

FINAL SUBMISSION
GEOTECHNICAL ENGINEERING EXPLORATION
WAIALE, POHAKUOKALA, AND ALAE BRIDGES
SEISMIC RETROFIT
KULA, MAUI, HAWAII
W.O. 8412-00(B) MARCH 25, 2026

Prepared for

WSP



GEOLABS, INC.
Geotechnical Engineering and Drilling Services

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THIS WORK WAS PREPARED BY
ME OR UNDER MY SUPERVISION.

A handwritten signature in blue ink, appearing to read "Gerald Y. Seki".

SIGNATURE

4-30-26
EXPIRATION DATE
OF THE LICENSE



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Hawaii • California



GEOLABS, INC.

Geotechnical Engineering and Drilling Services

March 25, 2026
W.O. 8412-00(B)

Mr. Randall Urasaki

WSP

1001 Bishop Street, Suite 2400
American Savings Bank Tower
Honolulu, HI 96813

Dear **Mr. Urasaki:**

Geolabs, Inc. is pleased to submit our report entitled "Geotechnical Engineering Exploration, Waiale, Pohakuokala, and Alae Bridges Seismic Retrofit, Kula, Maui, Hawaii."

Our work for the project was performed in general accordance with the scope of services outlined in our revised fee proposal dated January 26, 2023.

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 special arrangements are made for a longer sample storage period. Please contact our office for alternative sample storage requirements, if appropriate.

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

Very truly yours,

GEOLABS, INC.

Gerald Y. Seki, P.E.
Vice President

GS:JS:lf

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SUMMARY OF FINDINGS AND RECOMMENDATIONS
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The subsurface conditions at the three bridge project sites were explored by drilling and sampling six borings (1 per bridge abutment), extending to depths from approximately 45 to 51.5 feet below the existing ground surface. The field exploration generally encountered a pavement structure consisting of 4 to 5 inches of asphaltic concrete at Waiale and Alae Bridges and 5.5 to 9 inches of asphaltic concrete at Pohakuokala Bridge, overlying silty gravel base material. This pavement structure was underlain by fill material and residual soils, extending to varying depths below the existing pavement surface. The fill and residual soils were generally underlain by weathered basalt rock formation and basalt rock formation with interbedded layers of clinker materials, extending to the maximum depth explored.

The fill materials generally consisted of loose silty sand and stiff sandy silt, medium dense silty gravel, and stiff to very stiff sandy silts. The residual soils generally consisted of medium dense silty sand and silty gravel, and very dense gravelly cobbles. The basalt rock formations encountered were generally hard to very hard and ranged from slightly to moderately weathered.

We did not encounter groundwater in the borings at the time of our field exploration. However, it should be noted that groundwater levels are subject to change due to rainfall, time of year, seasonal precipitation, surface water runoff, and other factors.

Based on the information provided, the current retrofit design concept consists of utilizing a tie beam connected from the existing abutments to an anchor beam and soil nails to resist seismically induced loading. To reduce earth pressures on the abutment walls, lightweight cellular concrete backfill will be used as backfill within the tie beam and anchor beam excavations.

The soil-nailed anchor beam system for the bridge projects involves individual reinforcing bars grouted into drilled holes, connecting to an anchor beam and tie beams to resist seismically induced loading. Four 8-inch diameter soil nails with allowable tensile load capacities of 40 kips per anchor will be utilized for each bridge abutment. The installation of soil nails involves drilling a minimum 8-inch diameter hole at an inclination of approximately 20 degrees from horizontal, using ASTM A615 Grade 75 solid threaded bar with a minimum bar diameter of 1.5 inches and a double corrosion protection system for the nails.

SUMMARY OF FINDINGS AND RECOMMENDATIONS

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

END OF SUMMARY OF FINDINGS AND RECOMMENDATIONS

SECTION 1. GENERAL

1.1 Introduction

This report presents the results of our geotechnical engineering exploration performed for the *Waiale, Pohakuokala, and Alae Bridges* project in Kula, on the Island of Maui, Hawaii. The project location and general vicinity of each bridge are shown on the Project Location Map, Plate 1.

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

1.2 Project Considerations

The proposed project sites are located along Haleakala Highway and Kekaulike Avenue in Kula on the Island of Maui, Hawaii. We understand that it is desired to perform a seismic retrofit for the following bridges:

1. Waiale Bridge, Bridge No. 009003770500540: Three-span reinforced concrete girder bridge about 150 feet in length and 27 feet in width.
2. Pohakuokala Bridge, Bridge No. 009003780500349: Three-span reinforced concrete girder bridge about 110 feet in length and 25 feet in width.
3. Alae Bridge, Bridge No. 009003770500058: Three-span reinforced concrete girder bridge about 150 feet in length and 25 feet in width.

Based on the information provided, the current retrofit design concept consists of utilizing a tie beam connected from the existing abutments to an anchor beam and soil nails to resist seismically induced loading. To reduce earth pressures on the abutment walls, lightweight cellular concrete backfill will be used as backfill within the tie beam and anchor beam excavations.

1.3 Scope of Work

The purpose of our geotechnical exploration was to obtain an overview of the surface and subsurface conditions to develop a soil and/or rock data set to formulate geotechnical

recommendations for the seismic retrofit design of the proposed project. Our work was performed in general accordance with our revised fee proposal dated January 26, 2023. Our scope of work generally consisted of the following tasks and work efforts:

1. Research and review of available subsurface information in the project vicinity and existing as-built drawings.
2. Perform a site reconnaissance to observe the existing bridge conditions.
3. Application for the Hawaii Department of Transportation (HDOT) roadway permit.
4. Application and coordination of underground utility toning.
5. Mobilization and demobilization of a truck-mounted drill rig and operators to the project site and back.
6. Drilling and sampling of six borings at selected locations (two per bridge location) extending to depths of 45 to 51.5 feet below the existing ground surface for a total of approximately 298 lineal feet of exploration.
7. Coordination of the field exploration and logging of the boreholes by our field geologist.
8. Geotechnical laboratory testing of selected soil samples obtained during the field exploration as an aid in classifying the materials and evaluating their engineering properties.
9. Analyses of the field and laboratory data to formulate geotechnical engineering recommendations for the seismic retrofit design of the proposed project.
10. Preparation of this formal report summarizing our work and presenting our findings and recommendations.
11. Coordination of our overall work on the project by our project engineer.
12. Quality assurance of our work and client/design team consultation by our principal engineer.
13. 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 laboratory tests performed on selected soil and rock samples retrieved from our field exploration are presented in Appendix B. Photographs of core samples retrieved from the borings are presented in Appendix C.

END OF GENERAL

SECTION 2. SITE CHARACTERIZATION

Of interest to our geotechnical engineering analysis are the subsurface materials encountered at the project site, the engineering properties of the materials encountered, and the variability of the subsurface conditions across the project site. Therefore, the following sections provide a description of the geologic setting of the project site, the surface and subsurface conditions encountered at the site, and a discussion of the items needed for seismic design, such as seismicity, soil liquefaction, and the soil profile characteristics for seismic analysis.

2.1 Regional Geology

The Island of Maui was built by two major volcanoes: the older West Maui (Tertiary Epoch) and the more recent East Maui, also known as Haleakala (Pleistocene Epoch). The Isthmus of Maui is a narrow, gently sloping plain between these two volcanoes.

The Isthmus of Maui was created by lava flows from Haleakala ponding on West Maui. It is comprised of alluvium washed from the slopes of West Maui and East Maui (Haleakala). The erosional processes are dominated by the detachment of soil and rock masses from the mountain walls, and the soil materials are transported downslope toward the isthmus primarily by gravity as colluvium. Once these materials reach the stream in the central portion of a valley, alluvial processes become dominant, and the sediments are transported and deposited as alluvium.

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

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

headwaters. This continued erosion generates material, which is transported downslope covering the older alluvial soil deposits. Depending on the local base level and rate of transport, these newer sediments are generally transient in terms of geologic time. In addition, their consistency and density are generally less than those of the older, partially consolidated deposits.

The project site generally is located on the western flank of Haleakala Volcano. The bulk of the Haleakala Shield Volcano was built during the late Pliocene and early Pleistocene Epoch by thinly bedded basaltic lava flows of the Honomanu Volcanic Series. During the Pleistocene Epoch, the characteristics of the lava changed to very hard, thickly bedded flows of andesitic composition. These lavas have been grouped as the Kula Volcanic Series. Typically, the basalt rock formation consists of thinly to thickly bedded a'a and pahoehoe type lava flows. Near the end of the eruptions of the Kula Volcanic Series, the Haleakala Volcano rose to a summit possibly 3,500 feet higher than the present crater rim. At the end of the accumulation of the Kula Volcanic Series, volcanic activity became very infrequent, and stream erosion began to make progress in wearing away the mountain and carving master valleys. Finally, after this period of erosion, volcanic activity resumed. The principal rift zones reopened right across the depression at the top of the mountain and down the flanks, and the great erosional valleys were flooded with lava flows and filled with cinder cones.

2.2 Existing Site Conditions

The project sites are located along Haleakala Highway and Kekaulike Avenue in Kula on the Island of Maui, Hawaii. The project locations are presented on the Site Plan, Plates 2.1 through 2.3. A site visit was conducted on December 29, 2022 to obtain an overview of the existing bridges and geologic features to assist in interpreting the anticipated subsurface conditions. A brief description of each bridge is provided in the subsections below.

Waiale Bridge (Bridge No. 009003770500540)

The Waiale Bridge is located on Haleakala Highway at Mile Post 3.74 and spans over the Waiale Gulch. The bridge was constructed in 1929 and is about 150 feet in length and 27 feet in width. The finish grade of the top of the deck is about +2,760 to +2,764 feet

Mean Sea Level (MSL). The bridge is a three-span bridge supported by two end abutments and two piers. The two abutments and two piers are supported by spread footings. Based on the available as-built plans, the bottom elevation of the abutment spread footings is approximately +2,750 feet MSL at the north abutment, +2,757 feet MSL at the south abutment, +2,728 feet MSL at the north intermediate pier, and +2,726 feet MSL at the south intermediate pier. Photographs of the existing bridge structure obtained during our site visit are provided below for reference.



Waiale Bridge North Abutment – Surface Fills and Weathered Basalt Observed



Waiale Bridge South Abutment – Surface Fills and Weathered Basalt Observed



Waiale Bridge North Pier – Basalt Rock Formation Outcrop Observed



Waiale Bridge South Pier – Basalt Rock Formation Outcrop Observed

Pohakuokala Bridge (Bridge No. 009003780500349)

The Pohakuokala Gulch Bridge is located on Haleakala Highway at Mile Post 5.66 and spans over the Pohakuokala Gulch. The bridge was constructed in 1929 and is about 110 feet in length and 25 feet in width. The finish grade of the top of the deck is about +3,275 to +2,764 feet MSL. The bridge is a three-span bridge supported by two end abutments and two piers. The two abutments and two piers are supported by spread footings. Based on the available as-built plans, the bottom elevations of the abutment spread footings are approximately +3,260 feet MSL at the north abutment, +3,275 feet MSL at the south abutment, and +3,244 feet MSL at the intermediate piers. Photographs of the existing bridge structure obtained during our site visit are provided below for reference.



Pohakuokala Gulch Bridge North Abutment – Surface Fills and Weathered Basalt



Pohakuokala Gulch Bridge South Abutment – Clinker and Weathered Basalt Observed



Pohakuokala Gulch Bridge North Pier – Basalt Rock Formation Outcrop Observed



Pohakuokala Gulch Bridge South Pier – Weathered Basalt Rock Observed

Alae Bridge (Bridge No. 009003770500058)

The Alae Bridge is located on Kekaulike Avenue at Mile Post 8.57 and spans over the Naalae Gulch. The bridge was constructed in 1932 and is about 150 feet in length and 25 feet in width. The finish grade of the top of the deck is about +3,275 to +2,764 feet MSL. The bridge is a three-span bridge supported by two end abutments and two piers. The two abutments and two piers are supported by spread footings. Based on the available as-built plans, the bottom elevations of the abutment spread footings are approximately +3,244 feet MSL at the north abutment, +3,248 feet MSL at the south abutment, +3,237 feet MSL at the north pier, and +3,219 feet MSL at the south pier. Photographs of the existing bridge structure obtained during our site visit are provided below for reference.



Alae Bridge North Abutment – Surface Fills and Weathered Basalt Observed



South Abutment – Heavy Vegetation Obstructing View of Surface Conditions



Bridge Piers – Looking South



Southeast Alae Channel Sidewalls – Weathered Basalt Outcrop Observed

2.3 Subsurface Conditions

We explored the subsurface conditions at each bridge project site by drilling and sampling six borings, designated as Boring Nos. W-1 and W-2 (Waiale Bridge), P-1 and P-2 (Pohakuokala), and A-1 and A-2 (Alae Bridge), extending to depths ranging from approximately 45 to 51.5 feet below the existing ground surface. Bulk soil samples, designated as Bulk-1 through Bulk-6, were obtained near the ground surface at each boring location to evaluate the pavement support characteristics of the near-surface soils. The approximate boring and bulk sample locations are shown on the Site Plan, Plates 2.1 through 2.3. The subsurface conditions encountered at each bridge are provided in the subsections below.

2.3.1 Waiale Bridge

We explored the subsurface conditions at Waiale Bridge by drilling and sampling two borings, designated as Boring Nos. W-1 and W-2. The borings were drilled generally behind the existing abutment structures. Our field exploration at the bridge generally encountered a pavement structure consisting of 4 to 5 inches of asphaltic concrete overlying 10 to 11 inches of silty gravel base material. The pavement structure was

underlain by fill material generally consisting of loose silty sand and stiff sandy silt extending to depths of about 4.5 to 5.5 feet below the existing pavement surface. The fill was generally underlain by residual soil consisting of soft to stiff clayey silts extending to depths of about 17 to 21 feet below the existing ground surface.

The residual soils were generally underlain by weathered basalt rock formation and basalt rock formation with interbedded layers of clinker materials, extending to the maximum depth explored of about 51 feet below the existing pavement surface. The weathered basalt and clinker generally consisted of dense silty gravel and sandy gravel with some cobbles. The basalt rock formations encountered were generally hard to very hard and moderately to slightly weathered.

We did not encounter groundwater in the borings at the time of our field exploration. However, it should be noted that groundwater levels are subject to change due to rainfall, time of year, seasonal precipitation, surface water runoff, and other factors.

2.3.2 Pohakuokala Bridge

We explored the subsurface conditions at Waiale Bridge by drilling and sampling two borings, designated as Boring Nos. P-1 and P-2. The borings were drilled generally behind the existing abutment structures. Our field exploration at the bridge generally encountered a pavement structure consisting of 5.5 to 9 inches of asphaltic concrete overlying 6 to 9.5 inches of silty and sandy gravel base material. The pavement structure was underlain by fill material and residual soils generally consisting of medium dense to loose silty gravel and stiff to very stiff sandy silts extending to depths of about 8 to 10 feet below the existing pavement surface.

The fill and residual soils were generally underlain by weathered basalt rock formation and basalt rock formation with interbedded layers of clinker materials, extending to the maximum depth explored of about 50.5 feet below the existing pavement surface. The weathered basalt and clinker generally consisted of medium dense to very dense silty sands and sandy gravel. The basalt rock formations encountered were generally hard to very hard and moderately weathered to unweathered.

We did not encounter groundwater in the borings at the time of our field exploration. However, it should be noted that groundwater levels are subject to change due to rainfall, time of year, seasonal precipitation, surface water runoff, and other factors.

2.3.3 Alae Bridge

We explored the subsurface conditions at Alae Bridge by drilling and sampling two borings, designated as Boring Nos. A-1 and A-2. The borings were drilled generally behind the existing abutment structures. Our field exploration at the bridge generally encountered a pavement structure consisting of 4 to 5 inches of asphaltic concrete overlying 13 to 14 inches of silty gravel base material. The pavement structure was underlain by fill material generally consisting of medium dense to very dense silty sand/gravel and gravelly gobbles extending to depths of about 6 to 13 feet below the existing pavement surface. The fill was generally underlain by residual soil consisting of medium dense silty sand and silty gravel, and very dense gravelly cobbles extending to depths of about 6 to 13 feet below the existing ground surface.

The fills were generally underlain colluvium, volcanic ash deposits, residual soils, and saprolite. The colluvial deposits generally consisted of very dense clayey cobbles. The volcanic ash generally consisted of loose to medium dense silty sands. The residual and saprolite generally consisted of stiff sandy silts and very dense sandy gravel. The colluvium, volcanic ash deposits, residual soils and saprolite were generally underlain by basalt rock formation with interbedded layers of clinker materials, extending to the maximum depth explored of about 51.5 feet below the existing pavement surface. The clinker generally consisted of sandy gravel and the basalt rock formations encountered were generally hard to very hard and slightly weathered.

We did not encounter groundwater in the borings at the time of our field exploration. However, it should be noted that groundwater levels are subject to change due to rainfall, time of year, seasonal precipitation, surface water runoff, and other factors.

Detailed descriptions of the field exploration methodology are presented in Appendix A. Descriptions and graphic representations of the materials encountered in the borings are presented on the Logs of Borings in Appendix A. Laboratory tests were performed on selected soil and rock samples, and the test results are presented in Appendix B. Photographs of core samples retrieved from the borings are presented in Appendix C.

2.4 Seismic Design Parameters

Based on the AASHTO 2020 LRFD Bridge Design Specifications, 9th Edition, the project site may be subject to seismic activity, and seismic design considerations will need to be addressed for the project. The following sections provide discussions on seismicity, the potential for liquefaction at the project site, and the soil/rock profile (site class) for seismic design.

2.4.1 Earthquakes and Seismicity

In general, earthquakes throughout the world are caused by shifts in the tectonic plates. In contrast, earthquake activity in Hawaii is linked primarily to volcanic activity; therefore, earthquake activity in Hawaii generally occurs before or during volcanic eruptions. In addition, earthquakes may result from the underground movement of magma that comes close to the surface but does not erupt. The Island of Hawaii experiences thousands of earthquakes each year, but most are so small that they can 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 earthquakes in Hawaii (over 90 percent) are related to volcanic activity, the risk of seismic activity and degree of ground shaking diminishes with increased distance from the Island of Hawaii. The Island of Hawaii has experienced numerous earthquakes greater than Magnitude 5 (M5+); however, earthquakes are not confined only to the Island of Hawaii.

To a lesser degree, the Island of Maui has experienced several earthquakes greater than Magnitude 5. Therefore, moderate to strong earthquakes have occurred in the County of Maui. Due to the relatively short period of documented earthquake monitoring in the State of Hawaii, information pertaining to earthquakes on the Island of Maui may not be complete. However, the following are two examples of reported earthquakes within the County of Maui. A Magnitude 6.8 earthquake occurred on the Island of Maui in 1938, and the Lanai Earthquake of 1871 was estimated at a Magnitude 7 or greater. Based on these two reported earthquakes, the Island of Maui may be considered seismically active.

2.4.2 Liquefaction Potential

Based on the AASHTO 2020 LRFD Bridge Design Specifications, 9th Edition, the project site may be subject to seismic activity, and the potential for soil liquefaction at the project site will need to be evaluated.

Soil liquefaction is a condition where saturated cohesionless soils 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 mobility sufficient 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.

In general, the subsurface information obtained from the drilled borings indicated that the project site is underlain by fills overlying colluvium, volcanic ash, residual soils, and saprolite overlying hard basalt rock formation with interbedded clinker layers. Based on the subsurface conditions encountered in our field exploration, the geology in the area, and our engineering analyses, the potential for soil liquefaction at the project site is non-existent due to the presence of basalt rock formation and the absence of loose granular soils and groundwater table within the depths explored. Therefore, the potential for liquefaction is not a design consideration at this project site.

2.4.3 Soil Profile Type for Seismic Design

Based on the subsurface materials encountered and available data, it is our opinion that the project sites 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 WAIALE BRIDGE AASHTO 2020 LRFD BRIDGE DESIGN SPECIFICATIONS 1,000-YEAR RETURN PERIOD (~7% PROBABILITY OF EXCEEDANCE IN 75 YEARS)	
Parameter	Value
MCE Peak Bedrock Acceleration, PBA (Site Class B)	0.259g
Spectral Response Acceleration (Site Class B), S_s	0.591g
Spectral Response Acceleration (Site Class B), S_1	0.171g
Site Class	"C"
Site Coefficient, F_{pga}	1.14
Site Coefficient, F_a	1.17
Site Coefficient, F_v	1.63
MCE Peak Ground Acceleration, PGA (Site Class C) or A_s	0.296g
Design Spectral Response Acceleration, S_{DS}	0.687g
Design Spectral Response Acceleration, S_{D1}	0.279g

SEISMIC DESIGN PARAMETERS POHAUKOKALA BRIDGE AASHTO 2020 LRFD BRIDGE DESIGN SPECIFICATIONS 1,000-YEAR RETURN PERIOD (~7% PROBABILITY OF EXCEEDANCE IN 75 YEARS)	
Parameter	Value
MCE Peak Bedrock Acceleration, PBA (Site Class B)	0.259g
Spectral Response Acceleration (Site Class B), S_s	0.589g
Spectral Response Acceleration (Site Class B), S_1	0.172g
Site Class	"C"
Site Coefficient, F_{pga}	1.14
Site Coefficient, F_a	1.16
Site Coefficient, F_v	1.63
MCE Peak Ground Acceleration, PGA (Site Class C) or A_s	0.296g
Design Spectral Response Acceleration, S_{DS}	0.687g
Design Spectral Response Acceleration, S_{D1}	0.279g

SEISMIC DESIGN PARAMETERS ALAE BRIDGE AASHTO 2020 LRFD BRIDGE DESIGN SPECIFICATIONS 1,000-YEAR RETURN PERIOD (~7% PROBABILITY OF EXCEEDANCE IN 75 YEARS)	
Parameter	Value
MCE Peak Bedrock Acceleration, PBA (Site Class B)	0.258g
Spectral Response Acceleration (Site Class B), S_s	0.585g
Spectral Response Acceleration (Site Class B), S_1	0.171g
Site Class	"C"
Site Coefficient, F_{pga}	1.14
Site Coefficient, F_a	1.17
Site Coefficient, F_v	1.63
MCE Peak Ground Acceleration, PGA (Site Class C) or A_s	0.294g
Design Spectral Response Acceleration, S_{DS}	0.681g
Design Spectral Response Acceleration, S_{D1}	0.279g

END OF SITE CHARACTERIZATION

SECTION 3. DISCUSSION AND RECOMMENDATIONS

Based on our field exploration and as-built drawings, the existing abutment and pier structures are bearing on the underlying colluvium, residual soils, and hard basalt rock formation. Bearing pressure coefficient of sliding friction and passive pressure resistance parameters were developed for the project.

Based on the information provided, the current retrofit design concept consists of utilizing a tie beam connected from the existing abutments to an anchor beam and soil nails to resist seismically induced loading. To reduce earth pressures on the abutment walls, lightweight cellular concrete backfill will be used as backfill within the tie beam and anchor beam excavations.

A detailed discussion of these items and our geotechnical recommendations for the seismic retrofit design of the existing bridge structures and other geotechnical aspects of the project are presented in the following sections.

3.1 Stiffness Modeling Analysis

In order to evaluate the lateral load resistance of the existing bridge structures, foundation capacities and stiffness modeling parameters were estimated based on the subsurface materials encountered in the borings. Based on the anticipated subsurface conditions, the following analyses were performed in support of the seismic evaluation of the existing bridges:

1. Bearing Capacities of Shallow Foundations
2. Lateral Load Resistance of Shallow Foundations
3. Lateral Load Resistance of Abutment Fills
4. Static Earth Pressure
5. Active Lateral Earth Pressure

Based on the anticipated subsurface conditions, the following subsections provide brief descriptions of the foundation parameters and our preliminary recommendations for use in evaluating the existing bridge structures.

3.1.1 Bearing Capacities of Shallow Foundations

The ultimate and factored bearing capacities of the abutment and pier structures of the Waiale, Pohakuokala, and Alae Bridges supported on shallow foundations are provided in the Summary of Foundation Conditions, Plates 3.1 through 3.3.

3.1.2 Lateral Load Resistance of Shallow Foundations

For lateral load resistance of the shallow foundations, two components of lateral load resistance, as shown on Plates 3.1 through 3.3, may be used in combination. It should be noted that the lateral load resistance of shallow foundations is primarily from friction and adhesion (where provided) between the base of the footings and the supporting subgrade materials. The passive pressure resistance component should be secondary lateral load resistance mechanisms.

The component of passive pressure resistance should be reduced to account for strain compatibility due to the depth of the footings. Please note that the amount of deflection needed to mobilize the full passive pressure is indicated on Plates 3.1 through 3.3. For deflections less than the deflection needed to mobilize full passive pressure, a linearly interpolated reduced passive pressure may be used in evaluating the lateral load resistance of the footing.

3.1.3 Lateral Load Resistance of Abutment Fills

The abutment structures may be considered as a large footing bearing on the abutment fill soils for resistance of lateral loads during a seismic event in the longitudinal direction. For lateral loads imposed on the abutments, the stiffness of the abutment fill soils will generally be mobilized prior to the lateral resistance of its footings.

Provided that intimate contact exists between the backfill soil and the abutments, a soil stiffness spring equal to approximately 4 ksf per inch of deflection may be used in resisting lateral loads. To reduce the potential for shear failure in the abutment fill soils, the lateral deflection should be limited to 1.25 inches or less. Therefore, the maximum lateral load resistance of the abutment structure should be limited to 5 kips per square foot of the abutment wall face. The structural engineer should check the

structure displacement required in order to engage the abutment fill soils for lateral load resistance.

Please note that the stiffness of the abutment fill soils only applies to forces that cause the abutment wall to move into the backfill soil (longitudinal direction normal to the abutment wall face). Therefore, only the lateral load resistance of the abutment foundation and wing walls may be utilized to resist the lateral loads in the transverse direction.

3.1.4 Active Lateral Earth Pressure

Based on the subsurface conditions encountered during our field exploration, we believe compacted fills are anticipated directly behind the abutment walls. For abutment walls with level backfill, the estimated static lateral earth pressures acting on the abutment and wing wall structures are presented in the following table:

STATIC LATERAL EARTH PRESSURES	
<u>Active</u> (pcf)	<u>At-Rest</u> (pcf)
34	53

The lateral earth pressures shown above do not include hydrostatic pressures that might be caused by groundwater trapped behind the structures. The values provided above assume that existing on-site granular soils less than 3 inches in maximum dimension were used to backfill directly behind the retaining structures.

In general, an active condition may be used for structures that are free to deflect laterally by as much as 0.5 percent of the structure height. If the tops of the structures are not free to deflect beyond this degree or are restrained, the retaining structures should be evaluated for the at-rest condition.

Surcharge stresses due to areal surcharges, line loads, and point loads within a horizontal distance equal to the depth of the retaining structure should be considered in the design. For uniform surcharge stresses imposed on the loaded side of the structure, a rectangular distribution with a uniform pressure equal to 34 percent of the vertical surcharge pressure acting on the entire height of the structure, which is

free to deflect (cantilever), may be used in the design. For retaining structures that are restrained, a rectangular distribution equal to 53 percent of the vertical surcharge pressure acting over the entire height of the structure may be used for the design. Additional analyses during the design may be needed to evaluate the surcharge effects of point loads and line loads.

We understand that lightweight cellular concrete backfill will be used as backfill within the tie beam and anchor beam excavations. Lateral earth pressures acting on the existing abutment walls within the lightweight cellular concrete backfill zone will be negligible.

3.1.5 Dynamic Lateral Earth Pressure

Dynamic lateral earth forces due to seismic loading will need to be considered in the design of the retaining structures based on LRFD design methods. Seismic loading (PGA or $a_{max} = 0.296g$) is used to estimate the dynamic lateral earth pressures based on a peak bedrock acceleration (PBA) of $0.259g$. The table below summarizes the dynamic lateral earth forces acting on the retaining structure in the event of an earthquake.

DYNAMIC LATERAL EARTH PRESSURES DUE TO SEISMIC LOADING		
<u>Backfill Condition</u>	<u>Active Condition</u> (H^2 pounds per linear foot)	<u>At-Rest Condition</u> (H^2 pounds per linear foot)
Level	5.6	13.3

The active condition assumes that the walls will be allowed to move laterally by up to about 1 to 2 inches in the event of an earthquake. The at-rest condition should be used when walls are restrained (<0.5 inches of lateral movement). The resultant force of the dynamic lateral earth pressures should be assumed to act through the mid-height of the wall. It should be noted that the forces due to dynamic lateral earth pressures presented above are in addition to the static lateral earth pressures.

3.2 Soil-Nailed Anchor Beams

Based on the information provided, the current retrofit design concept consists of utilizing a tie beam connected from the existing abutments to an anchor beam and soil nails to resist seismically induced loading.

Items pertaining to the soil-nailed anchor beams are addressed in the subsequent subsections and include the following:

1. Excavation Preparation
2. Soil Nailed Anchor Beams
3. Soil Nail Installation
4. Soil Nail Testing

Construction of the soil-nailed anchor beam system should be performed by a specialty contractor experienced in the construction of soil-nails. Due to the specialized nature of the construction, a Geolabs representative should be present to observe the geotechnical aspects of the soil-nailed anchor beam system and testing of the soil nails.

3.2.1 Excavation Preparation

At the on-set of construction, loose material and debris should be removed from the face of the excavation to receive soil nails. Care should be taken in removing material so that the stability of the excavation is not compromised.

3.2.2 Soil Nailed Anchor Beams

A soil-nailed anchor beam system consists of a series of individual reinforcing bars grouted into drilled holes. The soil nails connect to an anchor beam, which will subsequently connect to a series of tie beams. The tie beam will connect to the back of the existing abutments to resist seismically induced loading. Based on the information provided, four tie beams will be required per bridge abutment. The tie beams will induce a total load demand of 160 kips onto the proposed anchor beam. Based on the current design concept, four soil nails with allowable tensile load capacities of 40 kips per anchor per soil nail will be utilized.

We understand that all bridge abutments will require the use of soil nails to anchor the tie beams. Based on the anticipated load demand, we recommended installing the soil nails by drilling a minimum 8-inch diameter hole with an inclination of

approximately 20 degrees from horizontal. The soil nail bar should consist of ASTM A615 Grade 75 solid threaded bar with a minimum bar diameter of 1.5 inches. We recommend using a double corrosion protection system for the nails. 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. A corrosion inhibitor, such as Coretec MCI 2005 NS Corrosion Inhibitor or equivalent, should also be included in the grout mix for the soil nail bar.

Based on our soil nail analysis, we recommend using design nail lengths of 30 to 45 feet, extending into the fill, residual soils, ash deposits, and basalt formations encountered in our field exploration. The following table provides a summary of soil nails for each bridge location.

SOIL NAIL SUMMARY		
WAIALE BRIDGE		
<u>Location</u>	<u>Abutment No. 1</u> (North)	<u>Abutment No. 2</u> (South)
Soil Nail Inclination (degrees)	20	20
Soil Nail Length (feet)	45	45
Drill Hole Diameter (feet)	8	8
Soil Nail Bar Size	No. 11 (Grade 75)	No. 11 (Grade 75)
POHAKUOKALA BRIDGE		
<u>Location</u>	<u>Abutment No. 1</u> (North)	<u>Abutment No. 2</u> (South)
Soil Nail Inclination (degrees)	20	20
Soil Nail Length (feet)	35	35
Drill Hole Diameter (feet)	8	8
Soil Nail Bar Size	No. 11 (Grade 75)	No. 11 (Grade 75)

SOIL NAIL SUMMARY		
ALAE BRIDGE		
<u>Location</u>	<u>Abutment No. 1</u> (North)	<u>Abutment No. 2</u> (South)
Soil Nail Inclination (degrees)	20	20
Soil Nail Length (feet)	30	30
Drill Hole Diameter (feet)	8	8
Soil Nail Bar Size	No. 11 (Grade 75)	No. 11 (Grade 75)

3.2.3 Soil Nail Installation

Construction of the permanent soil nails should be performed by a specialty contractor experienced in the construction of soil nails. Due to the specialized nature of the soil nail construction, a Geolabs representative should be present to observe the geotechnical aspects of the soil nail construction. Potentially difficult drilling conditions may be encountered during the installation of the soil nails due to the potential of cobbles and boulders within the fill materials anticipated at the project site. Utilizing a temporary casing may be required to maintain an open hole for the soil nail installation. In addition, due to potential variations of basalt rock within the subsurface, hard drilling conditions may be anticipated. The contractor should fully investigate the project site and make his/her own interpretation as to the character of the materials to be encountered. It is the contractor's responsibility to bid the project accordingly with sufficient contingency, taking into account potential caving-in, hard drilling, and difficult access conditions.

3.2.4 Soil Nail Testing

Based on the subsurface conditions anticipated behind the soil-nailed anchor beam, we recommend performing a minimum of one pre-production verification test on a sacrificial soil nail at each bridge site. In addition, proof tests should be performed on at least 10 percent of the production soil nails during construction to confirm the bond stresses used in the design.

The verification test should be loaded gradually to 150 percent of the design load for the creep test and subsequently to the maximum test load of about 200 percent of

the design load. In addition, the load at every loading increment should be held for at least 10 minutes, and creep tests should be held for at least 60 minutes. The proof tests should consist of subjecting the soil nail to at least 150 percent of the design loads and the load should be held for at least 10 minutes (until stable). The proof test nails may be incorporated into the permanent soil nail retaining wall, provided they satisfy the test criteria. Pullout tests and creep tests on the soil nails are integral components of the design for the soil nail retaining wall system. Therefore, we recommend conducting pullout tests and creep tests in the presence of a Geolabs representative.

3.3 Pavement

We understand new pavement will be required at each abutment to replace the roadway pavement structure that will be removed during the seismic retrofit project. In general, asphaltic concrete pavement overlying reinforced concrete relief slabs will be utilized behind the existing abutments. However, pavements overlying existing in-situ soils will be required beyond the relief slabs. Asphaltic concrete pavement sections beyond the relief slabs are provided in subsequent sections. The flexible pavement sections presented herein were generally determined based on the methodology described in Chapter 3 of the revised Pavement Design Manual dated March 2002. The Pavement Design Manual was prepared by the State of Hawaii - Department of Transportation, Highways Division, Material Testing and Research Branch. The pavement design methodology is based on the Hveem Stabilometer method developed and used by the California Department of Transportation (Caltrans).

3.3.1 Design Traffic Loading Conditions/Traffic Index

Based on the design guidelines from the revised Pavement Design Manual dated March 2002, the roadway at Waialae and Pohakuokala Bridges may be classified as a “Medium-Volume Rural” roadway, and the roadway at Alae Bridge may be classified as a “Low-Volume Paved” roadway. Therefore, new pavements for this project will need to be designed for a pavement life of 20 years. Design traffic parameters were obtained from the HDOT Planning Survey Engineer traffic data memorandum dated April 12, 2024. The following tables summarize the design traffic parameters used in our pavement analyses.

DESIGN TRAFFIC PARAMETERS FOR FLEXIBLE AND RIGID PAVEMENTS	
Waialae and Pohakuokala Bridge	
Design Period	20 Years
Average Daily Traffic (ADT)	Vehicles per day per direction
At Project Completion	4,500
50 Years After Project Completion	5,700
24-Hour Truck Traffic	6%
<u>Type of Axle</u>	<u>Truck Traffic Distribution</u>
2-Axle	79.94%
3-Axle	17.76%
4-Axle	0.21%
5-Axle	0.14%
6-Axle	0.29%
7-Axle	1.66%
Alae Bridge	
Design Period	20 Years
Average Daily Traffic (ADT)	Vehicles per day per direction
At Project Completion	1,700
50 Years After Project Completion	2,100
24-Hour Truck Traffic	6%
<u>Type of Axle</u>	<u>Truck Traffic Distribution</u>
2-Axle	83.15%
3-Axle	14.15%
4-Axle	1.11%
5-Axle	0.00%
6-Axle	0.21%
7-Axle	1.39%

Based on a design period of 20 years, the traffic volume provided, and the assumed truck traffic distribution, a Traffic Index (TI) of 9 was determined for the Waialae and Pohakuokala Bridges, and a Traffic Index (TI) of 8 was determined for the Alae Bridge project, respectively.

3.3.2 Design Subgrade Conditions

Based on our field exploration and the current design concept, we anticipate that the pavement subgrade soils may consist of recompacted on-site soils. We anticipate that majority of the pavement subgrade soils generally will consist of silty sands and sandy silts.

Laboratory Resistance (R) value testing was performed on the near-surface soils at each bridge abutment to evaluate the use of the near-surface soils as roadway fill material. The R-values obtained for the existing subgrade soils ranged from 10 to 70. A summary of R-value test results is provided in the table below.

SUMMARY OF R-VALUE TEST RESULTS		
Location	Bulk Surface Sample	R-Value
Waiale Bridge	Bulk-1	26
Waiale Bridge	Bulk-2	25
Pohakuokala Bridge	Bulk-3	70
Pohakuokala Bridge	Bulk-4	10
Alae Bridge	Bulk-5	23
Alae Bridge	Bulk-6	21

The results of the laboratory R-value tests are presented in Appendix B. If site grading exposes soils other than those assumed in the pavement design, additional tests should be performed to confirm and/or revise the recommended pavement section for actual field conditions.

3.3.3 Design Pavement Section

Typically, new HDOT pavements include a drainage layer to facilitate drainage and increase the pavement life. However, because the pavement sections at the existing bridges do not have a drainage layer, only pavement structural sections without a permeable drainage layer were considered in our pavement analyses.

Based on the results of our subsurface exploration and the R-value results obtained, a flexible pavement section was determined for each bridge. The HDOT Materials Testing and Research Branch indicated that the thickness of PMA concrete is equivalent to about two times the thickness of hot mix asphaltic concrete.

Flexible Pavement (Waiale Bridge)

2.0-Inch Polymer-Modified Asphaltic (PMA) Concrete
4.0-Inch Asphalt Concrete Base (92 Percent Max Theoretical Specific Gravity)
10.0-Inch Aggregate Subbase (95 Percent Relative Compaction)
16.0-Inch Total Pavement Thickness on Compacted Subgrade

Flexible Pavement (Pohakuokala Bridge)

2.0-Inch Polymer-Modified Asphaltic (PMA) Concrete
6.0-Inch Asphalt Concrete Base (92 Percent Max Theoretical Specific Gravity)
12.0-Inch Aggregate Subbase (95 Percent Relative Compaction)
20.0-Inch Total Pavement Thickness on Compacted Subgrade

Flexible Pavement (Alae Bridge)

2.0-Inch Polymer-Modified Asphaltic (PMA) Concrete
4.0-Inch Asphalt Concrete Base (92 Percent Max Theoretical Specific Gravity)
12.0-Inch Aggregate Subbase (95 Percent Relative Compaction)
18.0-Inch Total Pavement Thickness on Compacted Subgrade

Where asphaltic concrete pavement is placed over reinforced concrete relief slabs, we recommend the use of pavement reinforcement, such as Tensar Rapid Repair PG100 (or approved equivalent), to reduce the potential for reflective cracking. The pavement reinforcement should be installed along the full length of the asphaltic concrete pavement overlying the concrete slab and extended a minimum of 5 feet beyond each end of the slab.

3.3.4 Subgrade Preparation Below Pavement Sections

We anticipate that the project pavements will be constructed over the subgrade under the existing pavement. In general, the area within the contract grading limits should be cleared and grubbed thoroughly at the on-set of earthwork. Vegetation, debris, deleterious materials, and other unsuitable materials should be removed and properly disposed of off-site or in a designated area to reduce the potential for contamination of the excavated materials.

Soft and yielding areas encountered during clearing and grubbing below areas designated to receive future improvements should be over-excavated to expose stiff and/or dense materials. The resulting excavation should be backfilled with aggregate subbase materials. The excavated soft and/or organic soils should be properly disposed of off-site.

After clearing and grubbing, the future pavement areas should be excavated, where necessary, to the pavement subgrade level (i.e., bottom of the aggregate subbase layer). The pavement subgrade soils should be scarified to a depth of at least 8 inches (where possible), moisture-conditioned to at least 2 percent above optimum moisture content and compacted to no less than 95 percent relative compaction. Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same soil determined in accordance with AASHTO T180 (ASTM D1557). Optimum moisture is the water content (percentage by dry weight) corresponding to the maximum dry density.

Where shrinkage cracks are noted after compaction of the subgrade soils, we recommend scarifying and re-preparing the soils as recommended above. Saturation and subsequent yielding of the exposed subgrade due to inclement weather and poor drainage may require over-excavation of the soft areas and replacement with well-compacted fill.

3.3.5 Fill Materials

In general, the excavated on-site near-surface soils may be used as a source of general fill materials, provided that deleterious materials such as vegetation are removed and over-sized materials greater than 3 inches in maximum dimension are screened. Imported fill materials, if required, should consist of non-expansive select granular material, such as crushed coralline or basaltic materials. The materials should be well-graded from coarse to fine with particles no larger than 3 inches in the largest dimension and should contain between 10 and 30 percent particles passing the No. 200 sieve. The materials should have a laboratory California Bearing Ratio value of 20 or more and should have a maximum swell of 1 percent or less. Imported

fill materials should be tested for conformance with these recommendations prior to delivery to the project site for the intended use.

3.3.6 Compaction Requirements

In general, fill and backfill materials should be moisture-conditioned to above the optimum moisture content, placed in level lifts not exceeding 8 inches in loose thickness, and compacted to a minimum of 95 percent relative compaction. Aggregate base course (ABC) materials required for the new pavements should conform to the requirements stipulated in Subsection 703.06 of the Hawaii State Standards. The ABC should be moisture-conditioned to above the optimum moisture content, placed in 8-inch level loose lifts, and compacted to no less than 95 percent of the maximum density according to AASHTO T180 (ASTM D1557).

Asphaltic concrete base material should consist of asphalt-treated basaltic aggregate, placed in a layer not to exceed 6 inches in compacted thickness, and compacted to at least 92 percent of the maximum theoretical specific gravity determined in accordance with AASHTO T209 (ASTM D2041).

Asphaltic concrete material should be constructed in general accordance with Section 401 – Hot Mix Asphalt Pavement of the Hawaii State Standards.

Field density tests should be performed on the compacted fills and backfills in general accordance with ASTM D1556. In general, field density tests should be performed at the frequencies presented in the following table:

FIELD DENSITY TESTING FREQUENCY		
Material	Location of Material	Test Frequency
Treated Base	Pavements and Shoulders	One test per 100 linear feet of roadway per lift.
Aggregate Subbase	Pavements and Shoulders	One test per 100 linear feet of roadway per lift.
Subgrade (Sands and Gravels)	Pavements and Shoulders	One test per 100 linear feet of roadway per lift.
Trench Backfill	Utility Trenches	One test per 200 linear feet of trench per lift of backfill.

3.3.7 Pavement Drainage

Preliminary and final drawings and specifications for the proposed construction should be forwarded to Geolabs for review and written comments. This review is necessary to evaluate the general conformance of the plans and specifications with the intent of the pavement design recommendations provided herein. If this review is not made, Geolabs cannot be responsible for misinterpretation of our recommendations.

3.4 Design Review

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

3.5 Post-Design Services/Services During Construction

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

- Observation of the slope restoration backfill placement
- Observation of the soil nail load test program
- Observation of the production soil nail installation
- Observation of shotcrete placement
- Observation of pavement subgrade preparation and compaction

A Geolabs representative should also monitor other aspects of earthwork construction to observe compliance with the design concepts, specifications, or recommendations and to expedite suggestions for design changes that may be required in the event that subsurface conditions differ from those anticipated at the time this report was prepared. Geolabs should be accorded the opportunity to provide geotechnical engineering services during construction to confirm our assumptions in providing the recommendations presented herein.

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

END OF DISCUSSION AND RECOMMENDATIONS

SECTION 4. LIMITATIONS

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

The coordinates of the as-drilled boring locations indicated in this report are approximate, having been estimated by using a hand-held Global Positioning System (GPS) device. Elevations noted on borings were estimated from as-built drawings dated 1929 (Waiale and Pohakuokala Bridges) and 1932 (Alae Bridge). The physical locations and elevations of the borings should be considered accurate only to the degree implied by the methods used.

The stratification lines shown on the graphic representations of the borings depict the approximate boundaries between soil/rock types and, as such, may denote a gradual transition. Water level data from the borings were measured at times shown on the graphic representations and/or presented in the text of this report. These data have been reviewed, and interpretations made in the formulation of this report. It should be noted that groundwater levels are subject to change due to rainfall, seasonal precipitation, surface water runoff, and other factors.

This report has been prepared for the exclusive use of WSP for specific application to the proposed *Waiale, Pohakuokala, and Alae Bridges Seismic Retrofit* 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 design engineers in the design of the project. Therefore, this report may not contain sufficient data or the proper information to serve as a basis for determining construction cost estimates.

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

This geotechnical engineering exploration conducted at the project site was not intended to investigate the potential presence of hazardous materials existing at the 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

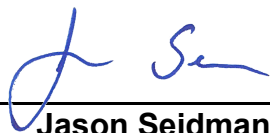
The following plates and appendices are attached and complete this report:

Project Location Map.....	Plate 1
Site Plan.....	Plates 2.1 thru 2.3
Summary of Foundation Conditions	Plates 3.1 thru 3.3
Field Exploration	Appendix A
Laboratory Tests	Appendix B
Photographs of Core Samples	Appendix C

-ΩΩΩΩΩΩΩΩΩ-

Respectfully submitted,

GEOLABS, INC.

By 
Jason Seidman
Senior Project Engineer

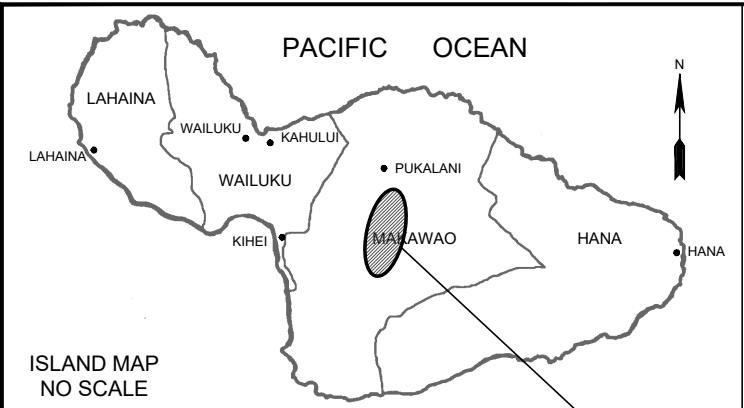
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Vice President

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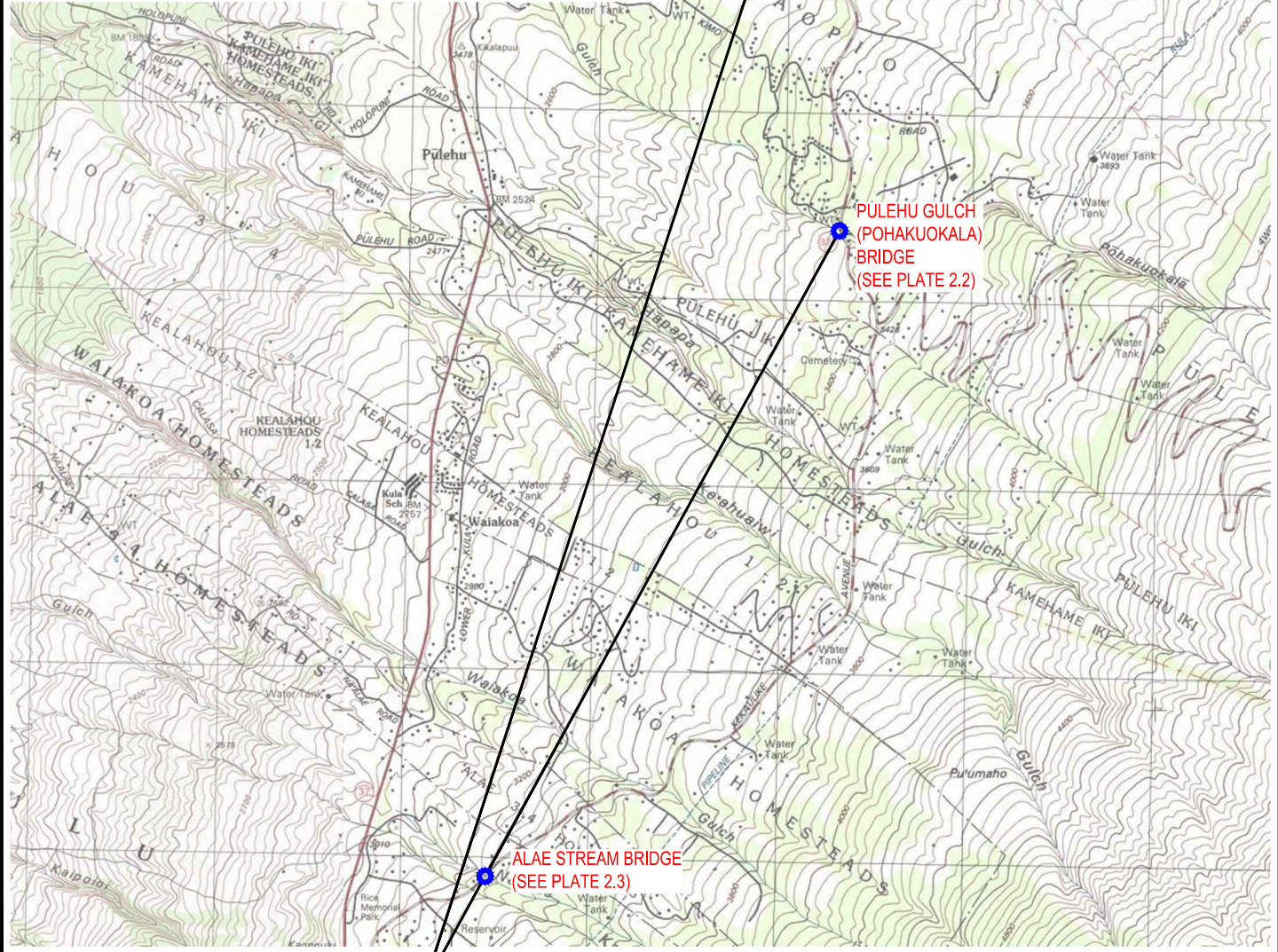
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PLATES

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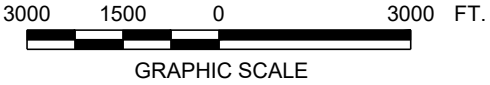


GENERAL PROJECT LOCATION»



PROJECT LOCATIONS»

PROJECT LOCATION MAP
WAIALE, POHAKUOKALA, AND ALAE BRIDGES SEISMIC RETROFIT
KULA, MAUI, HAWAII



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



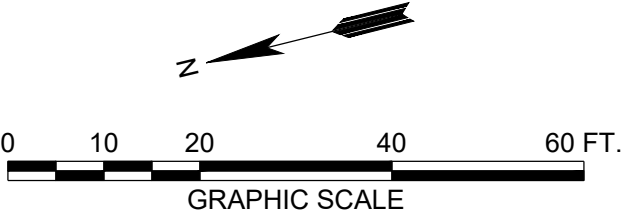
GEOLABS, INC.		
Geotechnical Engineering		
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SCALE	W.O.	
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SITE PLAN - WIALE GULCH BRIDGE
WIALE, POHAKUOKALA, AND ALAE BRIDGES SEISMIC RETROFIT
KULA, MAUI, HAWAII

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

REFERENCE: GOOGLE EARTH™ PRO. ©2024 GOOGLE, INC.
IMAGERY DATED FEBRUARY 11, 2024.



GEOLABS, INC.		
Geotechnical Engineering		
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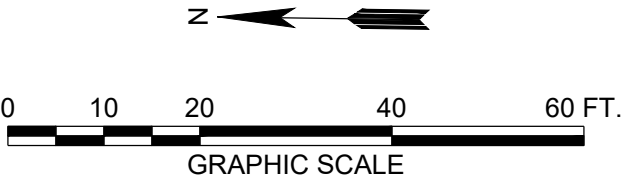
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LEGEND:

-  APPROXIMATE BORING LOCATION
-  APPROXIMATE BULK SAMPLE LOCATION

REFERENCE: GOOGLE EARTH™ PRO. ©2024 GOOGLE, INC.
IMAGERY DATED FEBRUARY 11, 2024.



**SITE PLAN - PULEHU GULCH
(POHAKUOKALA) BRIDGE**

WAIALE, POHAKUOKALA, AND ALAE BRIDGES SEISMIC RETROFIT
KULA, MAUI, HAWAII



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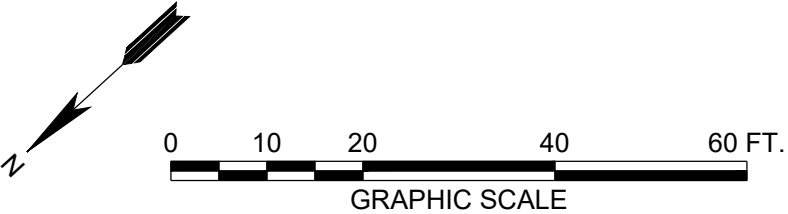
Geotechnical Engineering

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

REFERENCE: GOOGLE EARTH™ PRO. ©2024 GOOGLE, INC.
IMAGERY DATED JANUARY 23, 2022.



SITE PLAN - ALAE STREAM BRIDGE
WAIALE, POHAKUOKALA, AND ALAE BRIDGES SEISMIC RETROFIT
KULA, MAUI, HAWAII

GEOLABS, INC.		
Geotechnical Engineering		
DATE	DRAWN BY	PLATE
APRIL 2024	HYC	
SCALE	W.O.	2.3
1" = 20'	8412-00(B)	

SUMMARY OF FOUNDATION CONDITIONS

WAIALE BRIDGE

Location		Abutment #1 (North)	Pier #1 (North)	Pier #2 (South)	Abutment #2 (South)
Type of Foundation		Strip Footing	Spread Footing	Spread Footing	Strip Footing
Approx. Size of Footing		31.5' x 4'	5' x 7'	5' x 7'	31' x 3.25'
Elevation (feet MSL)	Existing Ground Elevation	+2760.0	+2761.0	+2762.0	+2764.0
	Approx. Bottom of Footing Elevation	+2750.0	+2728.0	+2726.0	+2757.0
Estimated Subsurface Conditions		Fill / Residual Soil	Basalt Rock	Basalt Rock	Fill / Residual Soil
Site Class (Seismic Analysis)		Site Class C (AASHTO LRFD, 2020)			
Foundation Capacity	Original Design Bearing Pressure (ksf)	2	5.4	5.4	2.6
	Estimated Ultimate Bearing Capacity (ksf)	12	30	30	12
	Factored Capacity (Strength Limit State)	5.4	13.5	13.5	5.4
	Vertical Spring (lbs/in ³)	N/A	460	460	N/A
Lateral Load Resistance (Pile Foundation)		N/A	N/A	N/A	N/A
Lateral Load Resistance (Shallow Foundation)	Sliding Resistance	Extreme Event Limit State Friction, $\tan \delta$	0.51	0.70	0.70
		Strength Limit State Friction, $\tan \delta$	0.40	0.56	0.56
	Passive Pressure Resistance	Extreme Event Limit State Friction, $K_p \gamma$ (pcf)	400	550	550
		Strength Limit State Friction, $K_p \gamma$ (pcf)	200	275	275
	Deflection to Mobilize Full Passive (inches)		0.16	0.1	0.1

SUMMARY OF FOUNDATION CONDITIONS

POHAKUOKALA BRIDGE

Location		Abutment #1 (North)	Pier #1 (North)	Pier #2 (South)	Abutment #2 (South)
Type of Foundation		Strip Footing	Strip Footing	Strip Footing	Strip Footing
Approx. Size of Footing		26' x 4'	26' x 4.5'	26' x 4.5'	26' x 2.5'
Elevation (feet MSL)	Existing Ground Elevation	+3273.0	+3275.0	+3277.0	+3279.0
	Approx. Bottom of Footing Elevation	+3260.0	+3244.0	+3244.0	+3275.0
Estimated Subsurface Conditions		Fill / Weathered Rock	Basalt Rock	Basalt Rock	Fill / Residual Soil / Weathered Rock
Site Class (Seismic Analysis)		Site Class C (AASHTO LRFD, 2020)			
Foundation Capacity	Original Design Bearing Pressure (ksf)	7	3.2	3.2	3
	Estimated Ultimate Bearing Capacity (ksf)	20	30	30	20
	Factored Capacity (Strength Limit State)	9	13.5	13.5	9
	Vertical Spring (lbs/in ³)	N/A	460	460	N/A
Lateral Load Resistance (Pile Foundation)		N/A	N/A	N/A	N/A
Lateral Load Resistance (Shallow Foundation)	Sliding Resistance	Extreme Event Limit State Friction, $\tan \delta$	0.51	0.70	0.70
		Strength Limit State Friction, $\tan \delta$	0.40	0.56	0.56
	Passive Pressure Resistance	Extreme Event Limit State Friction, $K_p \gamma$ (pcf)	400	550	550
		Strength Limit State Friction, $K_p \gamma$ (pcf)	200	275	275
	Deflection to Mobilize Full Passive (inches)		0.24	0.024	0.024

SUMMARY OF FOUNDATION CONDITIONS

ALAE BRIDGE

Location			Abutment #1 (North)	Pier #1 (North)	Pier #2 (South)	Abutment #2 (South)
Type of Foundation			Strip Footing	Spread Footing	Spread Footing	Strip Footing
Approx. Size of Footing			28' x 12.7'	5' x 7'	5' x 7'	28' x 11'
Elevation (feet MSL)	Existing Ground Elevation		+3270.0	+3269.0	+3267.0	+3266.0
	Approx. Bottom of Footing Elevation		+3244.0	+3237.0	+3219.0	+3248.0
Estimated Subsurface Conditions			Fill / Colluvium	Weathered Rock	Basalt Rock	Fill / Volcanic Ash/ Residual Soil
Site Class (Seismic Analysis)			Site Class C (AASHTO LRFD, 2020)			
Foundation Capacity	Original Design Bearing Pressure (ksf)		N/A	N/A	N/A	N/A
	Estimated Ultimate Bearing Capacity (ksf)		20	20	30	20
	Factored Capacity (Strength Limit State)		9	9	13.5	9
	Vertical Spring (lbs/in ³)		N/A	180	460	N/A
Lateral Load Resistance (Pile Foundation)			N/A	N/A	N/A	N/A
Lateral Load Resistance (Shallow Foundation)	Sliding Resistance	Extreme Event Limit State Friction, $\tan \delta$	0.51	0.51	0.70	0.51
		Strength Limit State Friction, $\tan \delta$	0.40	0.40	0.56	0.40
	Passive Pressure Resistance	Extreme Event Limit State Friction, $K_p \gamma$ (pcf)	400	400	550	400
		Strength Limit State Friction, $K_p \gamma$ (pcf)	200	200	275	200
	Deflection to Mobilize Full Passive (inches)		0.12	0.024	0.024	0.12

APPENDIX A

APPENDIX A

Field Exploration

We explored the subsurface conditions at the project site by drilling and sampling six borings, designated as Boring Nos. W-1 and W-2 (Waiale Bridge), P-1 and P-2 (Pohakuokala), and A-1 and A-2 (Alae Bridge), extending to depths ranging from approximately 45 to 51.5 feet below the existing ground surface. The borings were drilled using a truck-mounted drill rig equipped with continuous flight auger and coring tools. The boring locations are presented on the Site Plans, Plates 2.1 through 2.3.

Our geologist 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 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 in 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-6.2.

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 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.

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

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). Graphical representations of the materials encountered are presented on the Logs of Borings at the appropriate depths.

Recovery (REC) may be used as a subjective guide to the interpretation of the relative quality of rock masses, 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).

<u>Rock Quality</u>	<u>RQD</u> (%)
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.



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Soil Log Legend

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)

MAJOR DIVISIONS			USCS		TYPICAL DESCRIPTIONS		
COARSE-GRAINED SOILS	GRAVELS	CLEAN GRAVELS		GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES		
		LESS THAN 5% FINES		GP	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES		
		GRAVELS WITH FINES		GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES		
		MORE THAN 12% FINES		GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES		
	SANDS	CLEAN SANDS		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES		
		LESS THAN 5% FINES		SP	POORLY-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES		
		SANDS WITH FINES		SM	SILTY SANDS, SAND-SILT MIXTURES		
		MORE THAN 12% FINES		SC	CLAYEY SANDS, SAND-CLAY MIXTURES		
FINE-GRAINED SOILS	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)

Plate

A-0.1



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Soil Classification Log Key

(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) 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. accessory descriptions compose of the following: - with some: >12% - with a little: 5 - 12% - with traces of: <5% accessory descriptions are lower cased and follow the Primary and Secondary Constituents (i.e., SILTY GRAVEL with a little 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, 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: - with some: >12% - with a little: 5 - 12% - with traces of: <5% accessory descriptions are lower cased and follow the Primary and Secondary Constituents (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)

Plate

A-0.2

*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).



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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. Example: Dense, fine grain volcanic rock
Hard:	Specimen breaks with some difficulty after several hammer blows. Example: Vesicular, vugular, coarse-grained rock
Medium Hard:	Specimen can be broke by one hammer blow. Cannot be scraped by knife. SPT may penetrate by ~25 blows per inch with bounce. 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. 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. Example: Saprolite

Plate

A-0.3



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SEISMIC RETROFIT
KULA, MAUI, HAWAII

Log of
Boring

A-1

Laboratory			Field				Depth (feet)	Sample	Graphic	USCS	Approximate Ground Surface Elevation (feet): 3270 *
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)					Description
Sieve - #200 = 13.8%	20	93			24					GM	5-inch ASPHALTIC CONCRETE
	26				26						Brownish gray SILTY GRAVEL (BASALTIC) with some sand, moist (base material)
Direct Shear	18	62	40		22		5				Brown with black mottling SILTY GRAVEL with some sand and cobbles (basaltic), medium dense, moist (fill)
			27		10/0" Ref.		10				Gray GRAVELLY COBBLES , dense to very dense, moist (fill)
			10		25/1"		15				Gray with brown CLAYEY COBBLES (BASALTIC) with some sand, very dense, moist (colluvium)
					4		20				grades to loose
			0				25			SM	Brown SILTY SAND with a little gravel (basaltic), loose to medium dense (volcanic ash)
	89		0		6		30				
Sieve - #200 = 23.5%	84		26		11		35				

Date Started: July 17, 2023

Date Completed: July 17, 2023

Logged By: B. Aiu

Total Depth: 51.5 feet

Work Order: 8412-00(B)

Water Level: ▼ Not Encountered

Drill Rig: CME-45C TRUCK (Energy Transfer Ratio = 86.4%)

Drilling Method: 4" Solid-Stem Auger & PQ Coring

Driving Energy: 140 lb. wt., 30 in. drop

Plate

A - 1.1



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SEISMIC RETROFIT
KULA, MAUI, HAWAII

Log of
Boring

A-1

Laboratory			Field				Depth (feet)	Sample	Graphic	USCS	(Continued from previous plate)
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)					Description
UC= 9250 psi			8	0	25/1"		40			SM	grades to very dense with cobbles and boulders
			85	37	10/0" Ref.		45				Gray BASALT , closely to moderately fractured, slightly weathered, hard to very hard (basalt formation)
			65	23			50				
UC= 20490 psi							55				Boring terminated at 51.5 feet
							60				* Elevations estimated from As-built drawings dated 1929 (Waiale Bridge and Pohakualoa) and 1932 (Alae Bridge).
							65				
							70				

Date Started: July 17, 2023

Date Completed: July 17, 2023

Logged By: B. Aiu

Total Depth: 51.5 feet

Work Order: 8412-00(B)

Water Level: ▼ Not Encountered

Drill Rig: CME-45C TRUCK (Energy Transfer Ratio = 86.4%)

Drilling Method: 4" Solid-Stem Auger & PQ Coring

Driving Energy: 140 lb. wt., 30 in. drop

Plate

A - 1.2



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KULA, MAUI, HAWAII

Log of
Boring

A-2

Laboratory			Field				Depth (feet)	Sample	Graphic	USCS	Approximate Ground Surface Elevation (feet): 3265 *
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)					Description
Direct Shear Sieve - #200 = 16.0% LL=NP PI=NP TXUU S _u =1.1 ksf	9	84			49					GM	4-inch ASPHALTIC CONCRETE
	35				13					SM	Brownish gray SILTY GRAVEL (BASALTIC) with some sand, medium dense, moist (base material)
	47	56			9	0.8	5				Brownish gray with black mottling SILTY SAND with some gravel, medium dense, moist (fill)
										SM	Brown with black mottling SILTY SAND with a little fine gravel, loose, moist (volcanic ash)
Sieve - #200 = 34.4%	52				5		10				
TXUU S _u =0.5 ksf	51	47			15	1.5	15				
TXUU S _u =5.0 ksf	42	80			25	2.0	20			ML	Brown and gray SANDY SILT with some decomposed gravel, stiff, moist (residual soil)
Sieve - #200 = 7.1%	8	117			99		25			GP- GM	Gray with brown mottling SANDY GRAVEL (BASALTIC) with a little silt, very dense, moist (saprolite)
UC= 15060 psi UC= 12660 psi			100	80			30				Gray BASALT , moderately fractured, slightly weathered, hard to very hard (basalt formation)
			100	60			35				

Date Started: July 18, 2023

Date Completed: July 18, 2023

Logged By: B. Aiu

Total Depth: 50 feet

Work Order: 8412-00(B)

Water Level: ▼ Not Encountered

Drill Rig: CME-45C TRUCK (Energy Transfer Ratio = 86.4%)

Drilling Method: 4" Solid-Stem Auger & PQ Coring

Driving Energy: 140 lb. wt., 30 in. drop

Plate

A - 2.1



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Log of
Boring

A-2

Laboratory			Field				Depth (feet)	Sample	Graphic	USCS	(Continued from previous plate)
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)					Description
UC= 16380 psi			63	0						GP	Gray SANDY GRAVEL (BASALTIC) with a little cobbles, moist (clinker)
			67	38			40				Gray BASALT , moderately fractured, slightly weathered, hard to very hard
			40	10			45				
							50				Boring terminated at 50 feet
							55				
							60				
							65				
							70				

Date Started: July 18, 2023

Date Completed: July 18, 2023

Logged By: B. Aiu

Total Depth: 50 feet

Work Order: 8412-00(B)

Water Level: ▼ Not Encountered

Drill Rig: CME-45C TRUCK (Energy Transfer Ratio = 86.4%)

Drilling Method: 4" Solid-Stem Auger & PQ Coring

Driving Energy: 140 lb. wt., 30 in. drop

Plate

A - 2.2



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Log of
Boring

P-1

Laboratory			Field				Depth (feet)	Sample	Graphic	USCS	Approximate Ground Surface Elevation (feet): 3274 *
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)					Description
Direct Shear Sieve - #200 = 17.8% Direct Shear	33	60			23						9-inch ASPHALTIC CONCRETE
	23				4					GM GM	Gray SILTY GRAVEL , medium dense (base material) Brown SILTY GRAVEL with some sand, medium dense to loose, moist (fill)
	18	60			17		5				
Sieve - #200 = 30.9%	32				3/6" +50/4"		10			SM	Brown SILTY SAND with some gravel, very dense, moist (weathered basalt formation) Gray BASALT , severely fractured, unweathered to moderately weathered, hard to very hard (basalt formation) grades to moderately fractured grades to slightly fractured
			99	0							
			95	95			15				
UC= 10210 psi							20				
UC= 18000 psi			100	47							
							25				
			100	72							
UC= 18360 psi							30				
UC= 7870 psi			100	33							grades to severely to closely fractured
							35				

Date Started: July 13, 2023

Date Completed: July 14, 2023

Logged By: N. Colter

Total Depth: 45 feet

Work Order: 8412-00(B)

Water Level: ▼ Not Encountered

Drill Rig: CME-45C TRUCK (Energy Transfer Ratio = 86.4%)

Drilling Method: 4" Solid-Stem Auger & HQ Coring

Driving Energy: 140 lb. wt., 30 in. drop

Plate

A - 3.1



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WAIKALE, POHAUOKALA, AND ALAE BRIDGES
SEISMIC RETROFIT
KULA, MAUI, HAWAII

Log of
Boring

P-1

Laboratory			Field				Depth (feet)	Sample	Graphic	USCS	(Continued from previous plate)
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)					Description
UC= 17250 psi			100	18							
			40	0			40			GP	Brownish gray SANDY GRAVEL with some cobbles, moist (clinker)
							45				Gray BASALT , severely fractured, unweathered to moderately weathered, hard (basalt formation) Boring terminated at 45 feet
							50				
							55				
							60				
							65				
							70				

Date Started: July 13, 2023

Date Completed: July 14, 2023

Logged By: N. Colter

Total Depth: 45 feet

Work Order: 8412-00(B)

Water Level: ▼ Not Encountered

Drill Rig: CME-45C TRUCK (Energy Transfer Ratio = 86.4%)

Drilling Method: 4" Solid-Stem Auger & HQ Coring

Driving Energy: 140 lb. wt., 30 in. drop

Plate

A - 3.2



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WAIALE, POHA KUOKALA, AND ALAE BRIDGES
SEISMIC RETROFIT
KULA, MAUI, HAWAII

Log of
Boring

P-2

Laboratory			Field				Depth (feet)	Sample	Graphic	USCS	Approximate Ground Surface Elevation (feet): 3280 *
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)					Description
Direct Shear	45	53			13					GP	5.5-inch ASPHALTIC CONCRETE
	35				12					ML	Gray SANDY GRAVEL , medium dense, moist (base material)
TXUU S _u =3.6 ksf	37	70			28	>4.5	5			ML	Grayish brown SANDY SILT , medium stiff, moist (fill)
											Gray SANDY SILT , stiff to very stiff, moist (residual soil)
UC= 12270 psi	12		100	42	50/3"		10			SM	Gray SILTY SAND with some gravel, very dense, moist (weathered basalt)
											Bluish gray BASALT , severely fractured, moderately weathered, hard (basalt formation)
UC= 25820 psi			100	33			15				grades to slightly fractured grades to slightly weathered
			92	58			20				
UC= 25820 psi			67	27			25				
			33				30			GP	Brownish gray SANDY GRAVEL with some cobbles, medium dense, moist (clinker)
							35				

Date Started: July 12, 2023

Date Completed: July 13, 2023

Logged By: N. Colter

Total Depth: 50.5 feet

Work Order: 8412-00(B)

Water Level: ▽ Not Encountered

Drill Rig: CME-45C TRUCK (Energy Transfer Ratio = 86.4%)

Drilling Method: 4" Solid-Stem Auger & PQ Coring

Driving Energy: 140 lb. wt., 30 in. drop

Plate

A - 4.1



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WAIKALE, POHAUOKALA, AND ALAE BRIDGES
SEISMIC RETROFIT
KULA, MAUI, HAWAII

Log of
Boring

P-2

Laboratory			Field				Depth (feet)	Sample	Graphic	USCS	(Continued from previous plate)
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)					Description
UC= 11100 psi	11		71	10	38					GP	
			25	12			40				Gray BASALT , severely to closely fractured, moderately weathered, hard (basalt formation)
	11		60	12	25		45			GP	Brownish gray SANDY GRAVEL with a little cobbles, medium dense, moist (clinker)
							50				Brownish gray BASALT , closely fractured, moderately weathered, hard (basalt formation) Boring terminated at 50.5 feet
							55				
							60				
							65				
							70				

Date Started: July 12, 2023

Date Completed: July 13, 2023

Logged By: N. Colter

Total Depth: 50.5 feet

Work Order: 8412-00(B)

Water Level: Not Encountered

Drill Rig: CME-45C TRUCK (Energy Transfer Ratio = 86.4%)

Drilling Method: 4" Solid-Stem Auger & PQ Coring

Driving Energy: 140 lb. wt., 30 in. drop

Plate

A - 4.2



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WAIKALE, POHAUOKALA, AND ALAE BRIDGES
SEISMIC RETROFIT
KULA, MAUI, HAWAII

Log of
Boring

W-1

Laboratory			Field				Depth (feet)	Sample	Graphic	USCS	Approximate Ground Surface Elevation (feet): 2760 *
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)					Description
Sieve - #200 = 21.6%	39	64			16					GM	5-inch ASPHALTIC CONCRETE
	34				9					SM	Brownish gray SILTY GRAVEL with some sand, moist (base material)
											Brown with multi-color mottling SILTY SAND with some gravel, loose, moist (fill)
TXUU S _u =2.7 ksf	44	76			21	4.0	5			MH	Dark brownish gray CLAYEY SILT with a little sand, and gravel stiff, moist (residual soil)
	52				3		10				grades to medium stiff
LL=75 PI=26 TXUU S _u =5.2 ksf	62	65			19	3.8	15				grades to stiff
	14				50/4"		20				grades with some cobbles
			95	0							Gray with multi-color mottling BASALT , severely fractured, highly weathered, medium hard to hard (basalt formation)
UC= 19770 psi UC= 7770 psi			100	48			25				Gray BASALT , closely to moderately fractured, slightly weathered, very hard (basalt formation)
UC= 22250 psi			100	53			30				grades to slightly fractured
							35				

Date Started: July 10, 2023

Date Completed: July 10, 2023

Logged By: B. Aiu

Total Depth: 50 feet

Work Order: 8412-00(B)

Water Level: ▼ Not Encountered

Drill Rig: CME-45C TRUCK (Energy Transfer Ratio = 86.4%)

Drilling Method: 4" Solid-Stem Auger & PQ Coring

Driving Energy: 140 lb. wt., 30 in. drop

Plate

A - 5.1



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WAIKALE, POHAUOKALA, AND ALAE BRIDGES
SEISMIC RETROFIT
KULA, MAUI, HAWAII

Log of
Boring

W-1

Laboratory			Field				Depth (feet)	Sample	Graphic	USCS	(Continued from previous plate)
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)					Description
UC= 21850 psi			100	83							
			100	97			40				
UC= 10240 psi			100	100			45				
							50				Boring terminated at 50 feet
							55				
							60				
							65				
							70				

Date Started: July 10, 2023

Date Completed: July 10, 2023

Logged By: B. Aiu

Total Depth: 50 feet

Work Order: 8412-00(B)

Water Level: ▼ Not Encountered

Drill Rig: CME-45C TRUCK (Energy Transfer Ratio = 86.4%)

Drilling Method: 4" Solid-Stem Auger & PQ Coring

Driving Energy: 140 lb. wt., 30 in. drop

Plate

A - 5.2



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WAIALE, POHA KUOKALA, AND ALAE BRIDGES
SEISMIC RETROFIT
KULA, MAUI, HAWAII

Log of
Boring

W-2

Laboratory			Field				Depth (feet)	Sample	Graphic	USCS	Approximate Ground Surface Elevation (feet): 2760 *
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)					Description
Direct Shear	19	94			27					GM	4-inch ASPHALTIC CONCRETE
	22				13					ML	Brownish gray SILTY GRAVEL with some sand, moist (base material)
	20	63			14		5			ML	Brown with multi-color mottling SANDY SILT with some sand and gravel, medium stiff, moist (fill)
Sieve - #200 = 61.9%	28				18		10				grades to very stiff
	7	73			17		15				grades to stiff
	7				32		20			GM	Brownish gray SILTY GRAVEL with a little sand, medium dense, moist (weathered basalt)
UC= 6430 psi			83	0			25				Gray BASALT , closely fractured, moderately weathered, hard (basalt formation)
			75	17			30				
			62	0			35			GP	Brownish gray SANDY GRAVEL with some cobbles, moist (clinker)

Date Started: July 11, 2023

Date Completed: July 11, 2023

Logged By: B. Aiu

Total Depth: 51 feet

Work Order: 8412-00(B)

Water Level:  Not Encountered

Drill Rig: CME-45C TRUCK (Energy Transfer Ratio = 86.4%)

Drilling Method: 4" Solid-Stem Auger & PQ Coring

Driving Energy: 140 lb. wt., 30 in. drop

Plate

A - 6.1



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WAIKALE, POHAUOKALA, AND ALAE BRIDGES
SEISMIC RETROFIT
KULA, MAUI, HAWAII

Log of
Boring

W-2

Laboratory			Field				Depth (feet)	Sample	Graphic	USCS	(Continued from previous plate)
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)					Description
	5		75	7	10/0" Ref.					GP	
			58	7			40				Brownish gray BASALT , closely fractured, moderately weathered, hard (basalt formation)
			10				45				Brownish gray SANDY GRAVEL with some cobbles, dense to very dense, moist (clinker)
							50				Boring terminated at 51 feet
							55				
							60				
							65				
							70				

Date Started: July 11, 2023

Date Completed: July 11, 2023

Logged By: B. Aiu

Total Depth: 51 feet

Work Order: 8412-00(B)

Water Level: ▼ Not Encountered

Drill Rig: CME-45C TRUCK (Energy Transfer Ratio = 86.4%)

Drilling Method: 4" Solid-Stem Auger & PQ Coring

Driving Energy: 140 lb. wt., 30 in. drop

Plate

A - 6.2

APPENDIX B

APPENDIX B

Laboratory Tests

Moisture Content (ASTM D2216) and Unit Weight determinations (ASTM D2937) were performed on selected soil samples as an aid in the classification and evaluation of soil properties. The test results are presented on the Logs of Borings at the appropriate sample depths.

Two 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 B-1.

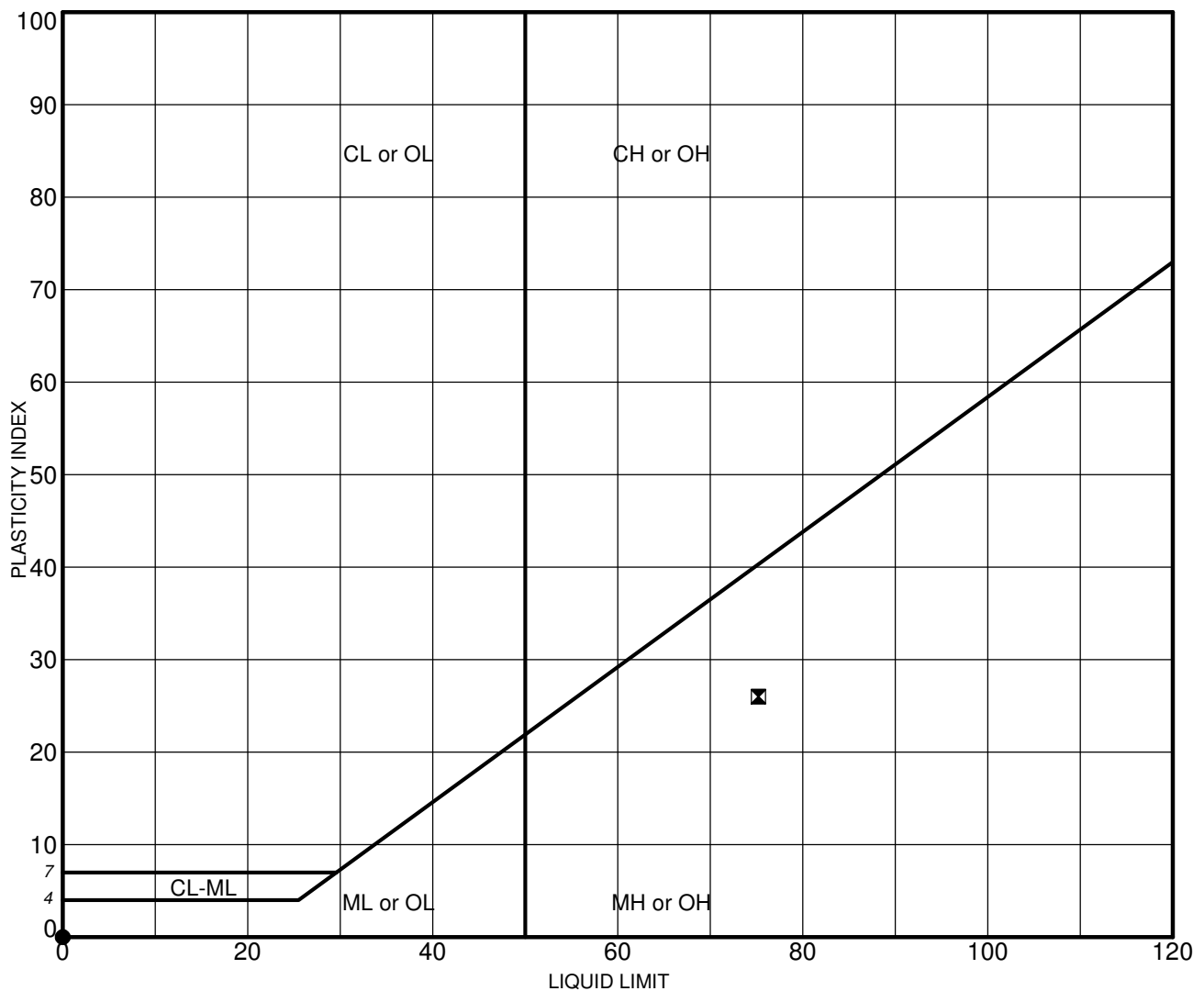
Fifteen Sieve Analysis tests (ASTM D6913) were performed on selected samples of the soils to evaluate the gradation characteristics of the soils and to aid in soil classification. Graphic representations of the grain size distributions are provided on Plates B-2 through B-4.

Nineteen Uniaxial Compressive Strength tests (ASTM D7012, Method C) were performed on selected rock core samples to evaluate the unconfined compressive strength of the volcanic tuff formation encountered. The test results are presented on Plate B-5.

Six Direct Shear tests (ASTM D3080) were performed on selected soil samples to evaluate the shear strength characteristics of the material tested. The test results are presented on Plates B-6 through B-11.

Six Unconsolidated Undrained Triaxial Compression (TXUU) tests were performed on selected relatively undisturbed soil samples in accordance with ASTM D2850. The approximate in-situ effective overburden pressure was used as the applied confining pressure for the “undisturbed” samples. The test results and the stress-strain curve are presented on Plates B-12 through B-17.

Six laboratory Resistance (R) Value tests (ASTM D2844) were performed by Ninyo & Moore on bulk samples of the near-surface soils to evaluate the pavement support characteristics of the soils. The test results are presented on Plates B-18 through B-23.



	Sample	Depth (ft)	LL	PL	PI	Description
●	A-2	5.0-6.5	NP	NP	NP	Brown with black mottling silty sand with a little gravel
⊠	W-1	15.0-16.5	75	49	26	Dark brownish gray clayey silt (MH) with a little sand and gravel

NP = NON-PLASTIC

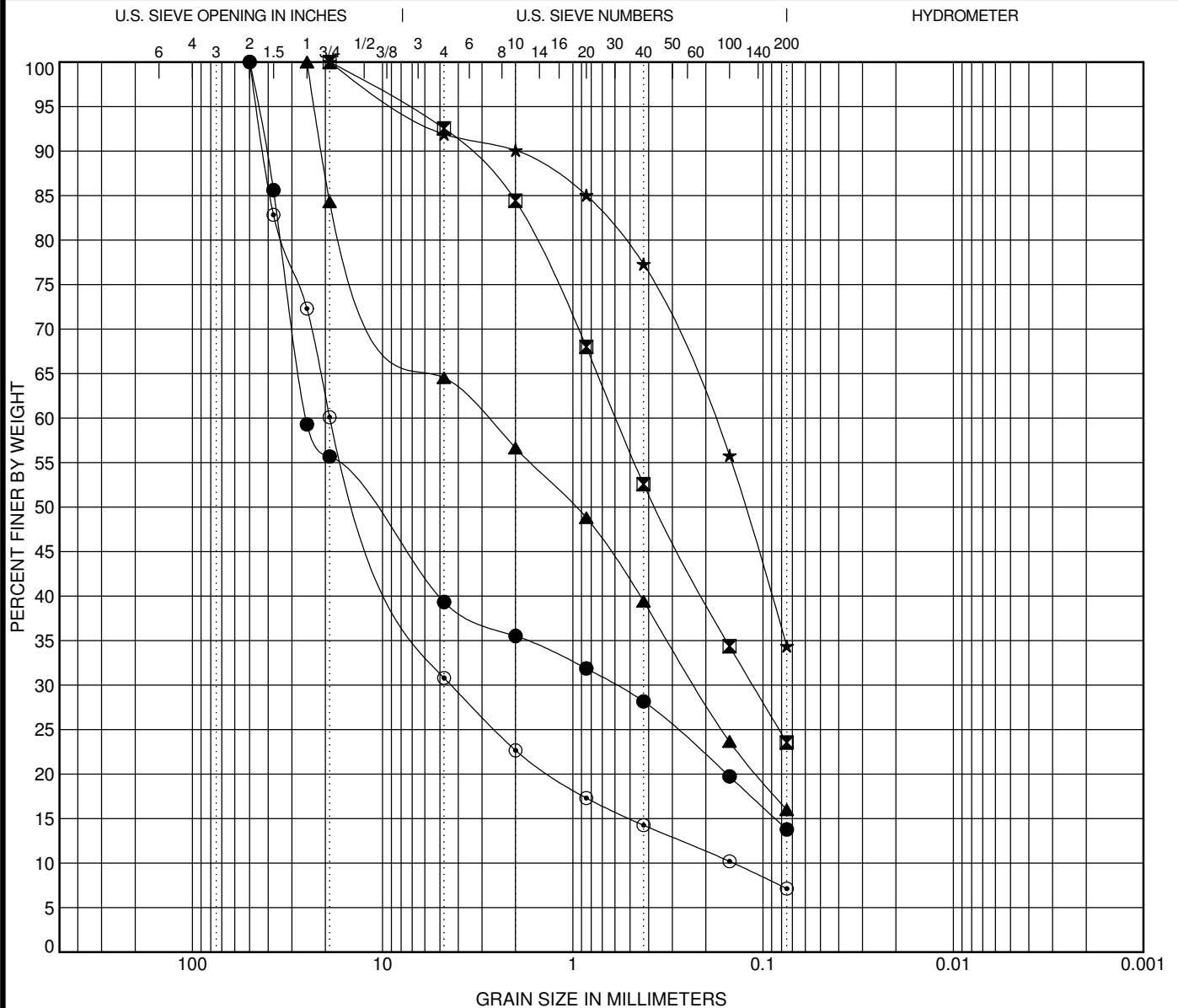


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ATTERBERG LIMITS TEST RESULTS - ASTM D4318

WAI'ALE, POHA'KUOKALA, AND ALAE BRIDGES
 SEISMIC RETROFIT
 KULA, MAUI, HAWAII

Plate
B - 1



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Sample	Depth (ft)	Description						LL	PL	PI	Cc	Cu	
●	A-1	1.0-2.5	Brown with black mottling silty gravel (GM) with some sand										
☒	A-1	31.5-33.0	Brown silty sand (SM) with a little gravel										
▲	A-2	2.5-4.0	Brownish gray with black mottling silty sand (SM) with some gravel										
★	A-2	10.0-11.5	Brownish gray with black mottling silty sand (SM) with a little gravel										
◎	A-2	25.0-26.5	Gray with brown mottling sandy gravel (GP-GM) with a little silt									7.0	131.7
	Sample	Depth (ft)	D100 (mm)	D60 (mm)	D30 (mm)	D10 (mm)	%Gravel	%Sand	%Fine				
●	A-1	1.0-2.5	50	25.274	0.6		60.7	25.6	13.8				
☒	A-1	31.5-33.0	19	0.594	0.114		7.5	69.0	23.5				
▲	A-2	2.5-4.0	25	2.895	0.228		35.5	48.5	16.0				
★	A-2	10.0-11.5	19	0.184			8.1	57.5	34.4				
◎	A-2	25.0-26.5	50	18.921	4.364	0.144	69.2	23.7	7.1				



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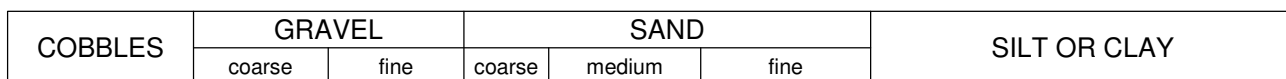
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GRAIN SIZE DISTRIBUTION - ASTM D6913

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SEISMIC RETROFIT
KULA, MAUI, HAWAII

Plate
B - 2



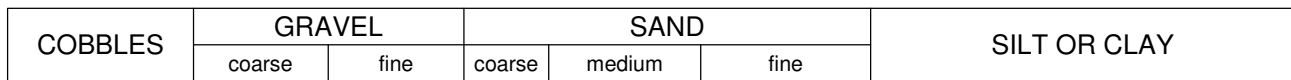
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W.O. 8412-00(B)

WAIALE, POHAUOKALA, AND ALAE BRIDGES
SEISMIC RETROFIT
KULA, MAUI, HAWAII

Plate
B - 3



W.O. 8412-00(B)

WAIALE, POHAUOKALA, AND ALAE BRIDGES
SEISMIC RETROFIT
KULA, MAUI, HAWAII

Plate
B - 4

Location	Depth	Length	Diameter	Length/ Diameter Ratio	Density	Load	Compressive Strength
	(feet)	(inches)	(inches)		(pcf)	(lbs)	(psi)
A-1	45 - 45.4	4.760	2.380	2.00	168.2	41,135	9,250
A-1	50 - 50.5	4.800	2.400	2.00	164.5	92,710	20,490
A-2	29 - 29.5	6.480	3.240	2.00	169.4	124,140	15,060
A-2	31.5 - 32	6.540	3.270	2.00	166.8	106,345	12,660
A-2	44 - 44.5	6.520	3.260	2.00	168.4	136,745	16,380
P-1	17 - 17.4	4.800	2.400	2.00	170.3	46,185	10,210
P-1	23 - 23.4	4.840	2.420	2.00	166.1	82,815	18,000
P-1	27 - 27.4	4.800	2.400	2.00	164.7	83,065	18,360
P-1	31 - 31.4	4.800	2.400	2.00	170.4	35,595	7,870
P-1	35.5 - 36	4.800	2.400	2.00	170.5	78,055	17,250
P-2	16.5 - 17	6.560	3.260	2.01	172.5	102,415	12,270
P-2	24.75 - 25.3	6.580	3.290	2.00	170.0	219,520	25,820
P-2	45.5 - 46.05	6.540	3.270	2.00	144.6	93,200	11,100
W-1	26 - 26.5	6.580	3.290	2.00	153.2	168,050	19,770
W-1	26.5 - 26.75	6.620	3.310	2.00	160.9	66,855	7,770
W-1	32.5 - 33	6.560	3.280	2.00	167.7	187,980	22,250
W-1	36.5 - 37	6.620	3.310	2.00	169.4	187,980	21,850
W-1	45.5 - 46	6.580	3.290	2.00	168.8	87,020	10,240
W-2	26 - 26.5	6.500	3.280	1.98	153.8	54,340	6,430

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 specimen that meets the requirements of Practice D4543

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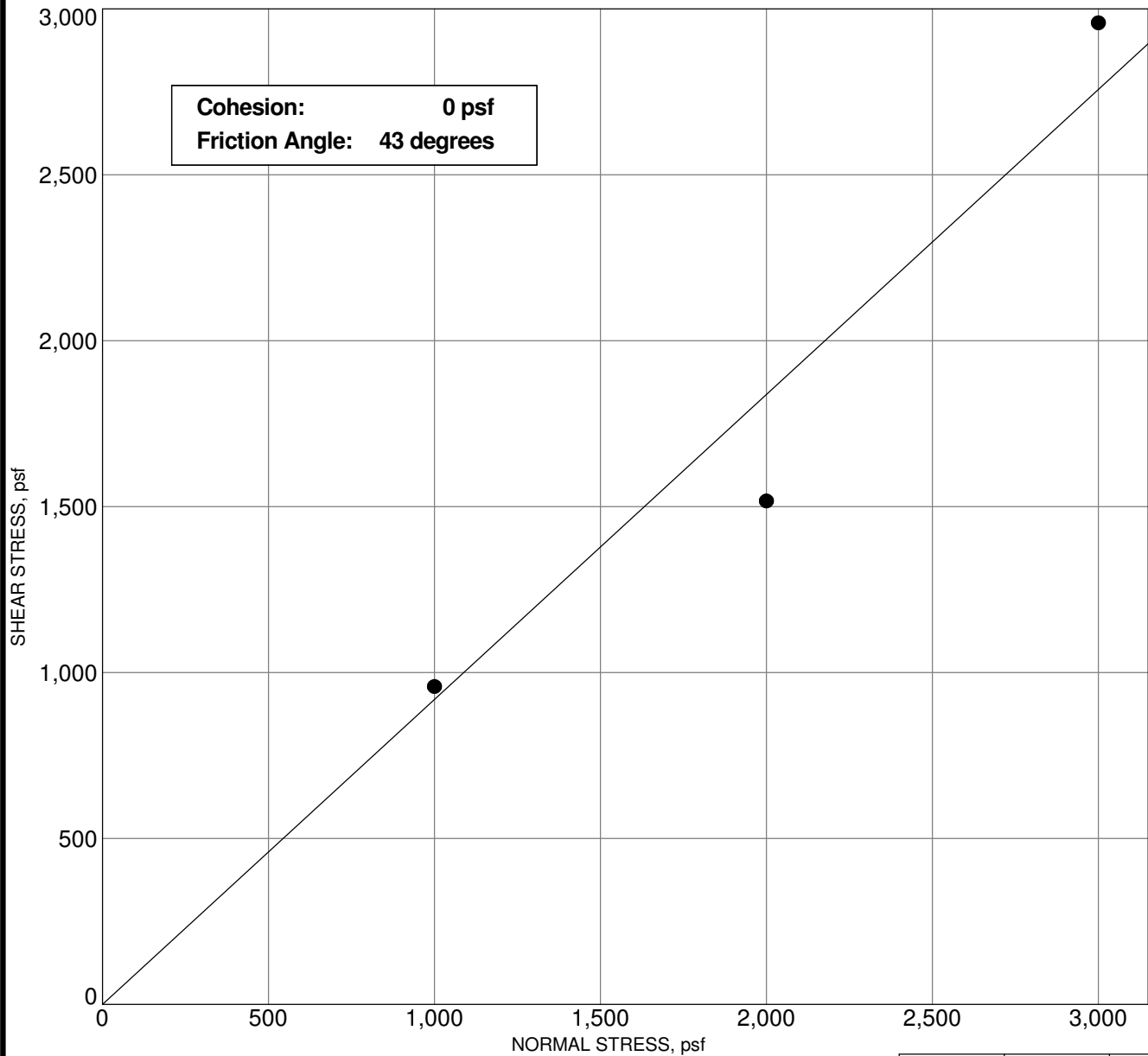
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UNIAXIAL COMPRESSIVE STRENGTH TEST

WAIKALE, POHA KUOKALA, AND ALAE BRIDGES
SEISMIC RETROFIT
KULA, MAUI, HAWAII

Plate
B - 5



Sample: A-1
Depth: 5.0 - 6.5 feet
Description: Brown with black mottling silty gravel with some sand

		Sample #1	Sample #2	Sample #3
INITIAL	Moisture Content, %	18.0	21.3	20.3
	Dry Density, pcf	61.6	72.7	70.9
	Height, inches	1.00	1.00	1.00
FINAL	Moisture Content, %	34.3	35.9	32.9
	Dry Density, pcf	59.4	74.8	72.0
	Height, inches	1.038	0.973	0.985
	Diameter, inches	2.42	2.42	2.42
	Deformation Rate, inch/minute	0.0024	0.0024	0.0024
	Normal Stress, psf	1000	2000	3000
	Peak Shear Stress, psf	958	1517	2958
	Shear Displacement, inches	0.43	0.41	0.42

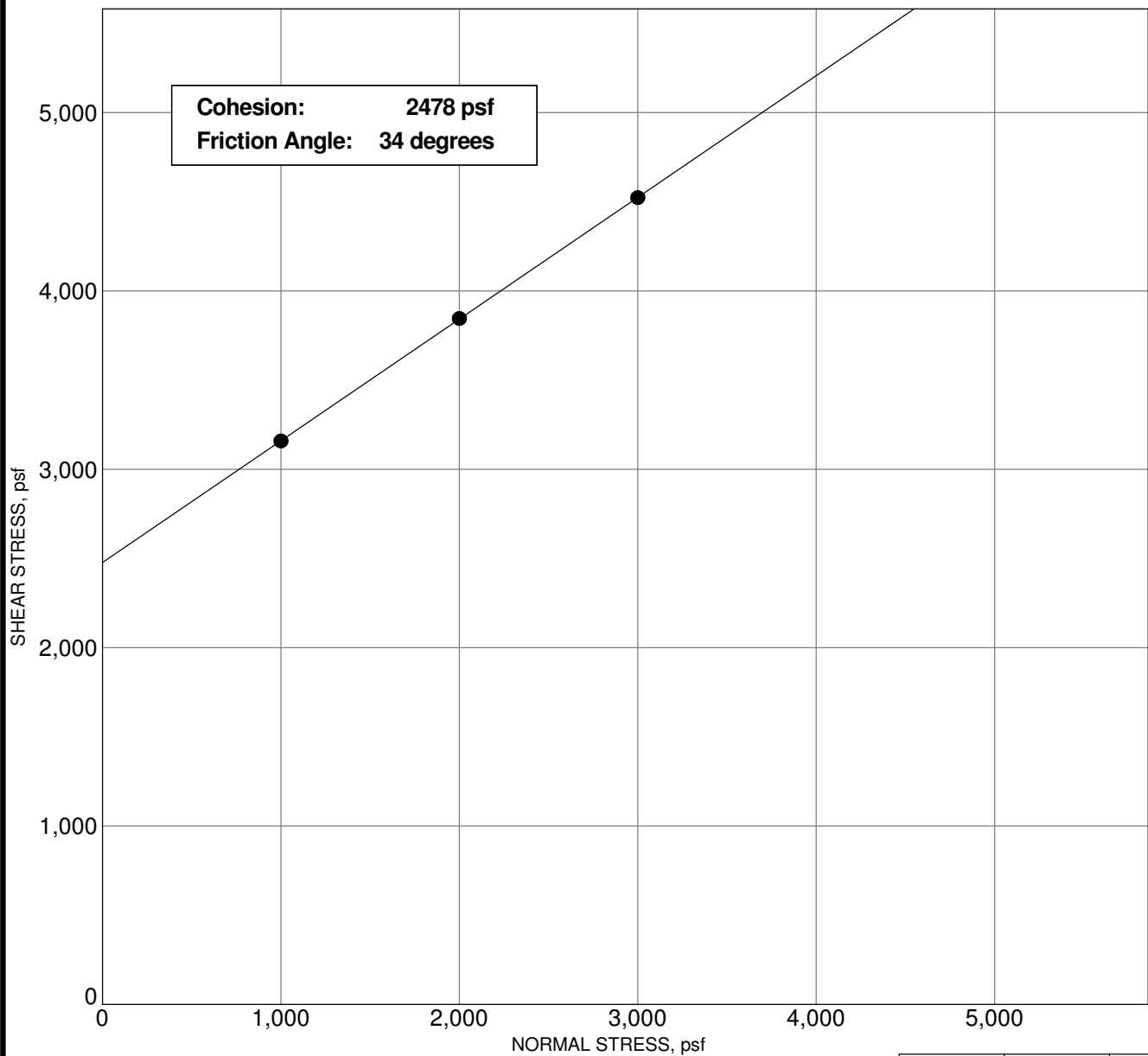


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DIRECT SHEAR TEST - ASTM D3080

WAIKALE, POHA KUOKALA, AND ALAE BRIDGES
SEISMIC RETROFIT
KULA, MAUI, HAWAII

Plate
B - 6



Sample: A-2
 Depth: 1.0 - 2.5 feet
 Description: Brownish gray with black mottling silty sand with some gravel

		Sample #1	Sample #2	Sample #3
INITIAL	Moisture Content, %	9.1	9.0	9.5
	Dry Density, pcf	84.3	87.4	82.1
	Height, inches	1.00	1.00	1.00
FINAL	Moisture Content, %	23.3	19.3	21.9
	Dry Density, pcf	85.7	86.6	84.3
	Height, inches	0.983	1.009	0.974
Diameter, inches		2.42	2.42	2.42
Deformation Rate, inch/minute		0.0025	0.0020	0.0026
Normal Stress, psf		1000	2000	3000
Peak Shear Stress, psf		3158	3845	4523
Shear Displacement, inches		0.43	0.38	0.42

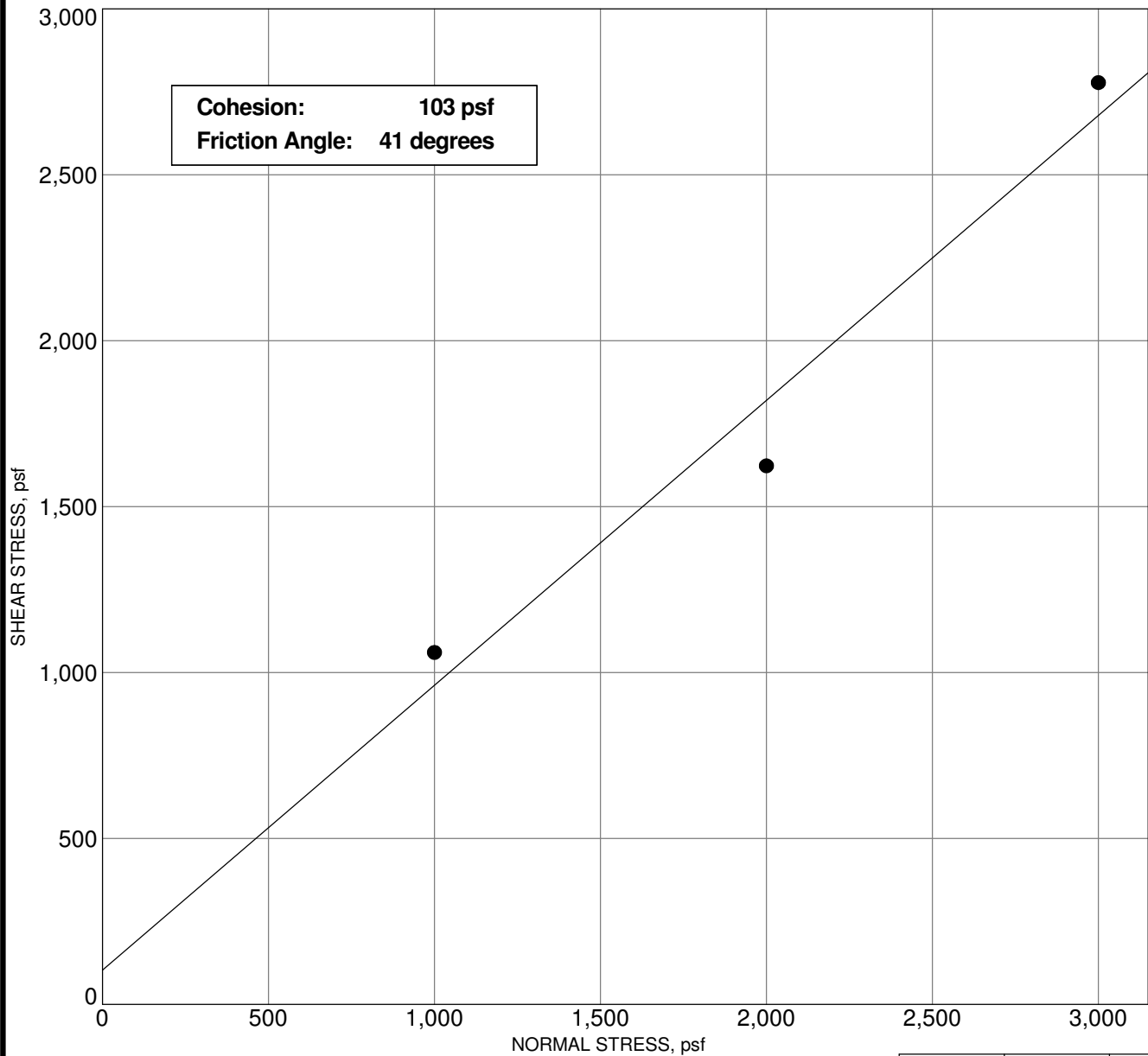


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DIRECT SHEAR TEST - ASTM D3080

WAIKALE, POHA KUOKALA, AND ALAE BRIDGES
 SEISMIC RETROFIT
 KULA, MAUI, HAWAII

Plate
B - 7



Sample: P-1
Depth: 1.0 - 2.5 feet
Description: Brown silty gravel with some sand

		Sample #1	Sample #2	Sample #3
INITIAL	Moisture Content, %	40.7	37.4	36.0
	Dry Density, pcf	40.5	43.4	43.0
	Height, inches	1.00	1.00	1.00
FINAL	Moisture Content, %	77.9	71.4	70.1
	Dry Density, pcf	39.5	45.1	44.8
	Height, inches	1.024	0.963	0.960
	Diameter, inches	2.42	2.42	2.42
	Deformation Rate, inch/minute	0.0024	0.0021	0.0022
	Normal Stress, psf	1000	2000	3000
	Peak Shear Stress, psf	1061	1623	2778
	Shear Displacement, inches	0.43	0.41	0.41



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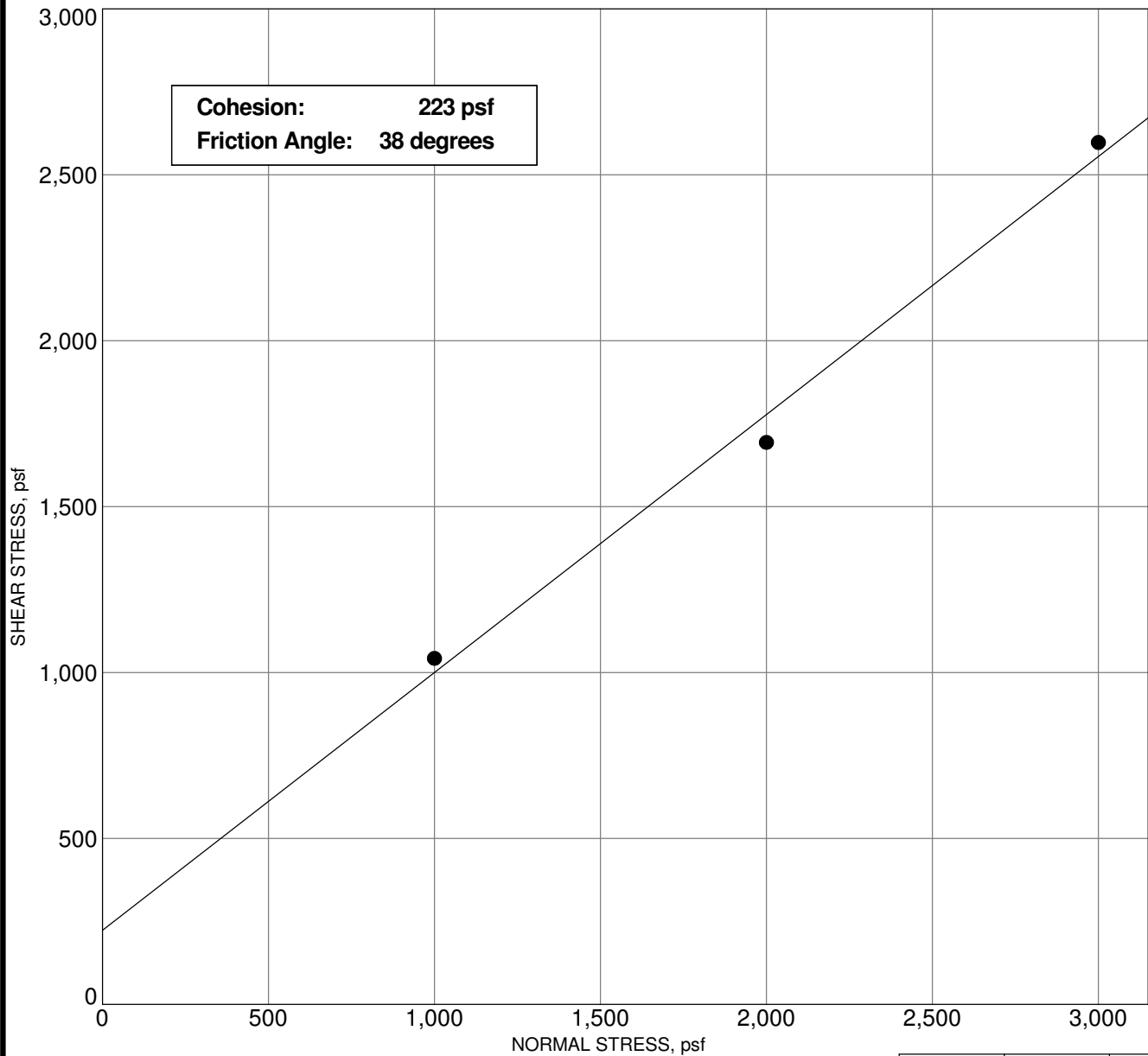
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W.O. 8412-00(B)

DIRECT SHEAR TEST - ASTM D3080

WAIKALE, POHA KUOKALA, AND ALAE BRIDGES
SEISMIC RETROFIT
KULA, MAUI, HAWAII

Plate
B - 8



Sample: P-1
Depth: 5.0 - 6.5 feet
Description: Brown silty gravel with some sand

		Sample #1	Sample #2	Sample #3
INITIAL	Moisture Content, %	17.6	25.9	38.7
	Dry Density, pcf	59.8	57.2	54.6
	Height, inches	1.00	1.00	1.00
FINAL	Moisture Content, %	54.2	53.5	57.5
	Dry Density, pcf	58.7	59.0	57.4
	Height, inches	1.019	0.970	0.951
	Diameter, inches	2.42	2.42	2.42
	Deformation Rate, inch/minute	0.0024	0.0020	0.0023
	Normal Stress, psf	1000	2000	3000
	Peak Shear Stress, psf	1043	1693	2597
	Shear Displacement, inches	0.43	0.40	0.41

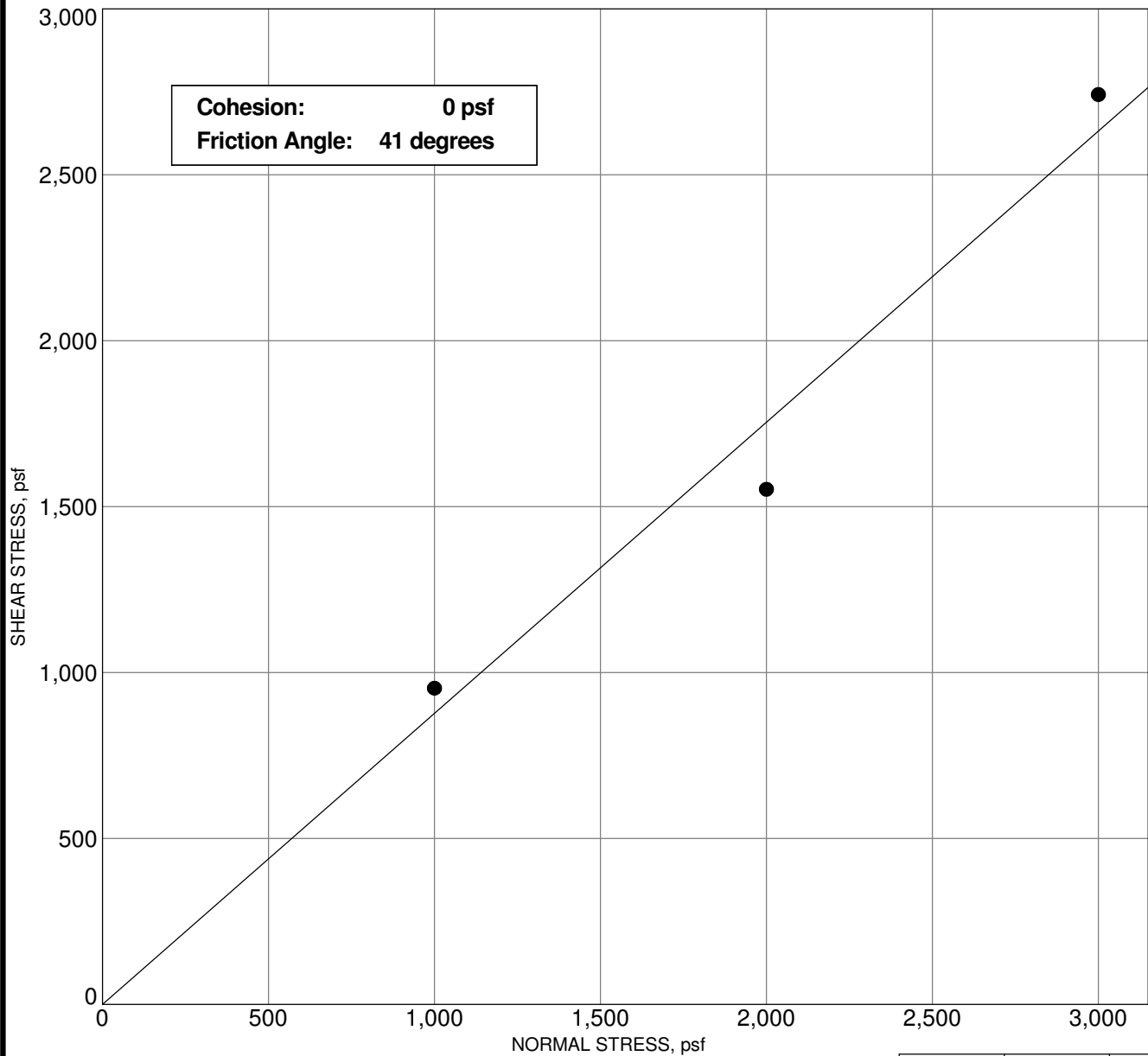


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DIRECT SHEAR TEST - ASTM D3080

WAIKALE, POHAUOKALA, AND ALAE BRIDGES
SEISMIC RETROFIT
KULA, MAUI, HAWAII

Plate
B - 9



Sample: P-2
Depth: 1.0 - 2.5 feet
Description: Gray sandy silt with a little gravel

		Sample #1	Sample #2	Sample #3
INITIAL	Moisture Content, %	44.7	40.7	45.3
	Dry Density, pcf	53.2	54.9	53.8
	Height, inches	1.00	1.00	1.00
FINAL	Moisture Content, %	80.4	72.9	71.6
	Dry Density, pcf	53.0	57.4	56.4
	Height, inches	1.002	0.957	0.953
	Diameter, inches	2.42	2.42	2.42
	Deformation Rate, inch/minute	0.0024	0.0022	0.0023
	Normal Stress, psf	1000	2000	3000
	Peak Shear Stress, psf	953	1552	2742
	Shear Displacement, inches	0.43	0.42	0.41

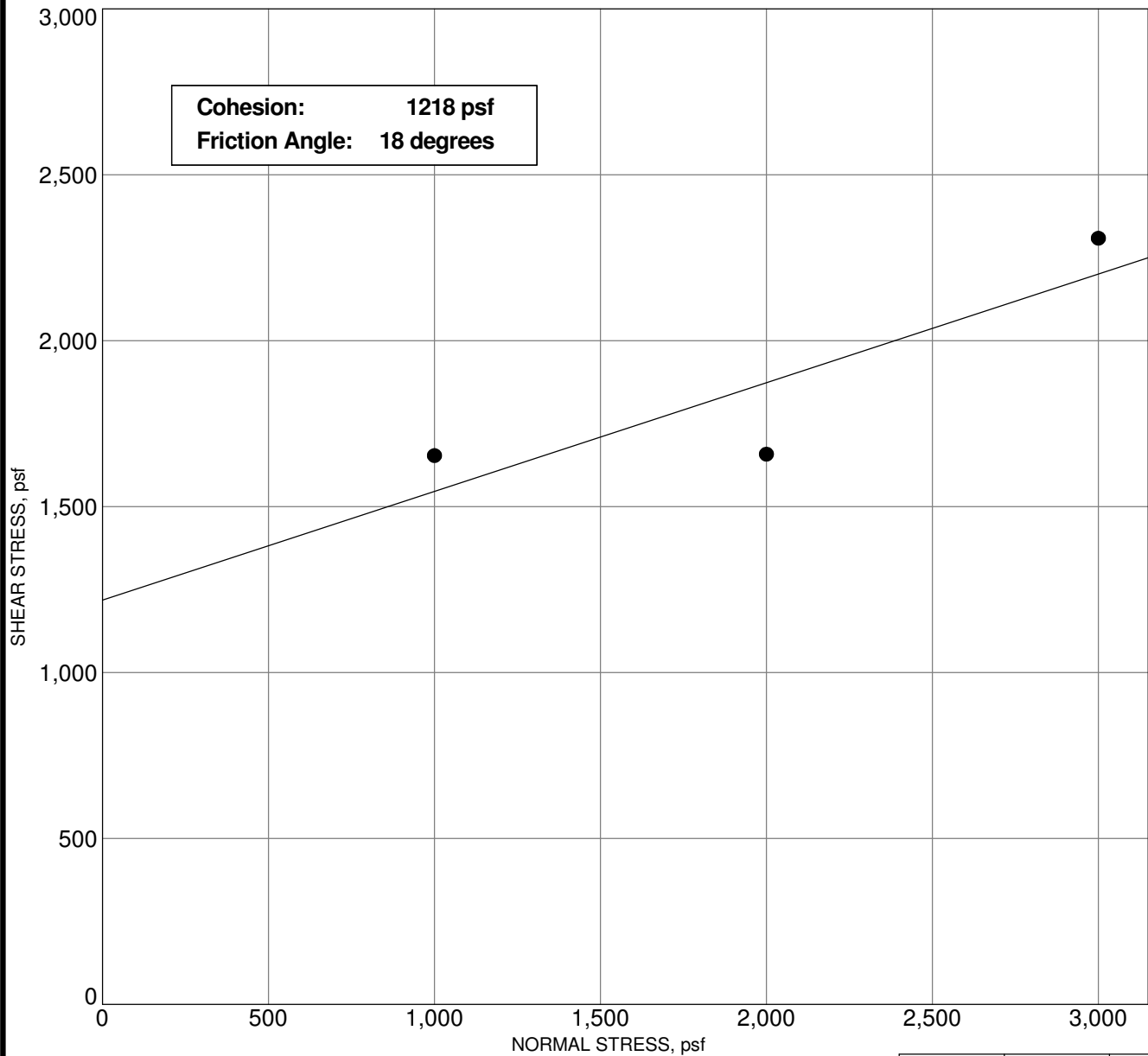


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GEOTECHNICAL ENGINEERING
W.O. 8412-00(B)

DIRECT SHEAR TEST - ASTM D3080

WAIKALE, POHAUOKALA, AND ALAE BRIDGES
SEISMIC RETROFIT
KULA, MAUI, HAWAII

Plate
B - 10



Sample: W-2
 Depth: 5.0 - 6.5 feet
 Description: Dark brownish gray sandy silt with traces of gravel

		Sample #1	Sample #2	Sample #3
INITIAL	Moisture Content, %	20.2	21.5	22.8
	Dry Density, pcf	63.1	65.6	65.4
	Height, inches	1.00	1.00	1.00
FINAL	Moisture Content, %	51.2	50.4	29.5
	Dry Density, pcf	61.8	67.0	68.6
	Height, inches	1.021	0.978	0.954
	Diameter, inches	2.42	2.42	2.42
	Deformation Rate, inch/minute	0.0024	0.0022	0.0023
	Normal Stress, psf	1000	2000	3000
	Peak Shear Stress, psf	1654	1658	2309
	Shear Displacement, inches	0.42	0.42	0.41

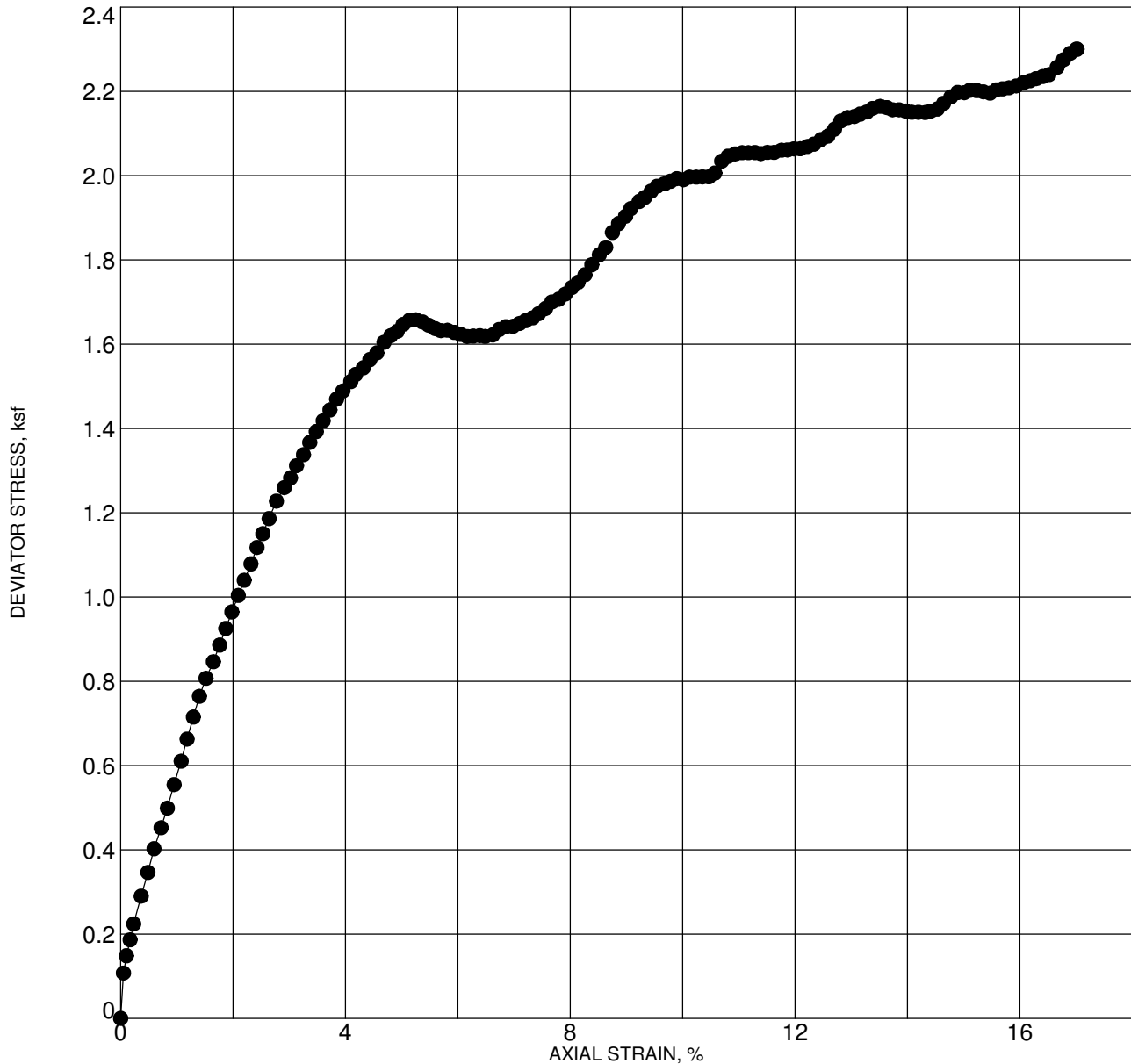


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DIRECT SHEAR TEST - ASTM D3080

WAIKALE, POHA KUOKALA, AND ALAE BRIDGES
 SEISMIC RETROFIT
 KULA, MAUI, HAWAII

Plate
B - 11



Max. Deviator Stress (ksf): 2.2

Confining Stress (ksf): 0.6

Location: A-2

Depth: 5.0 - 6.5 feet

Description: Brown with black mottling silty sand with a little gravel

Test Date: 8/23/2023

Dry Density (pcf)	57.2	Sample Diameter (inches)	2.413
Moisture (%)	48.3	Sample Height (inches)	5.067
Axial Strain at Failure (%)	14.9	Strain Rate (% / minute)	0.70



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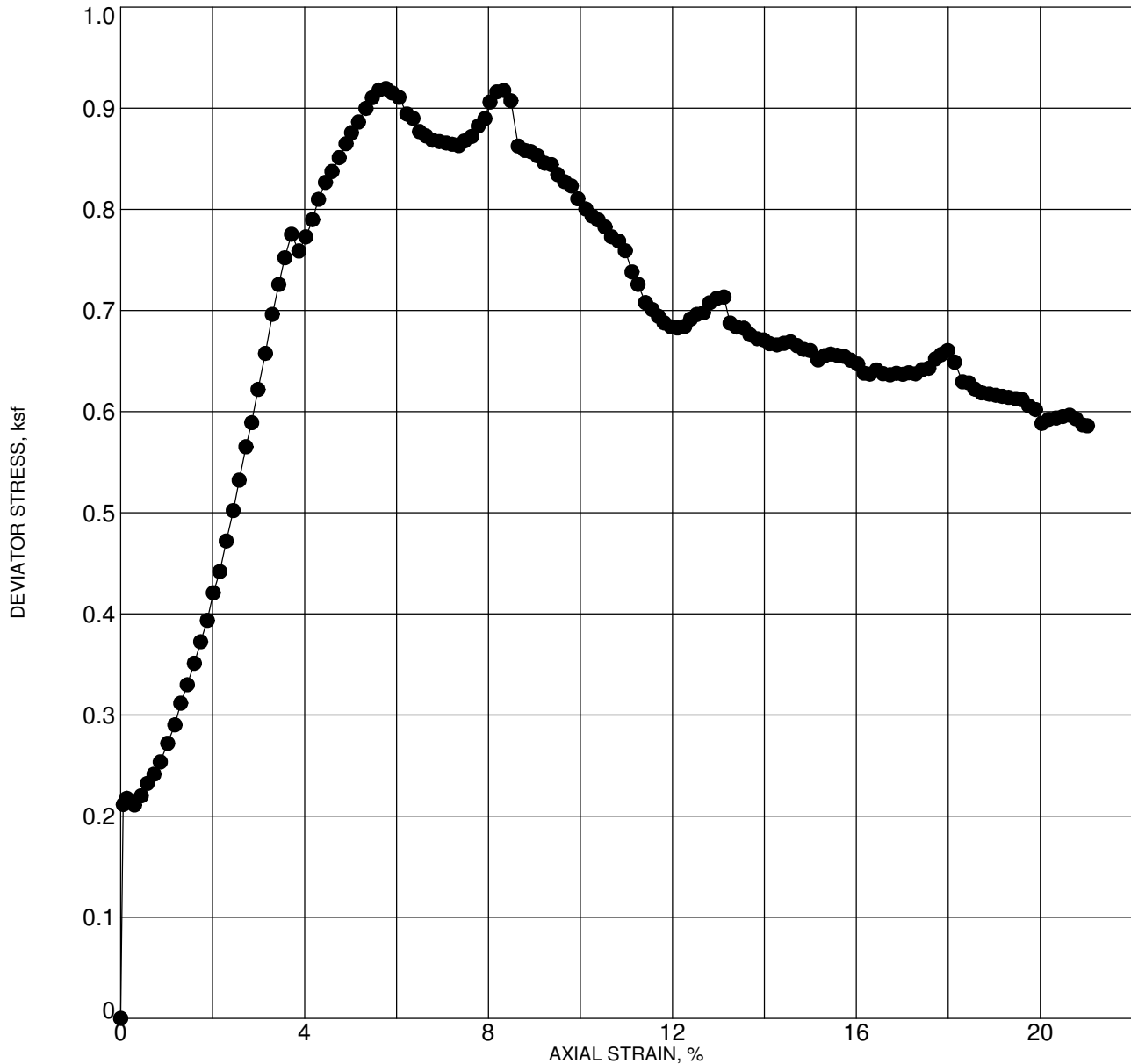
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W.O. 8412-00(B)

TRIAXIAL UU COMPRESSION TEST - ASTM D2850

WAI'ALE, POHA'KUOKALA, AND ALAE BRIDGES
SEISMIC RETROFIT
KULA, MAUI, HAWAII

Plate
B - 12



Max. Deviator Stress (ksf): 0.9

Confining Stress (ksf): 1.7

Location: A-2

Depth: 15.0 - 16.5 feet

Description: Brown with black mottling silty sand with a little gravel

Test Date: 8/23/2023

Dry Density (pcf)	48.2	Sample Diameter (inches)	2.413
Moisture (%)	50.6	Sample Height (inches)	5.067
Axial Strain at Failure (%)	8.3	Strain Rate (% / minute)	0.86



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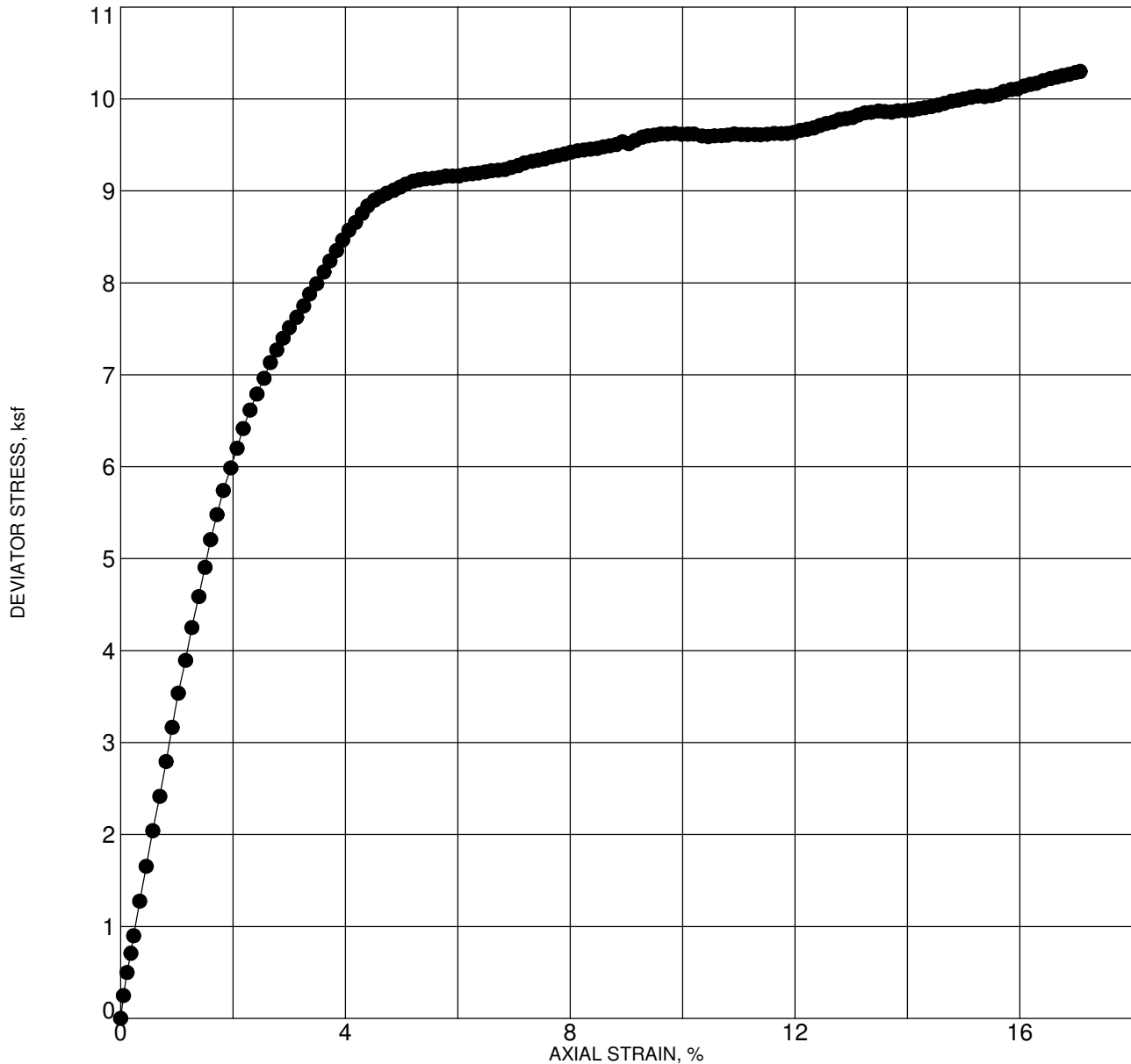
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W.O. 8412-00(B)

TRIAXIAL UU COMPRESSION TEST - ASTM D2850

WAIKALE, POHAUOKALA, AND ALAE BRIDGES
SEISMIC RETROFIT
KULA, MAUI, HAWAII

Plate
B - 13



Max. Deviator Stress (ksf): 10.0

Confining Stress (ksf): 2.2

Location: A-2

Depth: 20.0 - 21.5 feet

Description: Brown and gray sandy silt with some gravel

Test Date: 8/24/2023

Dry Density (pcf)	79.4	Sample Diameter (inches)	2.413
Moisture (%)	43.7	Sample Height (inches)	5.067
Axial Strain at Failure (%)	14.9	Strain Rate (% / minute)	0.71



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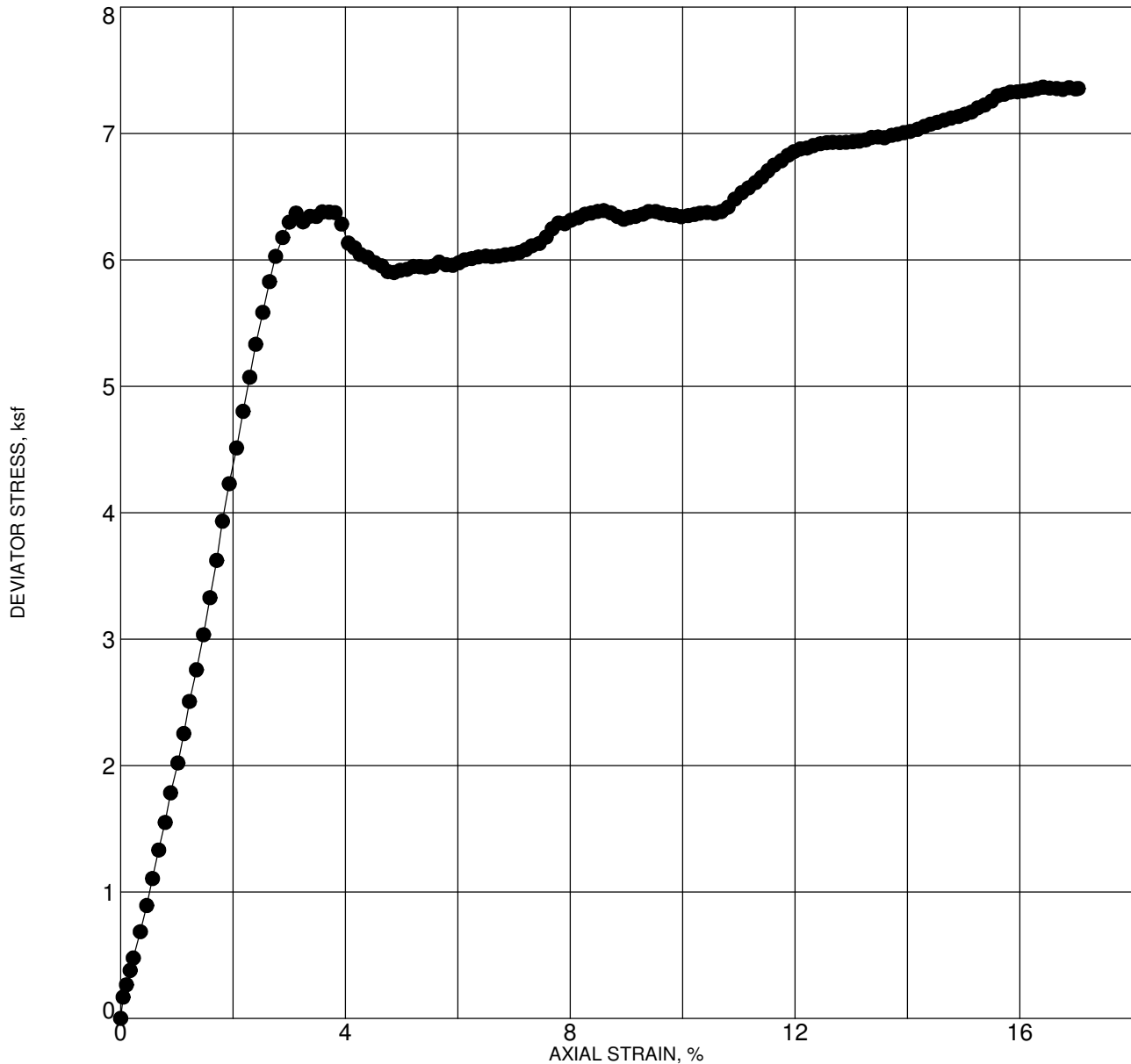
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TRIAXIAL UU COMPRESSION TEST - ASTM D2850

WAIALE, POHA KUOKALA, AND ALAE BRIDGES
SEISMIC RETROFIT
KULA, MAUI, HAWAII

Plate
B - 14



Max. Deviator Stress (ksf): 7.1

Confining Stress (ksf): 0.6

Location: P-2

Depth: 5.0 - 6.5 feet

Description: Gray sandy silt

Test Date: 8/24/2023

Dry Density (pcf)	69.9	Sample Diameter (inches)	2.413
Moisture (%)	37.1	Sample Height (inches)	5.067
Axial Strain at Failure (%)	14.9	Strain Rate (% / minute)	0.72



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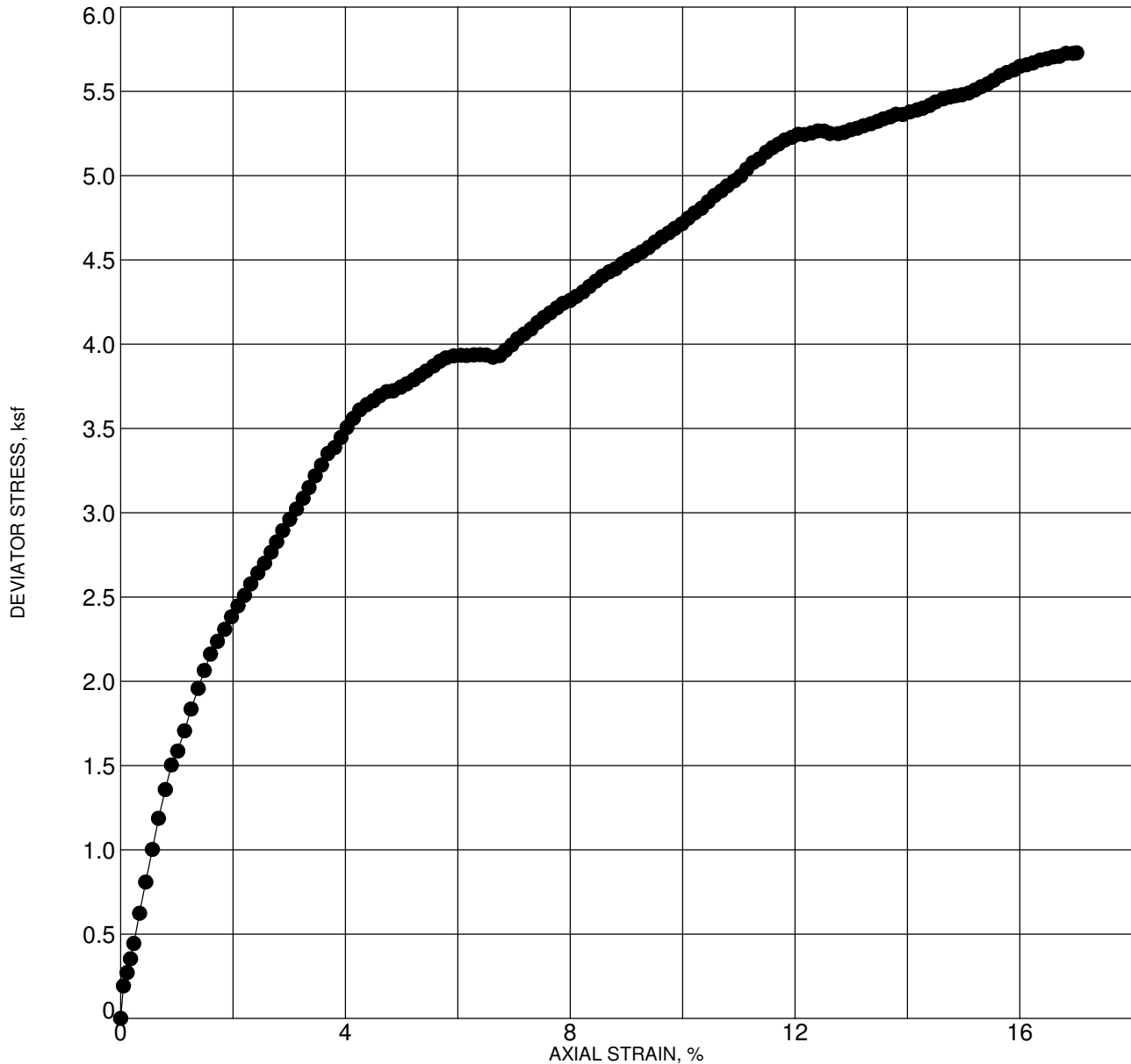
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TRIAXIAL UU COMPRESSION TEST - ASTM D2850

WAIALE, POHA KUOKALA, AND ALAE BRIDGES
SEISMIC RETROFIT
KULA, MAUI, HAWAII

Plate
B - 15



Max. Deviator Stress (ksf): 5.5

Confining Stress (ksf): 0.6

Location: W-1

Depth: 5.0 - 6.5 feet

Description: Dark brownish gray clayey silt with a little sand and gravel

Test Date: 8/23/2023

Dry Density (pcf)	76.2	Sample Diameter (inches)	2.413
Moisture (%)	43.7	Sample Height (inches)	5.067
Axial Strain at Failure (%)	15.0	Strain Rate (% / minute)	0.70



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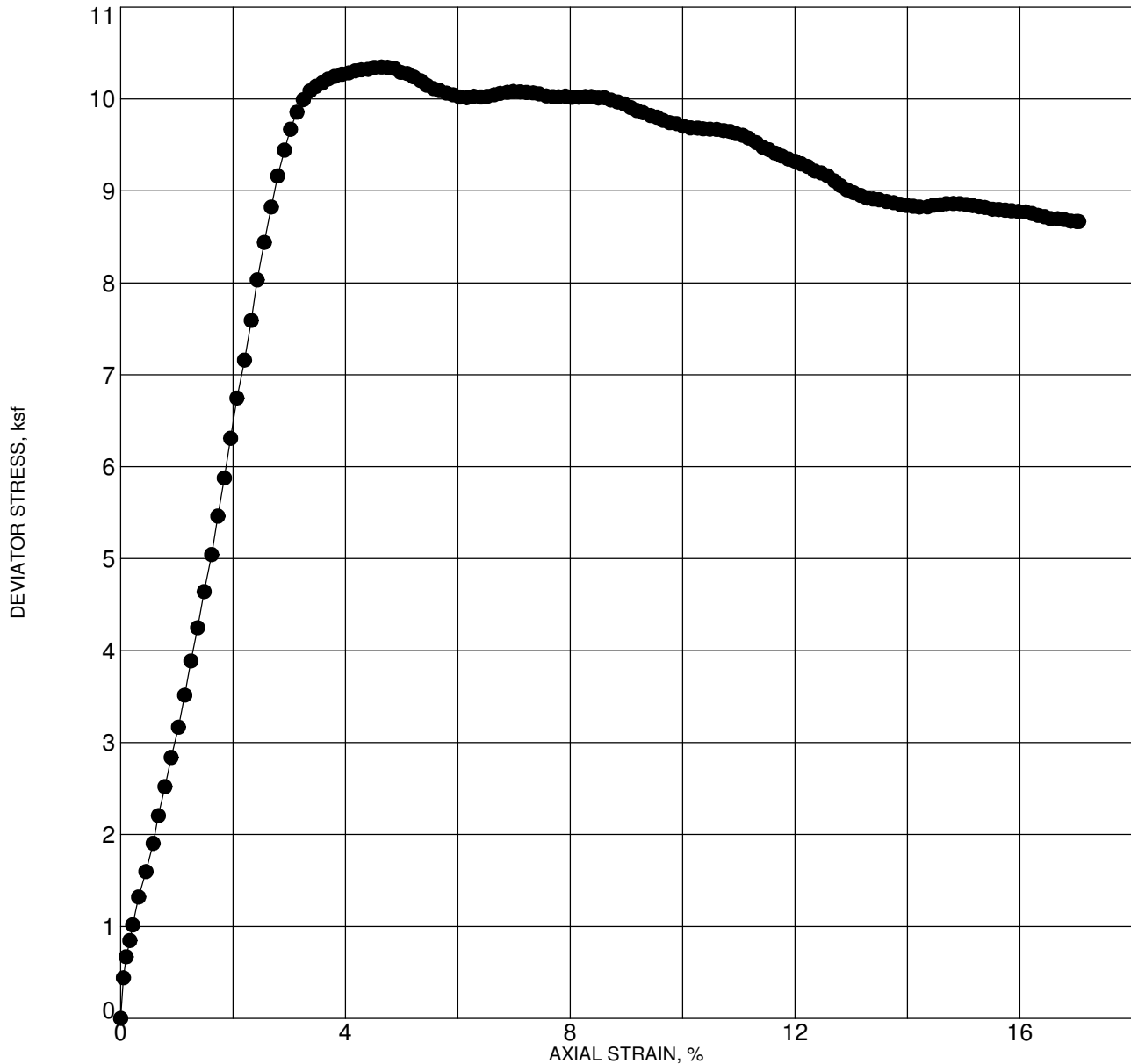
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TRIAXIAL UU COMPRESSION TEST - ASTM D2850

WAI'ALE, POHA'KUOKALA, AND ALAE BRIDGES
SEISMIC RETROFIT
KULA, MAUI, HAWAII

Plate
B - 16



Max. Deviator Stress (ksf): 10.3

Confining Stress (ksf): 1.7

Location: W-1

Depth: 15.0 - 16.5 feet

Description: Dark brownish gray clayey silt (MH) with a little sand and gravel

Test Date: 8/23/2023

Dry Density (pcf)	64.7	Sample Diameter (inches)	2.413
Moisture (%)	61.8	Sample Height (inches)	5.067
Axial Strain at Failure (%)	4.6	Strain Rate (% / minute)	0.70



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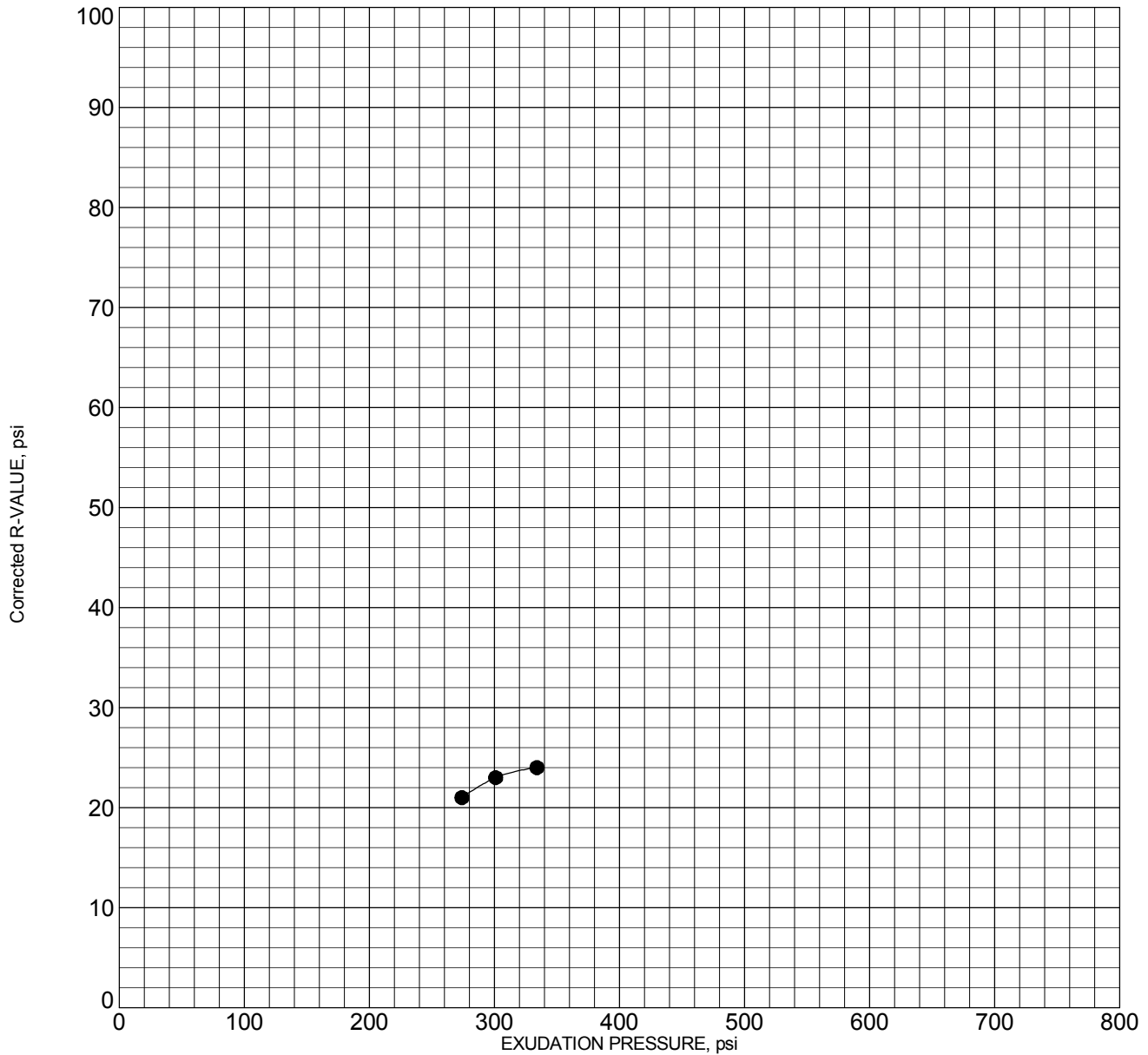
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TRIAXIAL UU COMPRESSION TEST - ASTM D2850

WAI'ALE, POHA KUOKALA, AND ALAE BRIDGES
SEISMIC RETROFIT
KULA, MAUI, HAWAII

Plate
B - 17



Sample: Bulk-5

Depth: 1.0 - 5.0 feet

R-Value at 300 psi Exudation Pressure:	23
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Description: Brown silty sand (SM) with traces of gravel

R-Value Test Performed by Ninyo & Moore

No.	Compaction Pressure	Density	Moisture Content	Horizontal Pressure @160 psi	Sample Height	Exudation Pressure	R-Value	Corrected R-Value
	(psi)	(pcf)	(%)	(psi)	(in)	(psi)		
1	100	88.7	35.9	97	2.55	334	24	24
2	100	85.6	36.4	100	2.56	301	23	23
3	100	84.1	36.9	103	2.55	274	21	21



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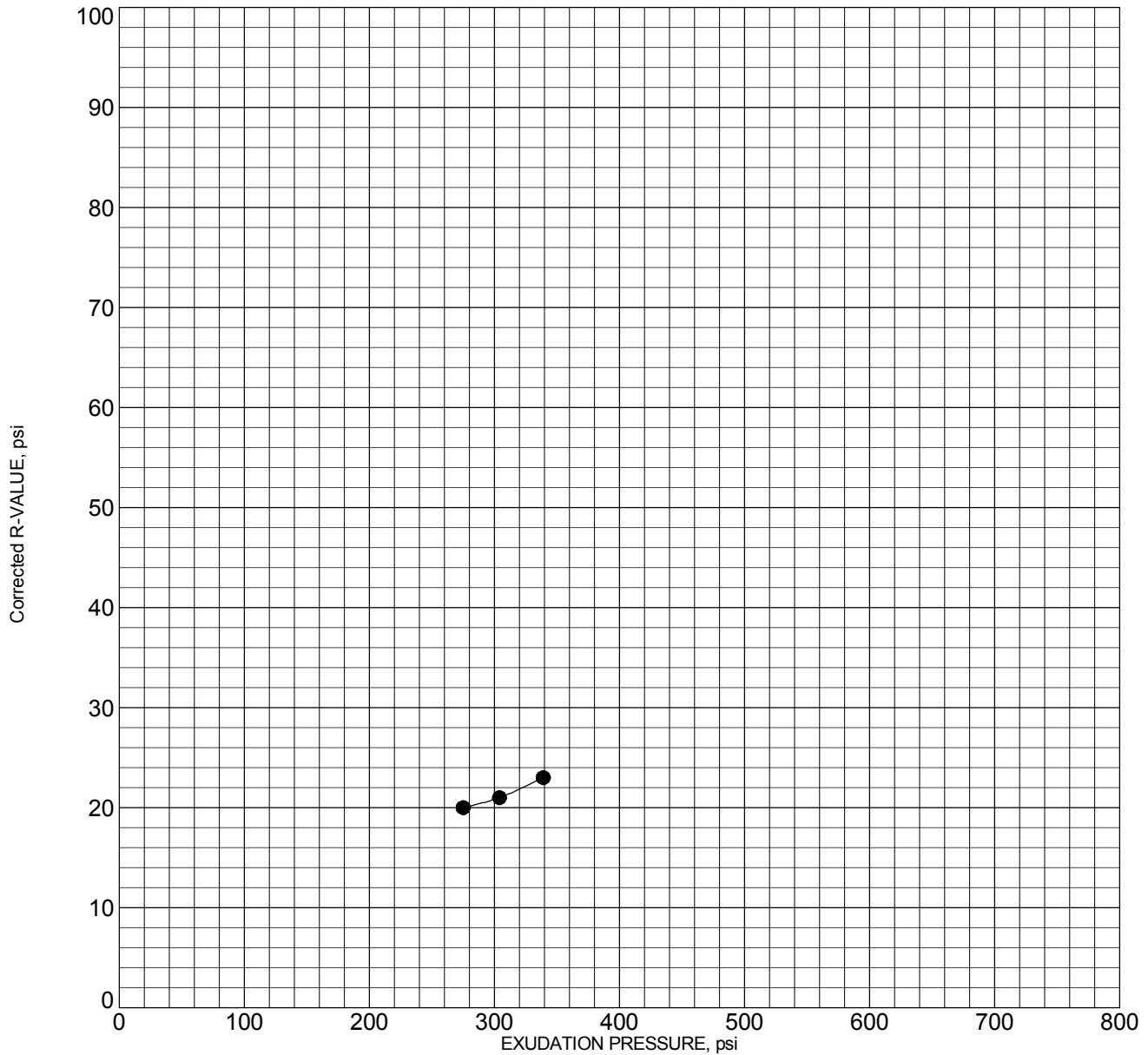
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R-VALUE AND EXPANSION PRESSURE - ASTM D2844

WAIALE, POHA KUOKALA, AND ALAE BRIDGES
SEISMIC RETROFIT
KULA, MAUI, HAWAII

Plate
B - 18



Sample: Bulk-6

Depth: 1.0 - 5.0 feet

R-Value at 300 psi Exudation Pressure: **21**

Description: Brown silty sand (SM) with some gravel

R-Value Test Performed by Ninyo & Moore

No.	Compaction Pressure	Density	Moisture Content	Horizontal Pressure @160 psi	Sample Height	Exudation Pressure	R-Value	Corrected R-Value
	(psi)	(pcf)	(%)	(psi)	(in)	(psi)		
1	100	89.2	35.0	100	2.49	339	23	23
2	100	87.1	25.5	104	2.53	304	21	21
3	100	84.9	36.0	106	2.45	275	20	20



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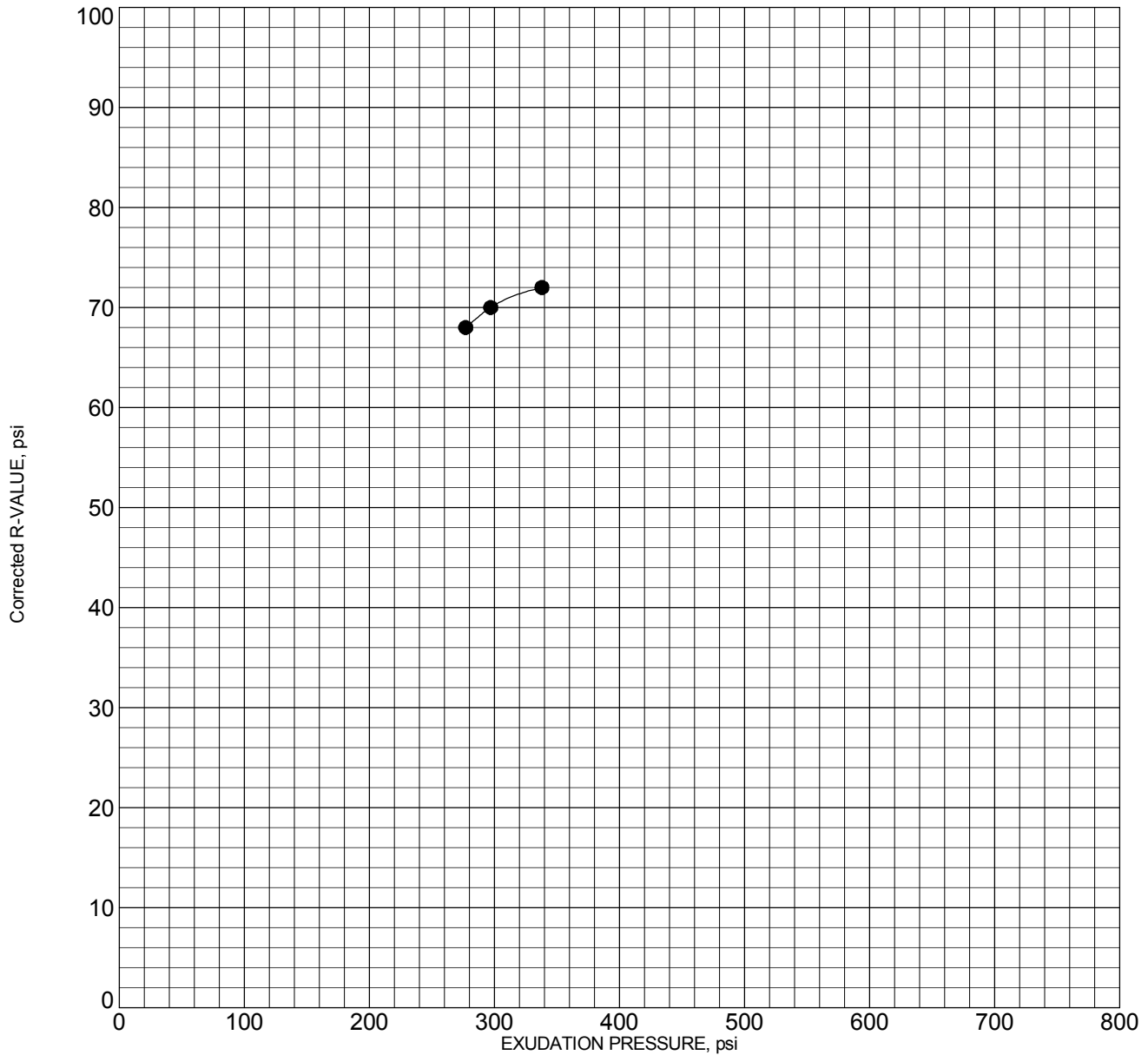
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R-VALUE AND EXPANSION PRESSURE - ASTM D2844

WAIALE, POHAUOKALA, AND ALAE BRIDGES
SEISMIC RETROFIT
KULA, MAUI, HAWAII

Plate
B - 19



Sample: Bulk-3

Depth: 1.0 - 5.0 feet

R-Value at 300 psi Exudation Pressure:	70
--	-----------

Description: Brown silty sand (SM) with some gravel

R-Value Test Performed by Ninyo & Moore

No.	Compaction Pressure	Density	Moisture Content	Horizontal Pressure @160 psi	Sample Height	Exudation Pressure	R-Value	Corrected R-Value
	(psi)	(pcf)	(%)	(psi)	(in)	(psi)		
1	250	90.0	31.7	28	2.44	338	72	72
2	250	85.1	32.2	30	2.45	297	70	70
3	250	82.2	32.7	32	2.48	277	68	68



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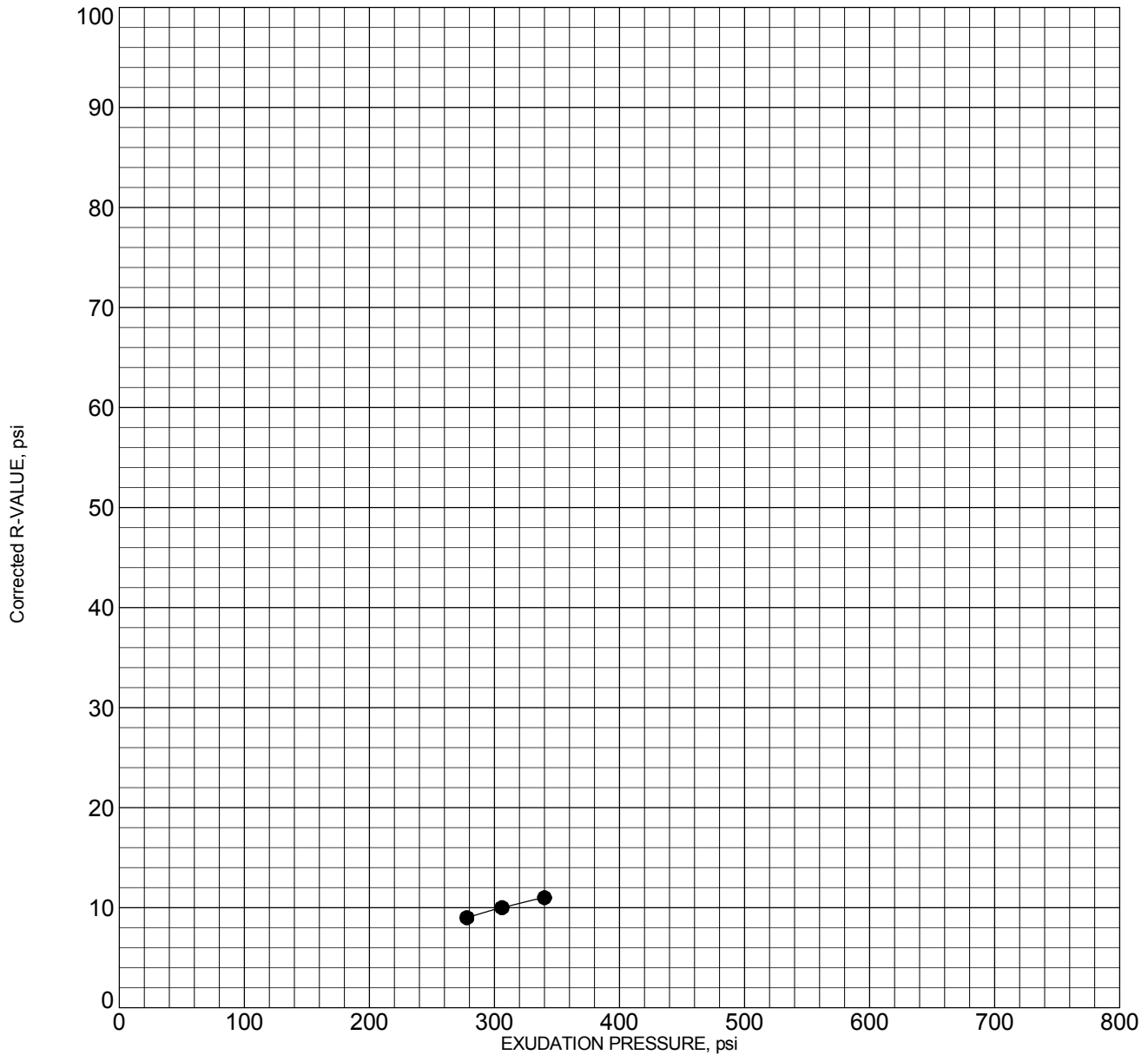
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R-VALUE AND EXPANSION PRESSURE - ASTM D2844

WAIALE, POHAUOKALA, AND ALAE BRIDGES
SEISMIC RETROFIT
KULA, MAUI, HAWAII

Plate
B - 20



Sample: Bulk-4

Depth: 1.0 - 5.0 feet

Description: Brown sandy silt (ML) with some gravel

R-Value at 300 psi Exudation Pressure:	10
--	-----------

R-Value Test Performed by Ninyo & Moore

No.	Compaction Pressure	Density	Moisture Content	Horizontal Pressure @160 psi	Sample Height	Exudation Pressure	R-Value	Corrected R-Value
	(psi)	(pcf)	(%)	(psi)	(in)	(psi)		
1	100	92.7	31.8	127	2.55	340	11	11
2	100	95.5	32.3	130	2.52	306	10	10
3	100	93.1	32.8	132	2.49	278	9	9



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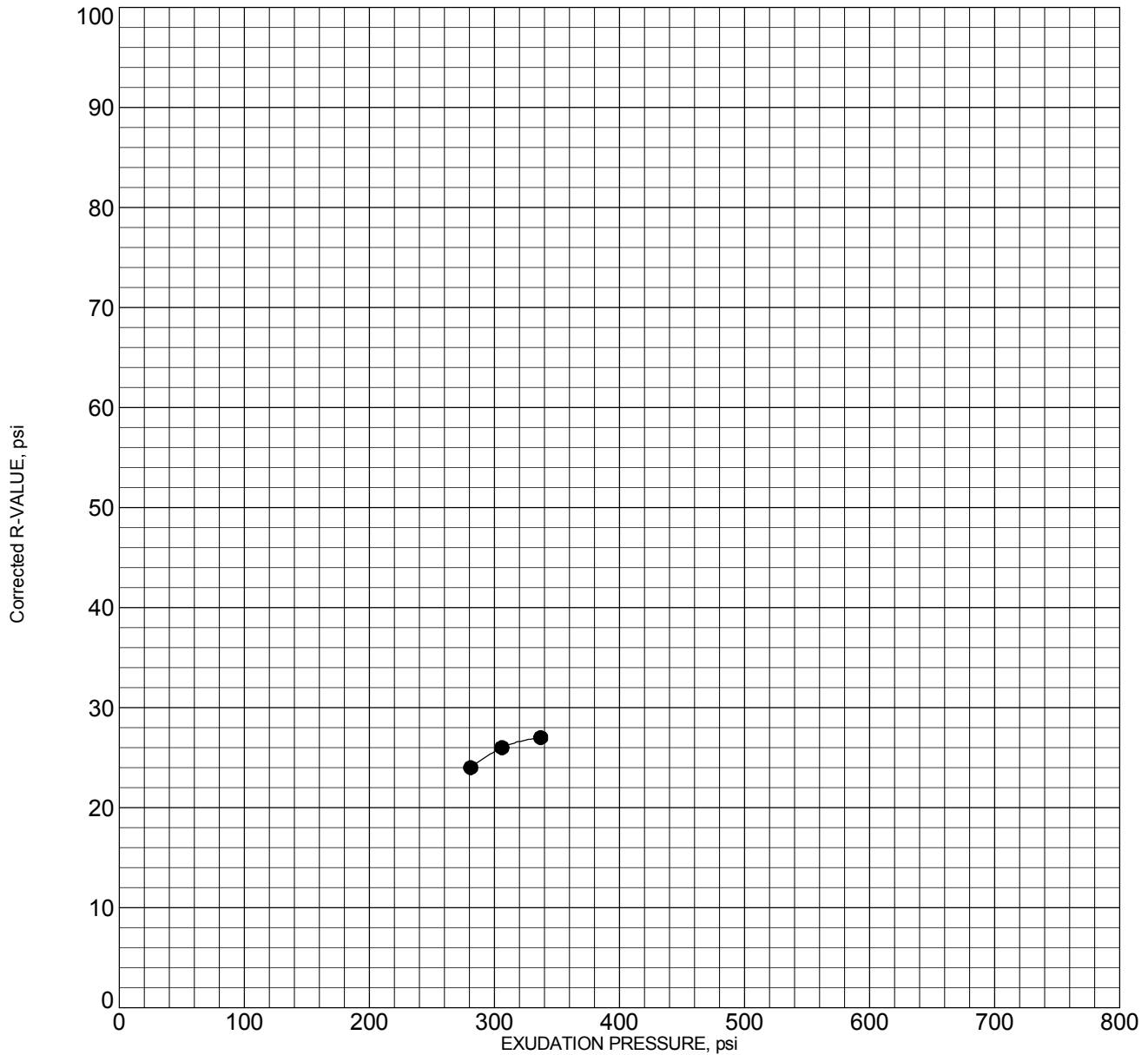
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R-VALUE AND EXPANSION PRESSURE - ASTM D2844

WAIALE, POHAKUOKALA, AND ALAE BRIDGES
SEISMIC RETROFIT
KULA, MAUI, HAWAII

Plate
B - 21



Sample: Bulk-1

Depth: 1.5 - 5.0 feet

R-Value at 300 psi Exudation Pressure:	26
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Description: Brown sandy silt (ML) with traces of gravel

R-Value Test Performed by Ninyo & Moore

No.	Compaction Pressure	Density	Moisture Content	Horizontal Pressure @160 psi	Sample Height	Exudation Pressure	R-Value	Corrected R-Value
	(psi)	(pcf)	(%)	(psi)	(in)	(psi)		
1	100	90.6	34.8	92	2.53	337	27	27
2	100	83.9	35.3	94	2.56	306	26	26
3	100	81.8	35.8	97	2.44	281	24	24



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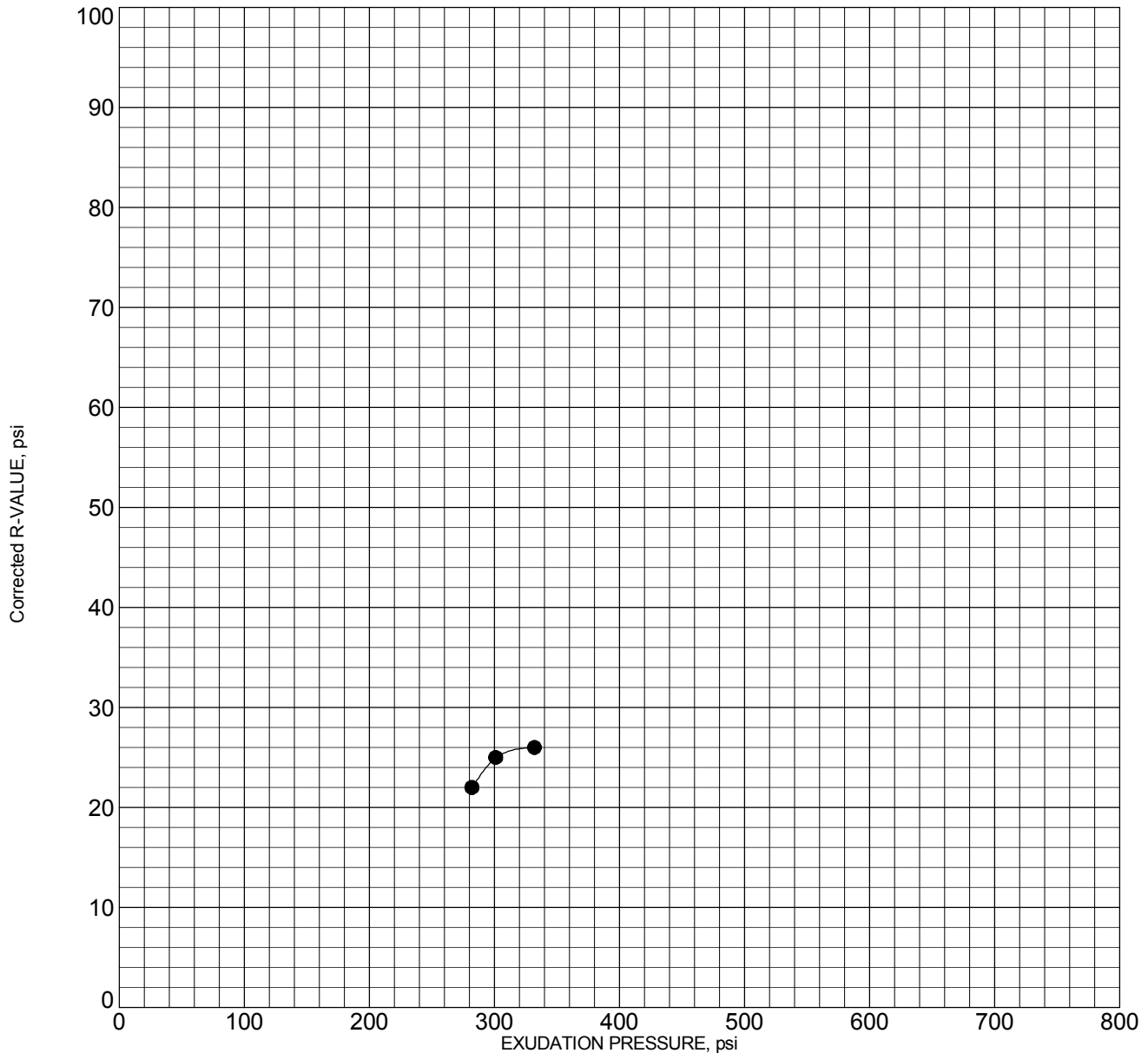
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R-VALUE AND EXPANSION PRESSURE - ASTM D2844

WAIALE, POHAKUOKALA, AND ALAE BRIDGES
SEISMIC RETROFIT
KULA, MAUI, HAWAII

Plate
B - 22



Sample: Bulk-2

Depth: 1.5 - 5.0 feet

Description: Brown sandy silt (ML) with a little gravel

R-Value at 300 psi Exudation Pressure:	25
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R-Value Test Performed by Ninyo & Moore

No.	Compaction Pressure	Density	Moisture Content	Horizontal Pressure @160 psi	Sample Height	Exudation Pressure	R-Value	Corrected R-Value
	(psi)	(pcf)	(%)	(psi)	(in)	(psi)		
1	100	93.5	32.6	94	2.56	332	26	26
2	100	92.6	33.1	96	2.48	301	25	25
3	100	88.7	33.6	101	2.44	282	22	22



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R-VALUE AND EXPANSION PRESSURE - ASTM D2844

WAIALE, POHA KUOKALA, AND ALAE BRIDGES
SEISMIC RETROFIT
KULA, MAUI, HAWAII

Plate
B - 23

APPENDIX C

WAIALE, POHAKUOKALA, AND ALAE BRIDGES
SEISMIC RETROFIT
KULA, MAUI, HAWAII

A-1 8.0' TO 51.5'



WAIALE, POHAKUOKALA, AND ALAE BRIDGES
SEISMIC RETROFIT
KULA, MAUI, HAWAII

A-2 28.5' TO 50.0'



WAIALE, POHAKUOKALA, AND ALAE BRIDGES
SEISMIC RETROFIT
KULA, MAUI, HAWAII

P-1 11.3' TO 30.0'



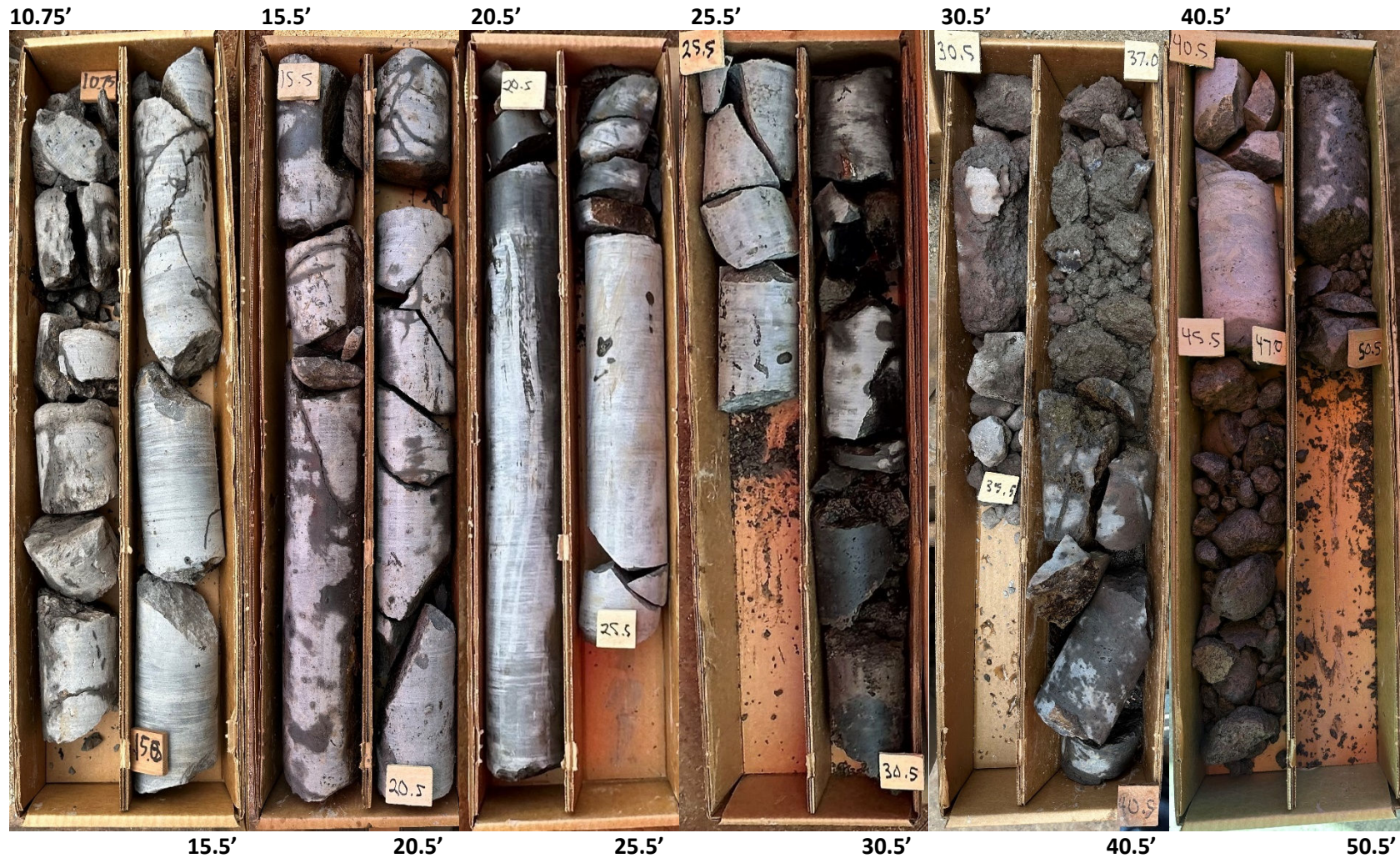
**WAIALE, POHAKUOKALA, AND ALAE BRIDGES
SEISMIC RETROFIT
KULA, MAUI, HAWAII**

P-1 30.0' TO 45.0'



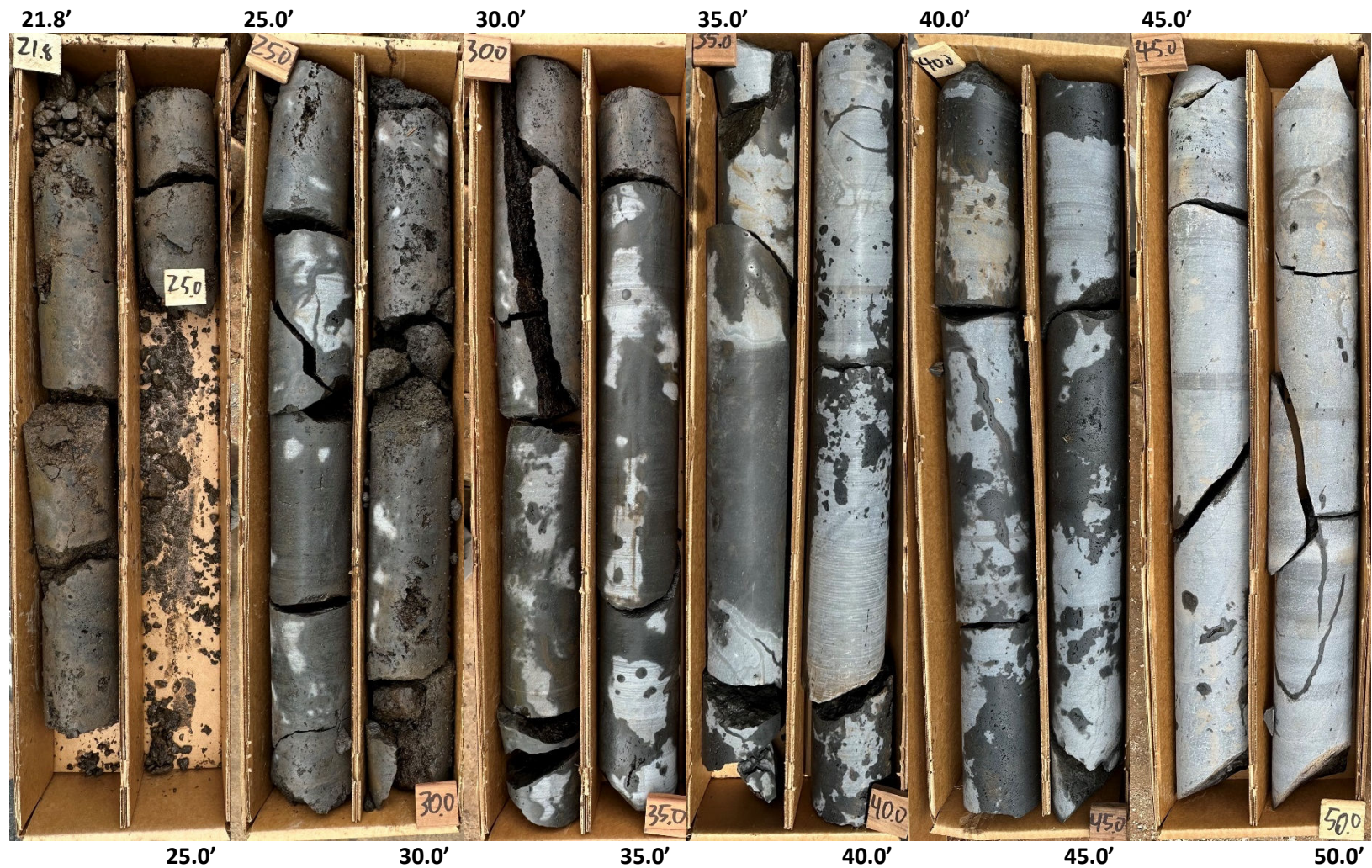
WAIALE, POHAKUOKALA, AND ALAE BRIDGES
SEISMIC RETROFIT
KULA, MAUI, HAWAII

P-2 10.75' TO 50.5'



WAIALE, POHAKUOKALA, AND ALAE BRIDGES
SEISMIC RETROFIT
KULA, MAUI, HAWAII

W-1 21.8' TO 50.0'



WAIALE, POHAKUOKALA, AND ALAE BRIDGES
SEISMIC RETROFIT
KULA, MAUI, HAWAII

W-2 24.0' TO 50.5'

