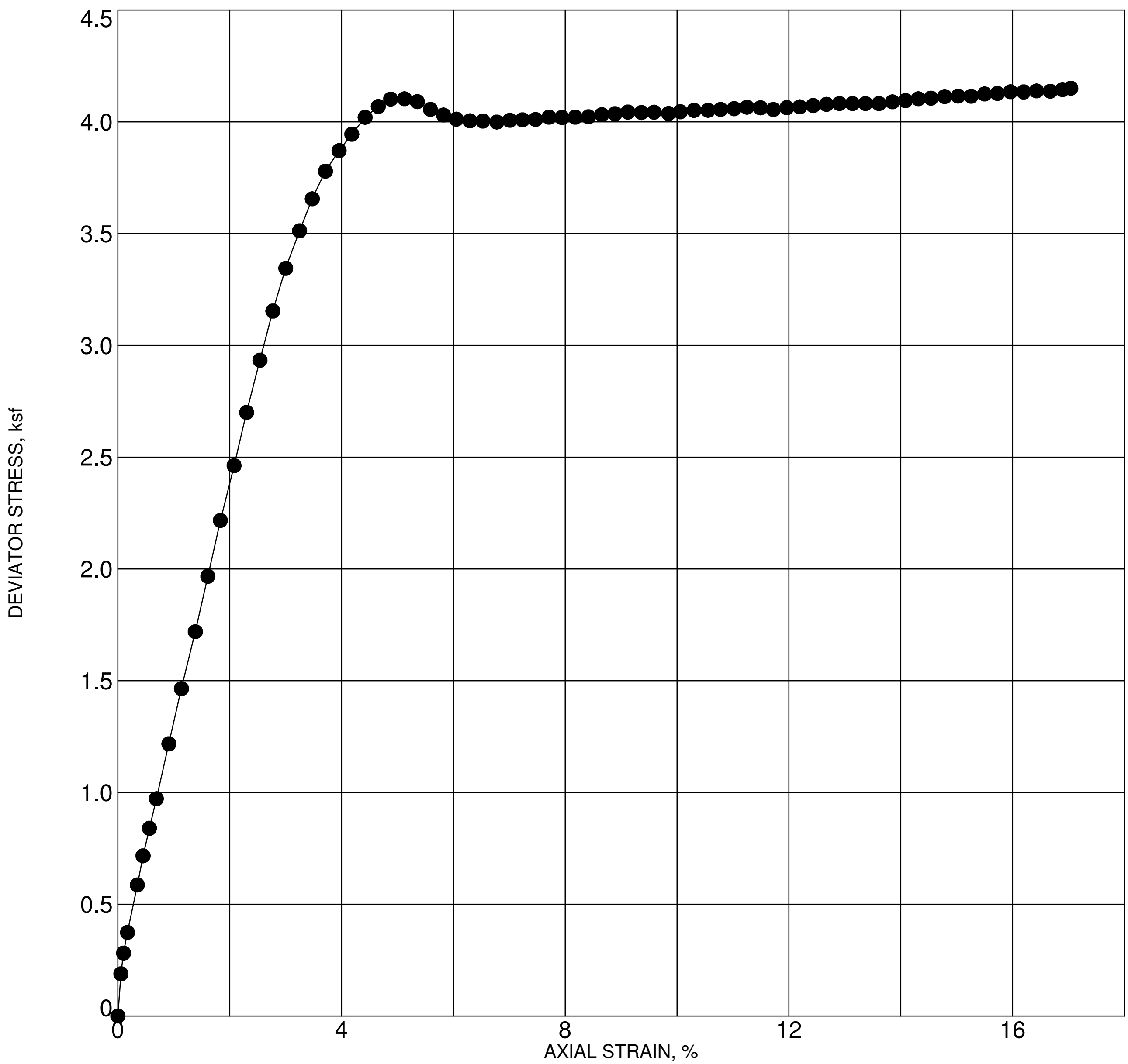
| | | | |
|---|---|---|---|
| Dry Density (pcf) | 71.0 | Sample Diameter (inches) | 2.413 |
| Moisture (%) | 44.8 | Sample Height (inches) | 5.100 |
| Axial Strain at Failure (%) | 15.0 | Strain Rate (% / minute) | 0.70 |

GEOLABS, INC.
GEOTECHNICAL ENGINEERING
W.O. 8063-00

**TRIAXIAL UU COMPRESSION TEST - ASTM D2850**

SEISMIC RETROFIT OF KAHOLO BRIDGE
HAWAII BELT ROAD, PROJECT NO. BR-019-2(072)
DISTRICT OF HAMAKUA, ISLAND OF HAWAII

Plate
C - 5




| | |
|---|---|
| Max. Deviator Stress (ksf): | 4.1 |
| Confining Stress (ksf): | 1.0 |

Location: B-3
Depth: 10.0 - 11.5 feet
Description: Orangish brown with gray mottling clayey silt (MH) with some sand
Test Date: 6/4/2021

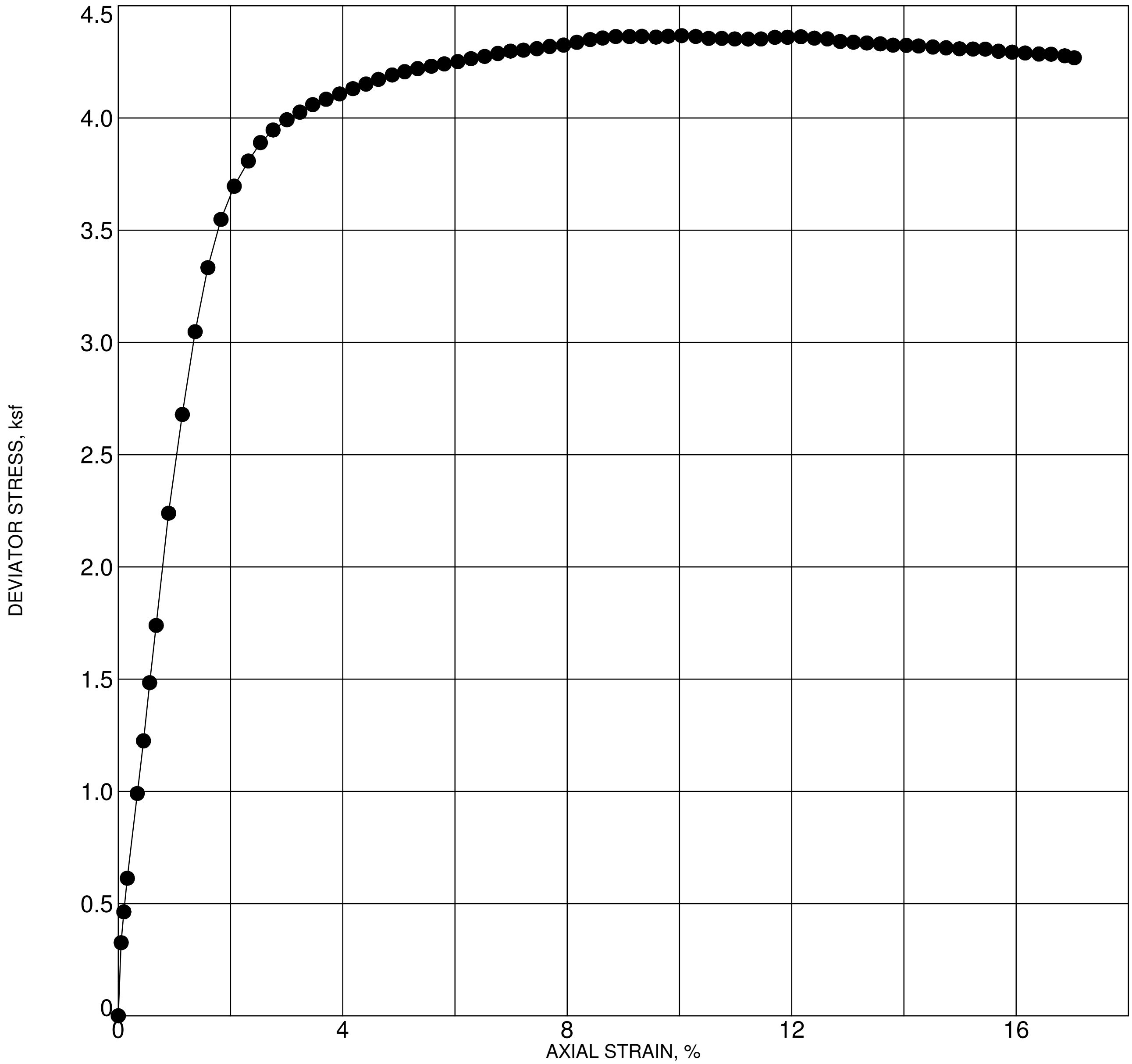
| | | | |
|---|---|---|---|
| Dry Density (pcf) | 65.1 | Sample Diameter (inches) | 2.413 |
| Moisture (%) | 49.7 | Sample Height (inches) | 5.100 |
| Axial Strain at Failure (%) | 14.8 | Strain Rate (% / minute) | 0.71 |

**GEOLABS, INC.**
GEOTECHNICAL ENGINEERING
W.O. 8063-00

**TRIAXIAL UU COMPRESSION TEST - ASTM D2850**

SEISMIC RETROFIT OF KAHOLO BRIDGE
HAWAII BELT ROAD, PROJECT NO. BR-019-2(072)
DISTRICT OF HAMAKUA, ISLAND OF HAWAII

Plate
C - 6




| | |
|---|---|
| Max. Deviator Stress (ksf): | 4.4 |
| Confining Stress (ksf): | 1.5 |

Location: B-3
Depth: 15.0 - 16.5 feet
Description: Orangish brown with gray mottling clayey silt with some sand
Test Date: 6/4/2021

| | | | |
|---|---|---|---|
| Dry Density (pcf) | 56.7 | Sample Diameter (inches) | 2.413 |
| Moisture (%) | 67.9 | Sample Height (inches) | 5.100 |
| Axial Strain at Failure (%) | 10.0 | Strain Rate (% / minute) | 0.70 |

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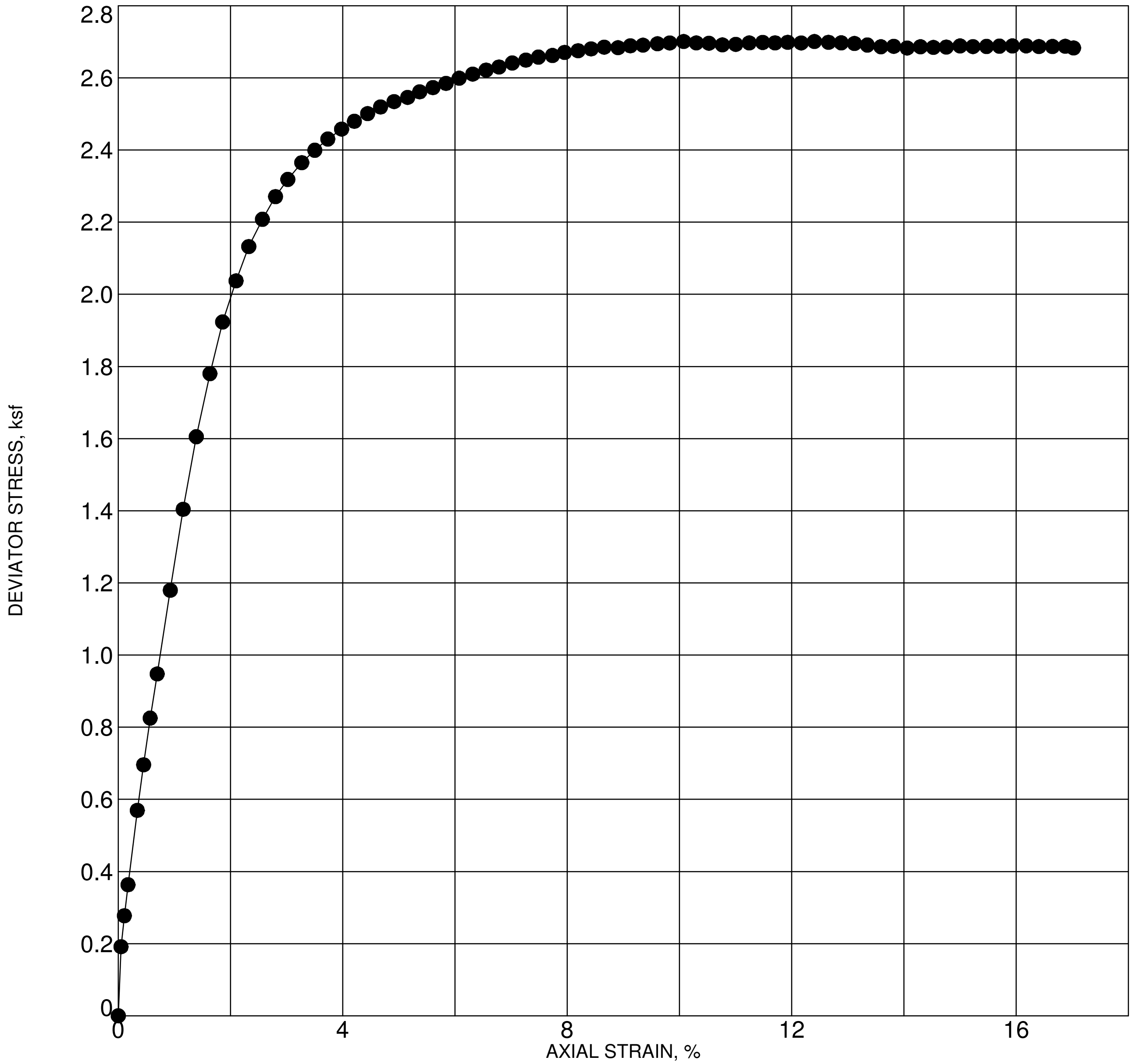
HAWAII BELT ROAD, PROJECT NO. BR-019-2(072)
DISTRICT OF HAMAKUA, ISLAND OF HAWAII

Plate
C - 7




| | |
|---|---|
| Max. Deviator Stress (ksf): | 2.7 |
| Confining Stress (ksf): | 1.0 |

Location: B-4
Depth: 10.0 - 11.5 feet
Description: Brown with orange mottling clayey silt with some sand
Test Date: 6/4/2021

| | | | |
|---|---|---|---|
| Dry Density (pcf) | 60.0 | Sample Diameter (inches) | 2.413 |
| Moisture (%) | 58.9 | Sample Height (inches) | 5.100 |
| Axial Strain at Failure (%) | 13.1 | Strain Rate (% / minute) | 0.70 |

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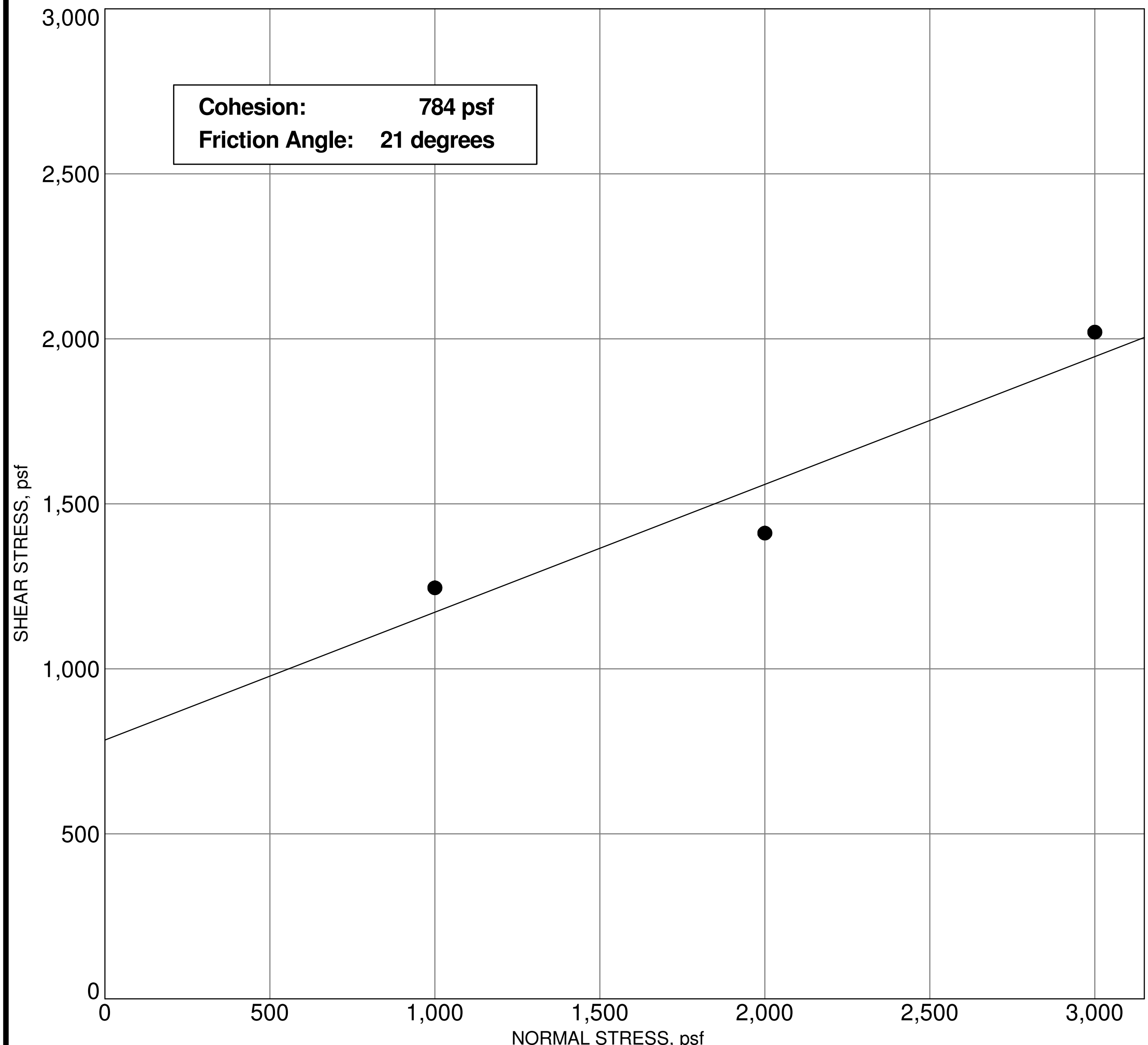
SEISMIC RETROFIT OF KAHOLO BRIDGE
HAWAII BELT ROAD, PROJECT NO. BR-019-2(072)
DISTRICT OF HAMAKUA, ISLAND OF HAWAII

Plate
C - 8

G_TXUU 8063-00.GPJ GEOLABS.GDT 10/1/21




| | | Sample #1 | Sample #2 | Sample #3 |
|---|---|---|---|---|
| INITIAL | Moisture Content, % | 88.1 | 82.2 | 87.4 |
| | Dry Density, pcf | 52.3 | 56.4 | 53.8 |
| | Height, inches | 1.00 | 1.00 | 1.00 |
| FINAL | Moisture Content, % | 70.9 | 51.4 | 53.7 |
| | Dry Density, pcf | 51.1 | 58.3 | 56.0 |
| | Height, inches | 1.024 | 0.967 | 0.962 |
| | Diameter, inches | 2.42 | 2.42 | 2.42 |
| | Deformation Rate, inch/minute | 0.0024 | 0.0021 | 0.0023 |
| | Normal Stress, psf | **1000** | **2000** | **3000** |
| | Peak Shear Stress, psf | **1245** | **1411** | **2020** |
| | Shear Displacement, inches | 0.43 | 0.41 | 0.42 |

Sample: B-1
Depth: 15.0 - 16.5 feet
Description: Orangish brown with gray mottling clayey silt with some sand

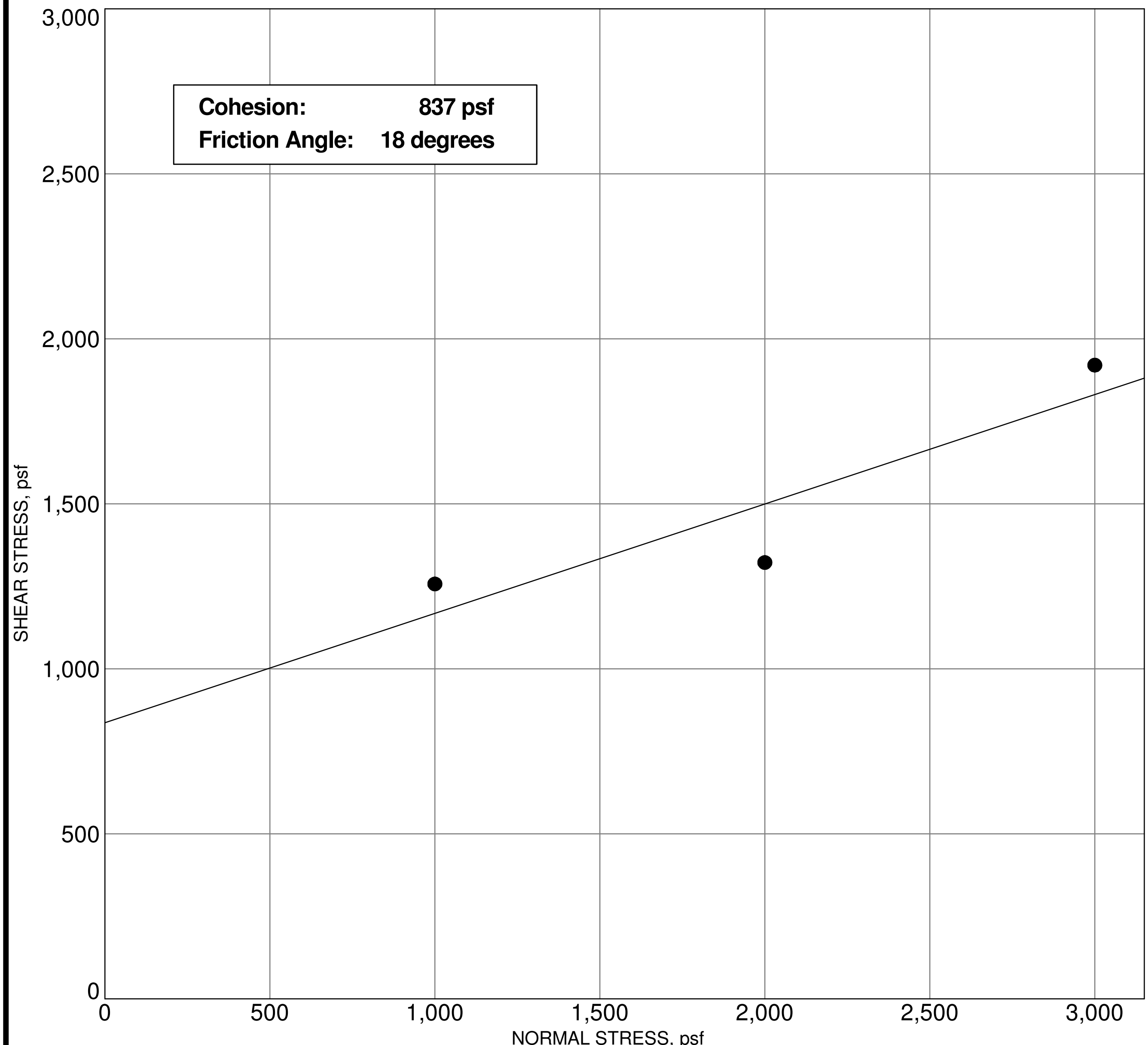
G_DIRECT_SHEAR 8063-00.GPJ GEOLABS.GDT 10/1/21

**GEOLABS, INC.**
GEOTECHNICAL ENGINEERING
W.O. 8063-00

## DIRECT SHEAR TEST - ASTM D3080

SEISMIC RETROFIT OF KAHOLO BRIDGE
HAWAII BELT ROAD, PROJECT NO. BR-019-2(072)
DISTRICT OF HAMAKUA, ISLAND OF HAWAII

Plate
C - 9




| | | Sample #1 | Sample #2 | Sample #3 |
|---|---|---|---|---|
| INITIAL | Moisture Content, % | 80.0 | 74.0 | 74.2 |
| | Dry Density, pcf | 57.2 | 57.6 | 60.0 |
| | Height, inches | 1.00 | 1.00 | 1.00 |
| FINAL | Moisture Content, % | 68.0 | 64.1 | 62.5 |
| | Dry Density, pcf | 56.5 | 58.5 | 60.6 |
| | Height, inches | 1.013 | 0.986 | 0.990 |
| | Diameter, inches | 2.42 | 2.42 | 2.42 |
| | Deformation Rate, inch/minute | 0.0024 | 0.0007 | 0.0005 |
| | Normal Stress, psf | **1000** | **2000** | **3000** |
| | Peak Shear Stress, psf | **1257** | **1322** | **1920** |
| | Shear Displacement, inches | 0.42 | 0.36 | 0.35 |

Sample: B-2
Depth: 15.0 - 16.5 feet
Description: Brown clayey silt with some sand

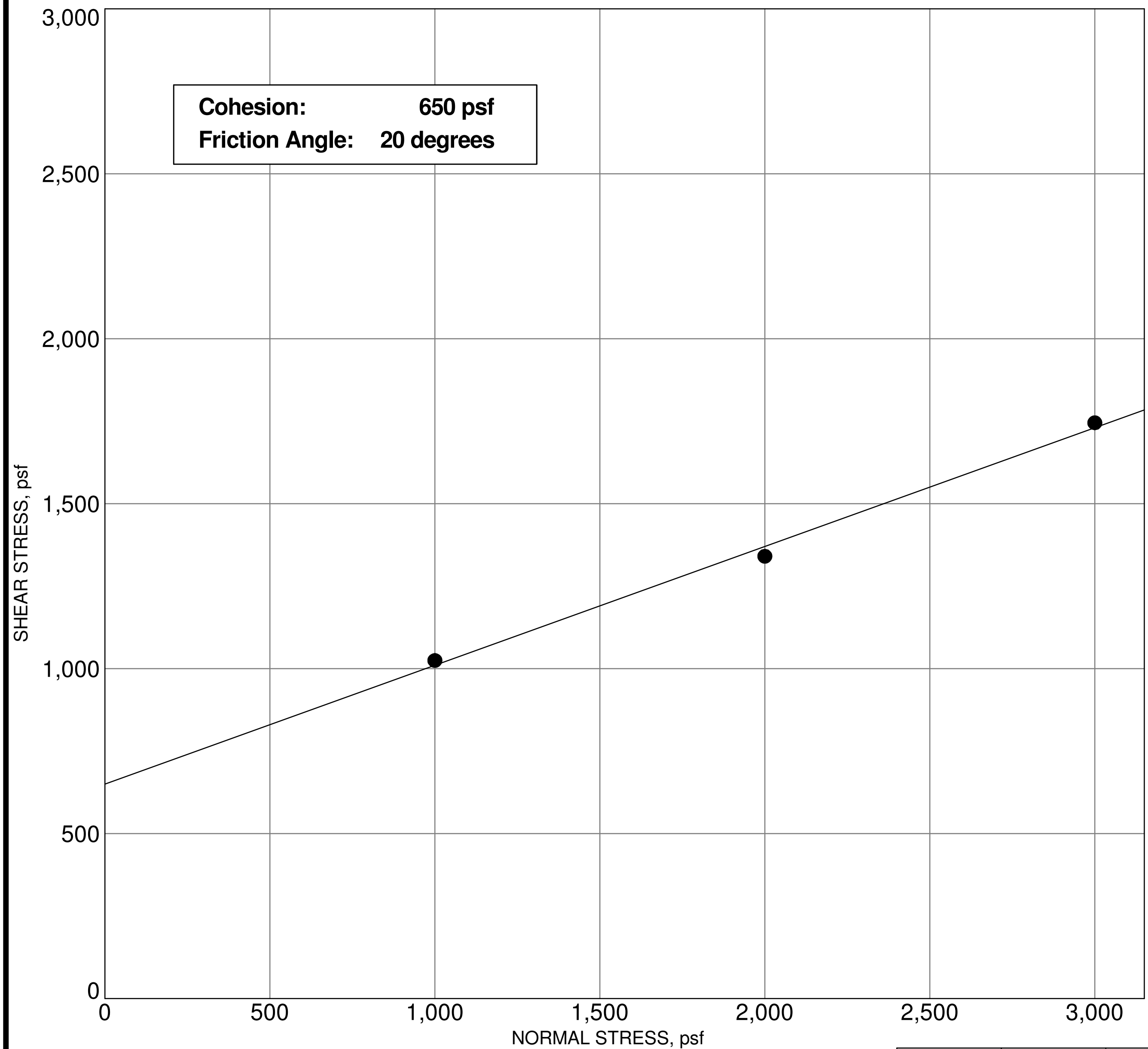
**GEOLABS, INC.**
GEOTECHNICAL ENGINEERING
W.O. 8063-00

**DIRECT SHEAR TEST - ASTM D3080**

SEISMIC RETROFIT OF KAHOLO BRIDGE
HAWAII BELT ROAD, PROJECT NO. BR-019-2(072)
DISTRICT OF HAMAKUA, ISLAND OF HAWAII

Plate
C - 10

G_DIRECT_SHEAR 8063-00.GPJ GEOLABS.GDT 10/1/21




| | | Sample #1 | Sample #2 | Sample #3 |
|---|---|---|---|---|
| INITIAL | Moisture Content, % | 94.2 | 65.4 | 71.2 |
| | Dry Density, pcf | 45.6 | 54.4 | 54.1 |
| | Height, inches | 1.00 | 1.00 | 1.00 |
| FINAL | Moisture Content, % | 73.9 | 69.8 | 67.2 |
| | Dry Density, pcf | 45.8 | 54.9 | 55.5 |
| | Height, inches | 0.996 | 0.992 | 0.975 |
| | Diameter, inches | 2.42 | 2.42 | 2.42 |
| | Deformation Rate, inch/minute | 0.0025 | 0.0022 | 0.0015 |
| | Normal Stress, psf | **1000** | **2000** | **3000** |
| | Peak Shear Stress, psf | **1025** | **1341** | **1745** |
| | Shear Displacement, inches | 0.43 | 0.42 | 0.37 |

Sample: B-3
Depth: 20.0 - 21.5 feet
Description: Orangish brown with gray mottling clayey silt (MH) with a little sand

G_DIRECT_SHEAR 8063-00.GPJ GEOLABS.GDT 10/1/21

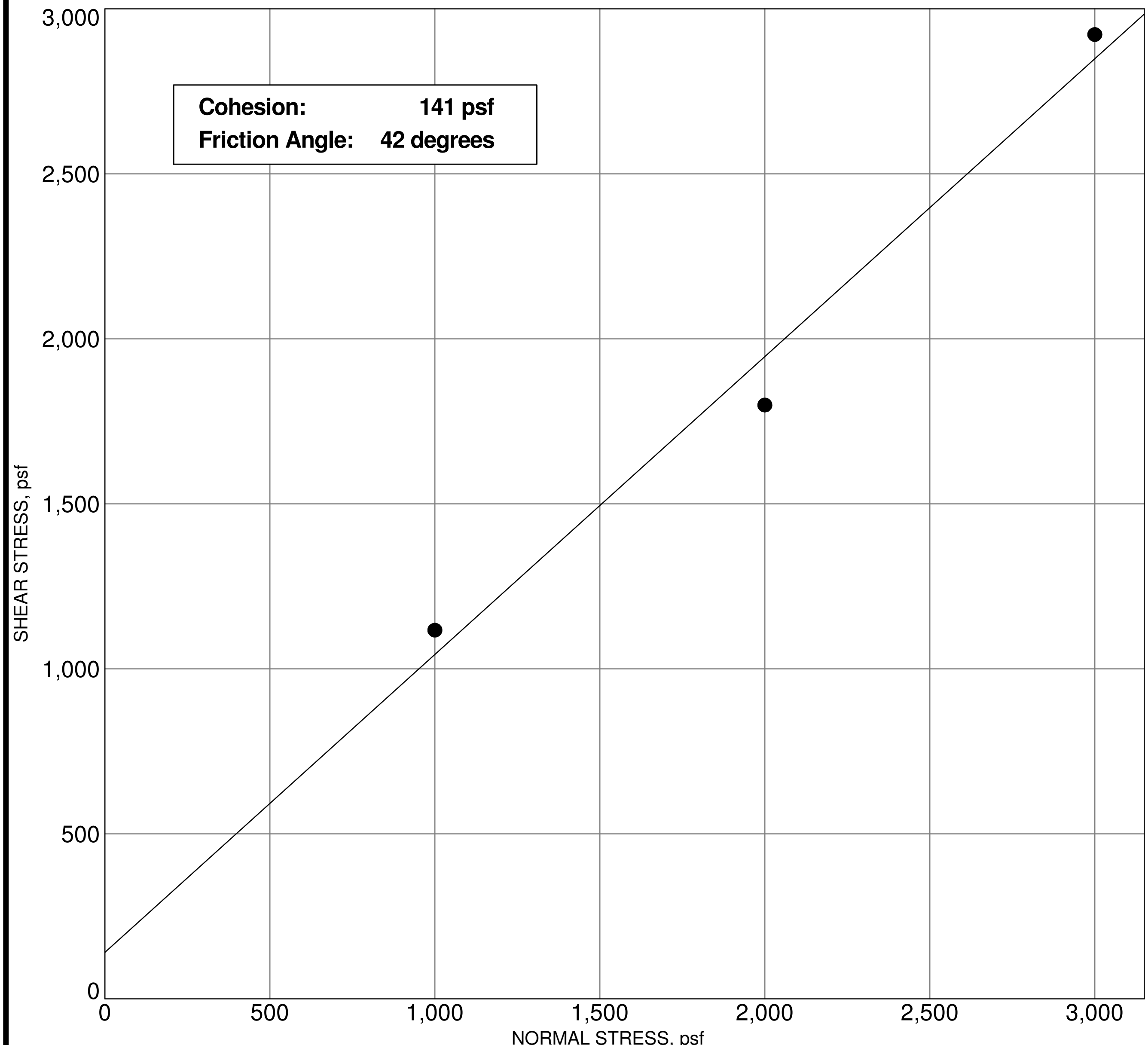
**GEOLABS, INC.**
GEOTECHNICAL ENGINEERING

W.O. 8063-00

**DIRECT SHEAR TEST - ASTM D3080**

SEISMIC RETROFIT OF KAHOLO BRIDGE
HAWAII BELT ROAD, PROJECT NO. BR-019-2(072)
DISTRICT OF HAMAKUA, ISLAND OF HAWAII

Plate
C - 11




| | | Sample #1 | Sample #2 | Sample #3 |
|---|---|---|---|---|
| INITIAL | Moisture Content, % | 74.4 | 61.7 | 68.1 |
| | Dry Density, pcf | 56.4 | 63.4 | 61.5 |
| | Height, inches | 1.00 | 1.00 | 1.00 |
| FINAL | Moisture Content, % | 66.8 | 55.6 | 56.3 |
| | Dry Density, pcf | 54.4 | 64.0 | 63.1 |
| | Height, inches | 1.038 | 0.991 | 0.974 |
| | Diameter, inches | 2.42 | 2.42 | 2.42 |
| | Deformation Rate, inch/minute | 0.0024 | 0.0023 | 0.0023 |
| | Normal Stress, psf | **1000** | **2000** | **3000** |
| | Peak Shear Stress, psf | **1117** | **1799** | **2922** |
| | Shear Displacement, inches | 0.43 | 0.41 | 0.41 |

Sample: B-4
Depth: 20.0 - 20.9 feet
Description: Brown with orange mottling clayey silt with some sand

G_DIRECT_SHEAR 8063-00.GPJ GEOLABS.GDT 10/1/21

**GEOLABS, INC.**
GEOTECHNICAL ENGINEERING
W.O. 8063-00

## DIRECT SHEAR TEST - ASTM D3080

SEISMIC RETROFIT OF KAHOLO BRIDGE
HAWAII BELT ROAD, PROJECT NO. BR-019-2(072)
DISTRICT OF HAMAKUA, ISLAND OF HAWAII

Plate
C - 12

# APPENDIX D

**SEISMIC RETROFIT OF KAHOLO BRIDGE**
**HAWAII BELT ROAD, PROJECT NO. BR-019-2(072)**
**DISTRICT OF HAMAKUA, ISLAND OF HAWAII**

---

**B-1 21.5’ TO 64.5’**

**21.5’** **49.0’**


**49.0’** **64.5’**

**B-1 64.5' TO 80.5'**

**64.5'** **73.0'**


**73.0'** **80.5'**

**B-2 29.0' TO 77.5'**

**29.0'** **67.5'**


**67.5'** **77.5'**

**B-2 77.5' TO 102.5'**

77.5' 87.5'


87.5' 102.5'

SEISMIC RETROFIT OF KAHOLO BRIDGE
HAWAII BELT ROAD, PROJECT NO. BR-019-2(072)
DISTRICT OF HAMAKUA, ISLAND OF HAWAII

**B-3 21.5' TO 63.0'**

**21.5'** **53.0'**


**53.0'** **63.0'**

**B-3 63.0' TO 91.0'**

63.0' 76.0' 88.0'


76.0' 88.0' 91.0'

# SEISMIC RETROFIT OF KAHOLO BRIDGE
# HAWAII BELT ROAD, PROJECT NO. BR-019-2(072)
# DISTRICT OF HAMAKUA, ISLAND OF HAWAII

**B-4 20.9' TO 58.5'**

**20.9'** **48.0'**


**48.0'** **58.5'**

**B-4 58.5' TO 76.0'**

58.5' 71.0'


71.0' 76.0'