

PRELIMINARY GEOTECHNICAL ENGINEERING EXPLORATION
NEW ANIMAL QUARANTINE STATION (AQS)
HALAWA, OAHU, HAWAII
W.O. 8052-10 DECEMBER 10, 2021

Prepared for

AHL



GEOLABS, INC.
Geotechnical Engineering and Drilling Services

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SUMMARY OF FINDINGS AND RECOMMENDATIONS

Based on our preliminary field exploration results, available boring data and geologic information, the subsurface conditions beneath the proposed New Animal Quarantine Station project site generally consists of a thin layer of dense to very dense fill underlain by stiff to hard older alluvium to the maximum depth explored of about 51.5 feet below the existing ground surface. Distinct cobble and boulder layers were encountered near the top of the older alluvium deposit in most of our borings, ranging in thickness from about 3 to 11 feet. Groundwater was encountered at depths of approximately 41.4 to 43.6 feet below the existing ground surface at the time of our field exploration, corresponding to elevations of about +49.4 to +56.1 feet Mean Sea Level (MSL). However, it should be noted that water levels may vary with seasonal precipitation, perched groundwater, groundwater withdrawal, and other factors.

Based on the anticipated subsurface conditions, we believe that new structures located on the competent ground may be supported on shallow footing foundations bearing on competent near-surface soils or new compacted fills placed to achieve the design finished grades. A preliminary allowable bearing pressure of up to 4,000 pounds per square foot (psf) may be used to design the shallow foundation bearing on the recompacted on-site soils and/or new compacted structural fills needed to achieve the finished grades.

We anticipate the walkways and/or first floor of the new structures will consist of concrete slabs-on-grade. Our field exploration and experience in the area indicate the near-surface clayey soils generally exhibit low to moderate shrink/swell characteristics when subjected to fluctuations in the soil moisture contents. To reduce the potential for appreciable structural distress resulting from swelling of the subgrade soils, we recommend properly preparing the subgrade soils prior to fill placement. In addition, we recommend providing 12 to 18 inches of non-expansive select granular fill materials below the cushion fill to support the concrete slabs-on-grades.

Based on the results of infiltration testing performed within Boring No. 107, an infiltration rate of 0.75 inches per hour may be used for the preliminary design. Due to the potential variability of the subsurface conditions, the absorption capacity of a disposal system should be confirmed by conducting additional infiltration tests during the final design phase and construction, if appropriate.

The information and preliminary recommendations presented herein are intended to be solely in support of the planning and preliminary engineering process; and, as such, may not be sufficient nor be appropriate for the detailed design of the individual structures and site elements of the project. Therefore, we recommend conducting additional field

exploration as the design for the individual structures and site elements progresses to allow for the formulation of project-specific recommendations for each structure and element.

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

END OF SUMMARY OF FINDINGS AND RECOMMENDATIONS

SECTION 1. GENERAL

This report presents the results of our preliminary geotechnical engineering exploration and engineering analyses performed for the *New Animal Quarantine Station* (AQS) project in Halawa on the Island of Oahu, Hawaii. The project location and general vicinity are shown on the Project Location Map, Plate 1.

This report summarizes our findings and presents our preliminary geotechnical recommendations based on field exploration, laboratory testing, and engineering analyses. The preliminary recommendations presented herein are intended for the preliminary design of new structures and site elements for the New Animal Quarantine Station project only. The findings and preliminary recommendations presented herein are subject to the additional field exploration and limitations noted at the end of this report.

1.1 Project Considerations

The new AQS facility will move from the present location on the east side of the Interstate Route H-3 Freeway to the west side of the H-3 Freeway in the Halawa area on the Island of Oahu, Hawaii. We understand that the new Oahu Correctional Community Center (OCCC) facility will occupy the existing AQS facility.

The new AQS Facility will consist of:

- AQS Building (one to two-story building)
- Animal kennel area
- Earth berms on the north and east sides of the kennel
- Retention basin at the lowest elevation area

The proposed AQS facility is approximately bounded by Halawa Valley Road to the west and north and the Interstate Route H-3 Freeway to the east and southeast. The general location of the project site and the layout of the proposed facility elements are shown on the Site Plan, Plate 2.

Based on the anticipated scope of work for the planned project, this preliminary geotechnical exploration report includes discussions on structure foundations, retaining walls, site grading, pavement, and infiltration results.

1.2 Purpose and Scope

The purpose of our geotechnical engineering exploration was to obtain an overview of the surface and subsurface conditions to develop an idealized soil/rock data set to formulate preliminary geotechnical engineering recommendations for the design of the planned project. The work was performed in general accordance with the scope of services outlined in our fee proposal dated February 3, 2021. The scope of work for this exploration included the following tasks and work efforts:

1. Review of available soils and other information in the general project vicinity.
2. Coordinate staking of borehole locations and underground utility line clearance, including submitting a One Call application.
3. Trail clearing to provide access for our truck-mounted drill rig using mechanized equipment.
4. Drilling and sampling of seven boreholes to depths of about 5 to 51.5 feet below the existing ground surface for a total of approximately 239 lineal feet.
5. Perform infiltration testing at one location at a depth of 5 feet.
6. Coordination of the trail clearing, field exploration, logging of boreholes and supervision of the infiltration test by our geologist.
7. Laboratory testing of selected soil and rock samples obtained during the field exploration as an aid in classifying the materials and evaluating their engineering properties.
8. Analyses of the field and laboratory data to formulate preliminary geotechnical recommendations for the design of the new facility.
9. Preparation of this report summarizing our work and presenting our findings and preliminary geotechnical engineering recommendations.
10. Coordination of our overall work on the project by our project engineer.
11. Quality assurance and client-design team consultation by our principal engineer.
12. 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 samples obtained from our field exploration are presented in Appendix B. Field Infiltration test results 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 on the items needed for seismic design, such as seismicity, soil liquefaction and soil profile characteristics for seismic analysis.

2.1 Regional Geology

The Island of Oahu was built by the extrusion of basaltic lava from the Waianae and Koolau Shield Volcanoes. The older Waianae Volcano is estimated to be middle to late Pliocene in age, and the Koolau Volcano is estimated to be late Pliocene to early Pleistocene in age. As volcanic activity at the Waianae Volcano ceased, lava flows from the Koolau Volcano banked against its eroded eastern slope forming the Schofield Plateau.

The Koolau Volcanic Shield was built during the late Pliocene Epoch and early Pleistocene Epoch by the extrusion of successive thin-bedded lava flows. The main shield-building stage ceased approximately 2.5 million years ago. Evidence from historic drilled wells indicates that the Island of Oahu has subsided by as much as 1,200 feet since the cessation of the early volcanic activity (Macdonald and Abbott, 1970). During the period of island subsidence, coral-algal reefs began to grow along the southern coast of Oahu forming embayments protected by barrier reefs. A series of lagoons formed behind the barrier reefs, and both terrigenous and marine sediments accumulated in the lagoons (Macdonald and Abbott, 1970).

During the Pleistocene Epoch (Ice Age), many sea level changes occurred as a result of widespread glaciation in the continental areas of the world. As the great continental glaciers accumulated, the level of the oceans fell because there was less water available to fill the oceanic basins. Conversely, as the glaciers receded (melted), global sea levels rose because the volume of water increased. The land mass comprising

the Island of Oahu remained essentially stable during these water level changes, and the fluctuations were eustatic in nature. These glacio-eustatic fluctuations resulted in stands of the sea that were both higher and lower relative to the present sea level on the Island of Oahu.

The higher sea level stands caused landform changes, including the accumulation of deltas and alluvial fans composed of terrigenous sediments in the heads of the old bays, the accumulation of reef deposits at correspondingly higher elevations, and the accumulation of lagoonal and/or marine sediments in the quiet lagoonal waters protected by barrier reefs. The concurrent growth of reefs and the accumulation of lagoonal sediments also resulted in the deposition of coral-algal limestone and marl materials within the predominantly lagoonal sedimentary unit.

The lower sea level stands caused streams to carve drainages into the coastal plain platforms composed of sedimentary and coral reef deposits. In addition, subaerial exposure of the calcareous sediments caused consolidation of the soft deltaic materials and lagoonal deposits and the induration of calcareous reef materials. Furthermore, renewed subaerial erosion acting at the upper elevations of the volcanic shield caused the downstream deposition of terrigenous alluvial sediments under relatively higher energy conditions.

During periods of no significant sea level changes, continued meandering stream action extended the alluvial deltas and fans seaward and deposited alluvial materials overlying the marine-lagoonal sediments.

As discussed above, the majority of the Island of Oahu was formed during the main volcanic shield building stage, which eventually experienced a hiatus. After a long period of volcanic inactivity, during which time erosion incised deep valleys into the Koolau Shield Volcano along with the accumulation of the terrigenous and lagoonal deposits along the coastal areas, volcanic activity returned to portions of the Island of Oahu as a series of localized lava flows followed by explosive cinder and tuff cone formations. These late eruptions belong to the Honolulu Volcanic Series and are believed to have occurred between about 30,000 and 800,000 years ago.

In summary, the project site is located on the Southern flank of the Koolau Volcano and on the lower portion of Halawa valley. In general, the subsurface materials underlying the project site consist of older alluvial soils and basalt formation overlain by recent fills.

2.2 Site Description

The proposed AQS facility generally encompasses the existing developed and undeveloped areas of land between Halawa Valley Street and the Interstate Route H-3 Freeway in the Halawa area on the Island of Oahu, Hawaii. The current Hawaii Department of Transportation field offices and the Department of Health Sanitation Branch facilities will remain. However, the current Waiahole Water System office and parking lot, the empty field next to the Sanitation Branch buildings, and currently unoccupied buildings on the northern end of the project site will be redeveloped.

At the time of our field exploration, the Waiahole Water System Office was still in use. The empty field to the east of these offices was surrounded by a chain-link fence. We understand that until recently, this field was used to raise cattle. Watering troughs were still present on the northern side of the site during our field exploration program. The unoccupied facility on the northern side of the project site generally consisted of one to two-story warehouse structures.

Based on our field observations and the topographic survey map provided, the existing ground surface is relatively flat with a gentle downward slope towards the southwest. Existing ground surface elevations range from about +118 feet Mean Sea Level (MSL) at the northeast to about +80 feet MSL at the southwest.

2.3 Subsurface Conditions

The subsurface conditions at the project site were explored by drilling and sampling seven borings, designated as Boring Nos. 101 through 107, extending to depths of about 5 to 51.5 feet below the existing ground surface. In addition, two bulk samples of the near-surface soils, designated as Bulk-1 and Bulk-2, were obtained to evaluate the pavement support characteristics of the near-surface soils. The approximate boring and bulk sample locations are shown on the Site Plan, Plate 2.

Based on our field exploration, the project site is generally underlain by a relatively thin layer of surface fill up to about 1.5 feet thick underlain by older alluvium extending to the maximum depth explored of about 51.5 feet below the existing ground surface. The surface fill was encountered in four of the drilled borings and generally consisted of dense to very dense silty/gravelly sand. The older alluvium generally consisted of stiff to hard silty clay and clayey/sandy silt with cobbles and boulders, and medium dense to very dense gravelly sand/cobbles with boulders. It should be noted that the clayey older alluvium near the groundwater table encountered in Boring No. 103 (depths of about 35 to 45 feet) graded to medium stiff. Distinct cobble and boulder layers were encountered near the top of the older alluvium deposit in most of our borings, ranging in thickness from about 3 to 11 feet.

Groundwater was encountered in the drilled borings at depths of approximately 41.4 to 43.6 feet below the existing ground surface at the time of our field exploration. These depths correspond to elevations of about +49.4 to 56.1 feet Mean Sea Level (MSL). However, it should be noted that water levels may vary with seasonal precipitation, perched groundwater, groundwater withdrawal, and other factors.

Detailed descriptions of the field exploration methodology and graphic representations of the materials encountered in the borings are presented on the Logs of Borings in Appendix A. We performed laboratory tests on selected soil samples obtained during our field exploration, and the test results are presented in Appendix B. Field Infiltration test results are presented in Appendix C.

2.4 Seismic Design Considerations

Based on the International Building Code, 2012 Edition (IBC 2012), the project site may be subjected to seismic activity, and seismic design considerations will need to be addressed for the project. The following subsections provide discussions on the seismicity of the Island of Oahu and the soil profile 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.

The effects of earthquakes occurring on the Islands of Hawaii and Maui may be felt on the Island of Oahu. For example, small landslides occurred on the Island of Oahu as a result of the Maui Earthquake of 1938 (M6.8). Some houses on the Island of Oahu were reportedly damaged as a result of the Lanai Earthquake of 1871 (M7+). In the last 150 years of recorded history, we are not aware of earthquakes greater than Magnitude 6 that have occurred on the Island of Oahu. An earthquake of Magnitude 4.8 to 5.0 occurred along the Diamond Head Fault in 1948 on the Island of Oahu. The moderate tremor resulted in broken store windows, ruptured building walls, and broken underground water mains.

2.4.2 Liquefaction Potential

Based on the International Building Code (2012 Edition), the project site should be evaluated for the potential for soil liquefaction. The effects of potential liquefaction may be taken into consideration in the design of the proposed redevelopment.

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

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

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

In general, the subsurface information obtained from the borings drilled indicate that the project site is underlain by stiff to hard alluvial deposits extending to the maximum depth of our drilled borings. 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 based on the soil and rock encountered and the absence of groundwater 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 at the project site, we believe the project site may be classified from a seismic analysis standpoint as a “Stiff Soil Profile” site corresponding to a Site Class D based on the ASCE Standard ASCE/SEI 7-10 (Table No. 20.3-1), referenced by the International Building Code, 2012 Edition. Based on Site Class D, the following seismic design parameters were estimated and may be used for seismic analysis of the project.

SEISMIC DESIGN PARAMETERS	
Parameter	Value
MCE Peak Bedrock Acceleration, PBA (Site Class B)	0.264
Spectral Response Acceleration (Site Class B), S_s	0.573
Spectral Response Acceleration (Site Class B), S_1	0.165
Site Class	“D”
Site Coefficient, F_a	1.342
Site Coefficient, F_v	2.139
Site Coefficient, F_{pga}	1.272
MCE Peak Ground Acceleration, PGA (Site Class D)	0.336
Design Spectral Response Acceleration, S_{DS}	0.512
Design Spectral Response Acceleration, S_{D1}	0.236

END OF SITE CHARACTERIZATION

SECTION 3. DISCUSSION AND RECOMMENDATIONS

Based on our preliminary field exploration results, available boring data and geologic information, the subsurface conditions across the project site are generally stiff/dense competent material. We envision new structures located on the competent ground may be supported on shallow footing foundations bearing on competent near-surface soils or new compacted fills placed to achieve the design finished grades. In addition, we envision that retaining walls, slabs-on-grade, new pavement sections, and new utilities will be required for the planned project.

Detailed discussions and preliminary geotechnical recommendations in support of the New Animal Quarantine Station project are presented in the following sections.

3.1 Shallow Foundations

We envision new structures may be supported on shallow footing foundations bearing on competent near-surface soils and/or new fills placed to achieve the design finished grades. For planning and preliminary design purposes, an allowable bearing pressure of up to 4,000 pounds per square foot (psf) may be used to design the shallow foundations bearing on the recompacted on-site soils and/or new compacted structural fills needed to achieve the finished grades. These bearing values are for sizing the footings based on dead-plus-live loads and may be increased by one-third ($\frac{1}{3}$) for transient loads, such as those caused by wind or seismic forces.

The bottom of footing excavations should be recompacted to at least 90 percent relative compaction to provide a relatively firm and smooth bearing surface prior to placing reinforcing steel and/or concrete. Soft and/or loose materials encountered at the bottom of footing excavations should be over-excavated to expose the underlying firm materials. The over-excavation may be backfilled with the on-site soils compacted to a minimum of 90 percent relative compaction, or the bottom of footing may be extended down to bear directly on the underlying competent materials.

In general, the bottom of footings should be embedded a minimum of 18 to 24 inches below the lowest adjacent finished grades. Footings constructed near tops of slopes or on sloping ground conditions should be embedded deep enough to provide a

minimum horizontal setback distance of 6 feet measured from the outside edge of the footings (base of footing) to the face of the slope. Footings adjacent to planned (or existing) retaining walls should be embedded deep enough to avoid surcharging the retaining wall foundations, or the planned retaining walls should be designed to resist the additional structural loads.

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

If structure foundations are designed and constructed in strict accordance with our recommendations, we estimate total settlements of footings supported on the recompacted on-site soils and/or new compacted structural fills to be on the order of about 1 inch or less. We estimate that differential settlements between adjacent footings supported on similar materials to be on the order of about 0.5 inches.

Lateral loads acting on the structures may be resisted by friction developed between the bottom of the foundation and the bearing soil and by passive earth pressure acting against the near-vertical faces of the foundation system. A coefficient of friction of 0.3 to 0.35 may be used for footings bearing on the recompacted on-site soils and/or new compacted structural fills. Resistance due to passive earth pressure may be estimated using an equivalent fluid pressure of 300 to 350 pounds per square foot per foot of depth (pcf). This assumes the soils around the footings are well-compacted. Unless covered by slabs or pavements, the passive pressure resistance in the upper 12 inches of soil should be neglected. In addition, the passive pressure resistance for foundations on slopes should be reduced.

3.2 Slabs-On-Grade

We anticipate the walkways and/or first floor of the new structures for the project will consist of reinforced concrete slabs-on-grade. Based on the existing topography and the anticipated finished grade, we envision the slabs-on-grade generally will be supported

on the recompacted on-site soils and/or new compacted fills placed to raise the existing ground surface to the finished subgrades.

Our field exploration and experience in the area indicate the near-surface clayey soils generally exhibit low to moderate shrink/swell characteristics when subjected to fluctuations in the soil moisture contents. Unless slabs-on-grade constructed above these expansive soils are properly designed, there is a potential for future distress to the lightly loaded slabs-on-grade resulting from shrinking and swelling of the clayey soils due to changes in the moisture content. To reduce the potential for appreciable structural distress resulting from swelling of the subgrade soils, we recommend properly preparing the subgrade soils prior to fill placement. In addition, we recommend providing 12 to 18 inches of non-expansive select granular fill materials below the cushion fill to support the concrete slabs-on-grades. Additional testing should be performed at each structure when their locations are finalized to confirm and/or modify these recommendations.

For interior building slabs (not subjected to vehicular traffic or sustained machinery vibration), we recommend placing a minimum 4-inch thick layer of cushion fill consisting of open-graded gravel (ASTM C33, No. 67 gradation) below the slabs. The open-graded gravel cushion fill would provide uniform support of the slabs and would serve as a capillary moisture break. To reduce the potential for appreciable future moisture infiltration through the slab and subsequent damage to floor coverings, an impervious moisture barrier, such as a plastic membrane, is recommended on top of the gravel cushion fill layer. Flexible floor coverings, such as carpet or sheet vinyl, should be considered because they can better mask minor slab cracking. In addition, we recommend designing interior walls to incorporate some flexibility in accommodating a small amount of possible ground movements.

Where the slabs will be subjected to vehicular traffic or sustained machinery vibration, such as trucks and/or forklifts, we recommend providing a 6-inch layer of compacted aggregate subbase below the slabs in lieu of the 4-inch thick gravel cushion fill layer. The moisture barrier also may be omitted for these slabs. The aggregate subbase should consist of crushed basaltic aggregates compacted to a minimum of 95 percent relative compaction.

For the design of structural slabs supported on aggregate subbase, a modulus of subgrade reaction of about 200 pounds per square inch per inch of deflection (pci) may be used for the compacted aggregate subbase. Where slabs are intended to function as rigid pavements for trucks, a minimum slab thickness of 6 inches may be used for preliminary design purposes. Provisions should be made for proper load transfer across the slab joints that will be subject to vehicular traffic. The thickened edges of slabs adjacent to unpaved areas should be embedded at least 12 inches below the lowest adjacent grade.

In order to reduce the potential for appreciable distress due to differential movements between the heavier footings and the lighter building slab, we recommend using free-floating slabs-on-grade with no structural connections to the wall and column foundations. Joint filler and sealant may be used to fill the openings between the edges of the slab and other structural elements. To further reduce the potential for appreciable distress to the building slabs-on-grade and foundations resulting from water infiltration into the subsurface from areas immediately adjacent to the building foundations, we recommend providing a concrete sidewalk (or pavement) around the perimeter of the new building. Construction joints should be provided at intervals equal to the width of the sidewalk with expansion joints at right-angle intersections.

Based on our experience with expansive soils, minor differential slab movements between the building slab and the abutting sidewalk slabs have been observed on several occasions. We believe this situation may be attributed to the lack of maintenance of the sidewalk subgrade moisture content after the initial subgrade preparation. It should be noted that the moisture content requirement of the clayey subgrades (at least 2 percent above the optimum moisture) is an important requirement considering the expansive nature of the on-site clayey soils. Therefore, the subgrade soils below the sidewalks should be properly moisture-conditioned and kept moist until the placement of the select granular fill and concrete. In addition, consideration may be given to structurally connecting the two abutting slabs with dowels or other structural connections, especially at the entrances to the building and other openings in the walls.

It should be emphasized that the areas adjacent to the slabs should be backfilled tightly against the slab edges with low expansion, relatively impervious soils. These areas should also be graded to divert water away from the slabs and to reduce the potential for water ponding around the slabs and foundations.

3.3 Site Grading

We understand that the design finished grades of the project have not been set at this time. Therefore, a grading plan was not available at the time this report was prepared. We anticipate minor cuts and fills may be required to achieve the design finish grade. Items of earthwork that are addressed in the subsequent subsections include the following:

1. Site Preparation
2. Fills and Backfills
3. Fill Placement and Compaction Requirements
4. Excavations
5. Cut and Fill Slopes

3.3.1 Site Preparation

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

Foundations and slabs of the existing structures to be demolished should be removed. Over-excavations resulting from the demolition operations should be backfilled with compacted fill material. Existing underground utilities to be abandoned should be removed, and the resulting excavation should be properly backfilled with the excavated on-site materials. The on-site materials should be moisture-conditioned to above the optimum moisture content, placed in 8-inch level loose lifts, and compacted to a minimum of 90 percent relative compaction. Utilities to be abandoned in-place under the proposed structure should be backfilled by pumping lean concrete or Controlled Low Strength Material (CLSM) under low pressure.

Subgrades, including cut areas, areas at grade, or areas designated to receive fills, should be scarified to a minimum depth of about 12 inches, moisture-conditioned to at least 2 percent above the optimum moisture content, and compacted to a minimum of 90 percent relative compaction. Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same soil determined in accordance with ASTM D1557. Optimum moisture is the water content (percentage by dry weight) corresponding to the maximum dry density.

Soft and/or loose, weak, yielding areas, or cavities disclosed during site preparation operations should be over-excavated to expose firm ground, and the resulting excavation should be backfilled with general fill materials compacted to a minimum of 90 percent relative compaction. The material resulting from the over-excavation should be removed and disposed of properly or used in landscaping areas, where appropriate.

Where shrinkage cracks are observed after the subgrade compaction, we recommend preparing the subgrade soil again as recommended above. Saturation and subsequent yielding of the exposed subgrade due to inclement weather and poor drainage may require over-excavating the soft areas and replacing these areas with well-compacted fill. The need for over-excavation due to soft subgrade soil conditions should be evaluated in the field.

3.3.2 Fills and Backfills

In general, the excavated on-site materials may be reused as a source of general fill materials provided that deleterious materials such as vegetation and/or organic matter are removed and over-sized materials greater than 3 inches in maximum dimension are screened.

Imported general fill materials needed to fill the site may consist of materials with a low to moderate expansion potential. Imported general fill materials should consist of soil materials with a maximum particle size of 3 inches or less with sufficient fines (between 10 and 60 percent particles passing the No. 200 sieve) to prevent the occurrence of voids in the compacted mass. In addition, general fill

materials should have a California Bearing Ratio (CBR) value of 8 or greater and a swell of 2 percent or less when tested in accordance with ASTM D1883. It should be noted that the general fill requirements presented herein are intended as guidelines only and may be modified based on additional laboratory testing and field observations on the available fill materials during construction.

Select granular or structural fill materials required for the project construction should consist of non-expansive select granular material, such as crushed basalt. The material should be well-graded from coarse to fine with particles no larger than 3 inches in largest dimension and should contain between 10 and 30 percent particles passing the No. 200 sieve. The material should have a laboratory CBR value of 20 or more and should have a maximum swell of 1 percent or less when tested in accordance with ASTM D1883.

Where required, imported fill materials should be tested for conformance with these recommendations prior to delivery to the project site for the intended use. An accredited testing laboratory should test the imported fill materials for conformance with these recommendations prior to delivery to the project site for the intended use.

Aggregate base course and aggregate subbase materials should meet the material requirements for Base Course and Subbase Course as specified in Subsections 703.06 and 703.17, respectively, of the Hawaii Standard Specifications for Road and Bridge Construction (2005). Imported fill materials should be tested for conformance with these recommendations prior to delivery to the project site for the intended use.

3.3.3 Fill Placement and Compaction Requirements

General fill materials should be moisture-conditioned to at least 2 percent above the optimum moisture, placed in level lifts of about 8 inches in loose thickness, and compacted to at least 90 percent relative compaction. Select granular fill materials should be moisture-conditioned to above the optimum moisture, placed in level lifts of about 8 inches in loose thickness, and compacted to at least 95 percent relative compaction. Aggregate base course and subbase 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

Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same soil established in accordance with ASTM D1557. Optimum moisture is the water content (percentage by weight) corresponding to the maximum dry density. Compaction should be accomplished by sheepfoot rollers, vibratory rollers, or other types of acceptable compaction equipment. Water tamping, jetting, or ponding should not be allowed to compact the fills.

3.3.4 Excavations

Based on the anticipated grading and our field exploration, excavations will generally be required for the construction of the foundations and installation of the drainage structures and underground utilities. Some of the excavations may encounter cobbles and boulders within the fill and older alluvium encountered during our field exploration. It should be noted that relatively thick layers of cobbles and boulders were encountered near the existing ground surface in most of our borings. It is anticipated that most of the materials may be excavated with normal heavy excavation equipment. However, excavations into the underlying cobble and boulder layers may require the use of hoerams.

The above discussions regarding the rippability of the subsurface materials are based on field data from the borings drilled at the site. Contractors should be encouraged to examine the site conditions and the subsurface data to make their own reasonable and prudent interpretation.

3.3.5 Cut and Fill Slopes

We envision the cut slopes at the project site will generally expose the stiff/dense fill and older alluvium encountered in the drilled borings. In addition, we understand that fill slopes will be required for the new earthen berms planned around the future kennel area. In general, cut slopes and permanent fill slopes constructed of the on-site soils may be designed with a slope inclination of two horizontal to one vertical

(2H:1V) or flatter. The filling operations should start at the lowest point and continue up in level horizontal compacted layers in accordance with the above general fill placement recommendations.

Fill slopes should be constructed by overfilling and cutting back to the design slope inclination to obtain a well-compacted slope face. Surface water should be diverted away from the tops of slopes, and slope planting should be provided as soon as possible to reduce the potential for erosion of the finished slopes.

3.4 Retaining Structures

Retaining structures may be required for the project construction. Based on the subsurface conditions encountered, the following guidelines may be used for the preliminary design of retaining structures.

3.4.1 Wall Foundations

In general, we believe the retaining structure foundations may be designed in accordance with the “Shallow Foundations” section herein. Retaining wall foundations should be at least 18 inches wide and the bottom should be embedded a minimum of 24 inches below the lowest adjacent finished grades.

For sloping ground conditions, the footing should extend deeper to obtain a minimum 6-foot setback distance measured horizontally from the outside edge of the footing (base of footing) to the face of the slope. Wall footings oriented parallel to the direction of the slope should be constructed in stepped footings.

3.4.2 Lateral Earth Pressures

Retaining structures should be designed to resist the lateral earth pressures due to the adjacent soils and surcharge effects. The recommended lateral earth pressures for design of retaining walls, expressed in equivalent fluid pressures of pounds per square foot per foot of depth (pcf), are presented in the following table for retaining wall backfills consisting of on-site clayey soils. These lateral earth pressures do not include hydrostatic pressures that might be caused by groundwater trapped behind the structures.

LATERAL EARTH PRESSURES FOR DESIGN OF RETAINING STRUCTURES			
<u>Backfill Condition</u>	<u>Earth Pressure Component</u>	<u>Active</u> (pcf)	<u>At-Rest</u> (pcf)
Level Backfill	Horizontal	40	60
	Vertical	None	None
Maximum 2H:1V Sloping Backfill	Horizontal	64	81
	Vertical	32	41

We recommend compacting the backfill behind retaining structures to between 90 and 95 percent relative compaction. Over-compaction of the retaining structure backfill should be avoided. The backfill materials should be moisture-conditioned to above the optimum moisture content prior to being utilized as backfill materials.

In general, the at-rest condition should be used for retaining structures where the top of the structure is restrained from movement prior to backfilling of the wall. The active condition should be used only for gravity retaining walls and retaining structures that are free to deflect by as much as 0.5 percent of the wall height.

Surcharge stresses due to areal surcharges, line loads, and point loads within a horizontal distance equal to the depth of the retaining structures should be considered in the design. For uniform surcharge stresses imposed on the loaded side of the retaining structure, a rectangular distribution with a uniform pressure equal to 53 percent of the vertical surcharge pressure acting on the entire height of the structure, which is restrained, may be used in the design. For retaining structures that are free to deflect (cantilever), a rectangular distribution equal to 36 percent of the vertical surcharge pressure acting over the entire height of the structure may be used for design.

3.4.3 Drainage

Retaining walls should be well-drained to reduce the build-up of hydrostatic pressures. A typical drainage system would consist of a 12-inch wide zone of permeable material, such as open-graded gravel (ASTM C33, No. 67 gradation),

placed directly around a perforated pipe (perforations facing down) at the base of the wall discharging to an appropriate outlet or weepholes. As an alternative, a prefabricated drainage product, such as MiraDrain, may be used instead of the drainage material. The prefabricated drainage product also should be connected hydraulically to a perforated pipe at the base of the wall.

Backfill behind the permeable drainage zone may consist of compacted on-site materials or free-draining compacted fills, where specified by the designer. Unless covered by concrete slabs, the upper 12 inches of backfill should consist of low-expansion, relatively impervious materials to reduce the potential for excessive water infiltration behind the walls.

3.5 Pavement Design

We envision flexible pavement is planned for the project. In general, we anticipate the vehicle loading for the project will consist of primarily passenger vehicles, light pick-up trucks, trash trucks, and occasional maintenance vehicles. For pavements, the flexible pavement design procedure based on the “Guide for the Design of Pavement Structures” (1993) developed by the American Association of State Highway and Transportation Officials (AASHTO) was used in the pavement design.

Based on the results of our field exploration, laboratory testing, and the above traffic volume assumptions, we recommend using the following flexible pavement sections for preliminary design purposes:

Flexible Pavements Subjected to Light Vehicular Traffic and Parking Areas

3.0-Inch Asphaltic Concrete

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

9.0-Inch Total Pavement Thickness on Moist Compacted Subgrade

The pavement subgrade soils should be scarified to a depth of at least 8 inches, moisture-conditioned to at least 2 percent above the optimum moisture content and compacted to no less than 95 percent relative compaction. Where scarification of the subgrades is not practical, subgrade materials should be proof-rolled with a minimum 10-ton vibratory drum roller for a minimum of eight passes. California Bearing Ratio tests

and/or field observations should be performed on the actual subgrade materials during construction to confirm that the above design sections are adequate.

The aggregate base and select granular fill material should also be compacted to a minimum of 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 ASTM D1557. Optimum moisture is the water content (percentage by dry weight) corresponding to the maximum dry density.

Paved areas should be sloped, and drainage gradients should be maintained to carry the surface water off-site. Surface water ponding should not be allowed on-site during or after construction. Where concrete curbs are used to isolate landscaping in or adjacent to the pavement areas, we recommend extending the curbs a minimum of 2 inches into the soils below the aggregate base or subbase course layers to reduce the potential for migration of landscape water into the pavement section. Alternatively, a subdrain system could be constructed to collect excess water from landscaping irrigation. For long-term performance, we recommend constructing a subdrain system adjacent to the paved/landscaped areas.

3.6 Underground Utility Lines

We envision new on-site utility lines (i.e., electric, water, sewer, and drain lines) may be required for the project. We anticipate most of the utility line trenches will be excavated in the compacted fills and/or stiff on-site alluvial soils. In general, we recommend using granular bedding consisting of 6 inches of free-draining granular materials (ASTM C33, No. 67 gradation) below the pipes for uniform support. Free-draining granular materials, such as No. 3B Fine gravel (ASTM C33, No. 67 gradation), also should be used for the initial trench backfill up to about 12 inches above the pipes.

It is critical to use this free-draining material to reduce the potential for the formation of voids below the haunches of the pipes and to provide adequate support for the sides of the pipes. Improper backfill material around the pipes and improper placement of the backfill could result in backfill settlement and pipe damage.

Where soft and/or loose compressible soils are encountered at or near the invert elevations, we recommend providing a subgrade stabilization layer consisting of 18 to 24 inches of No. 2 Rock (ASTM C33, No. 4 gradation) wrapped in a non-woven filter fabric (Mirafi 180N or equivalent) below the bedding layer for uniform support. The stabilization layer should extend beyond the sides of the pipe a minimum width of one-fourth the outside diameter of the pipe or 12 inches, whichever is greater.

The upper portion of the trench backfill from a level of 12 inches above the pipes to the top of the subgrade or finished grade may consist of the excavated granular materials with a maximum particle size of 6 inches or select granular fill materials. The backfill material should be moisture-conditioned to above the optimum moisture content, placed in maximum 8-inch level loose lifts, and mechanically compacted to at least 90 percent relative compaction. In areas where trenches will be in paved areas, the upper 3 feet of the trench backfill below the pavement finished grade should be compacted to no less than 95 percent relative compaction.

3.7 Field Infiltration Testing

Infiltration testing was conducted at one boring location (Boring No. 107) drilled at the project site to evaluate the infiltration characteristics of the subsurface materials. The test was performed in general accordance with the procedures in Appendix D of the State of Maryland Department of the Environment “Stormwater Design Manual, Volumes I and II” (Rev. 2009). These procedures are consistent with the other states’ procedures and generally may be considered an industry standard.

The field infiltration test was performed by augering the boring to a test depth of about 5 feet below the existing ground surface. Upon reaching the test depth, a 4-inch inside diameter PVC solid casing was set to the bottom of the drilled hole to allow infiltration only through the soil exposed at the bottom of the boring. A falling head infiltration test was performed to determine the infiltration rate of the underlying subsurface materials. The test consists of four trials of filling the casing with about 24 inches of water and taking periodic readings over a 1-hour trial period or until the hole drains completely. The infiltration rate is then calculated based on the results of the fourth and final trial. Results of the final infiltration rate are presented in the table below. Details

of our field infiltration test are presented in Appendix D.

SUMMARY OF FIELD INFILTRATION TEST RESULTS			
<u>Test Location</u>	<u>Test Depth</u> (feet)	<u>Test Elevation</u> (feet MSL)	<u>Final Infiltration Rate</u> (inches/hour)
B-107	5	+91	0.75

Results of the infiltration testing indicated that the infiltration rate measured at Boring No. 107 was about 0.75 inches per hour. It should be noted that the infiltration values presented above are the infiltration rates through the subsurface materials exposed at the bottom of the tested borehole and reduce the lateral spreading of the water in the subsurface materials. Due to the potential variability of the subsurface conditions, the absorption capacity of a disposal system should be confirmed by conducting additional infiltration tests during construction, if appropriate.

3.8 Drainage

Finished grades outside the new structures should be sloped to shed water away from the slabs and foundations and to reduce the potential for ponding around the structures. In addition, it is advised to install roof gutter systems around the buildings and to divert the discharge away from the slab and foundation areas. Excessive landscape watering near the slabs and foundations also should be avoided. Planters next to foundations should be avoided or have concrete bottoms and drains to reduce the potential for excessive water infiltration into the subsurface.

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

3.9 Additional Field Exploration

This exploration was conducted on a preliminary basis to obtain an overview of the general subsurface conditions within the New Animal Quarantine Station area. The information and preliminary recommendations presented herein are intended to solely be in support of the planning process and, as such, may not be sufficient nor appropriate for the detailed design of the individual structures and site elements of the project. Therefore, we recommend that additional field exploration be conducted as the design for the individual structures and site elements progresses to allow for the formulation of project-specific recommendations for each structure and element.

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 and bulk samples. Variations of the subsurface conditions between and beyond the field borings may occur, and the nature and extent of these variations may not become evident until construction is underway. If variations then appear evident, it will be necessary to re-evaluate the recommendations presented herein.

The locations of the field borings indicated in this report were approximate, having been estimated by taping from reference points and visible features shown on the topographic survey map transmitted by Architects Hawaii Limited on May 26, 2020. The physical locations and elevations of the borings should be considered accurate only to the degree implied by the methods used.

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

This report has been prepared for the exclusive use of AHL and their project consultants for specific application to the *New Animal Quarantine Station* project as described herein in accordance with generally accepted geotechnical engineering principles and practices. No warranty is expressed or implied.

This report has been prepared solely for the purpose of assisting the architects and engineers in the preliminary planning of the project. Therefore, this report may not contain sufficient data, or the proper information, for use to form the basis for preparation of construction cost estimates.

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

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

END OF LIMITATIONS

CLOSURE

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

Project Location Map..... Plate 1
Site Plan..... Plate 2
Field Exploration Appendix A
Laboratory Tests Appendix B
Infiltration Test Data Appendix C

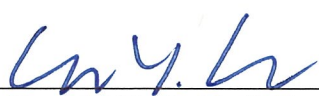
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Respectfully submitted,

GEOLABS, INC.

By 

Nicholas Kam, P.E.
Project Engineer

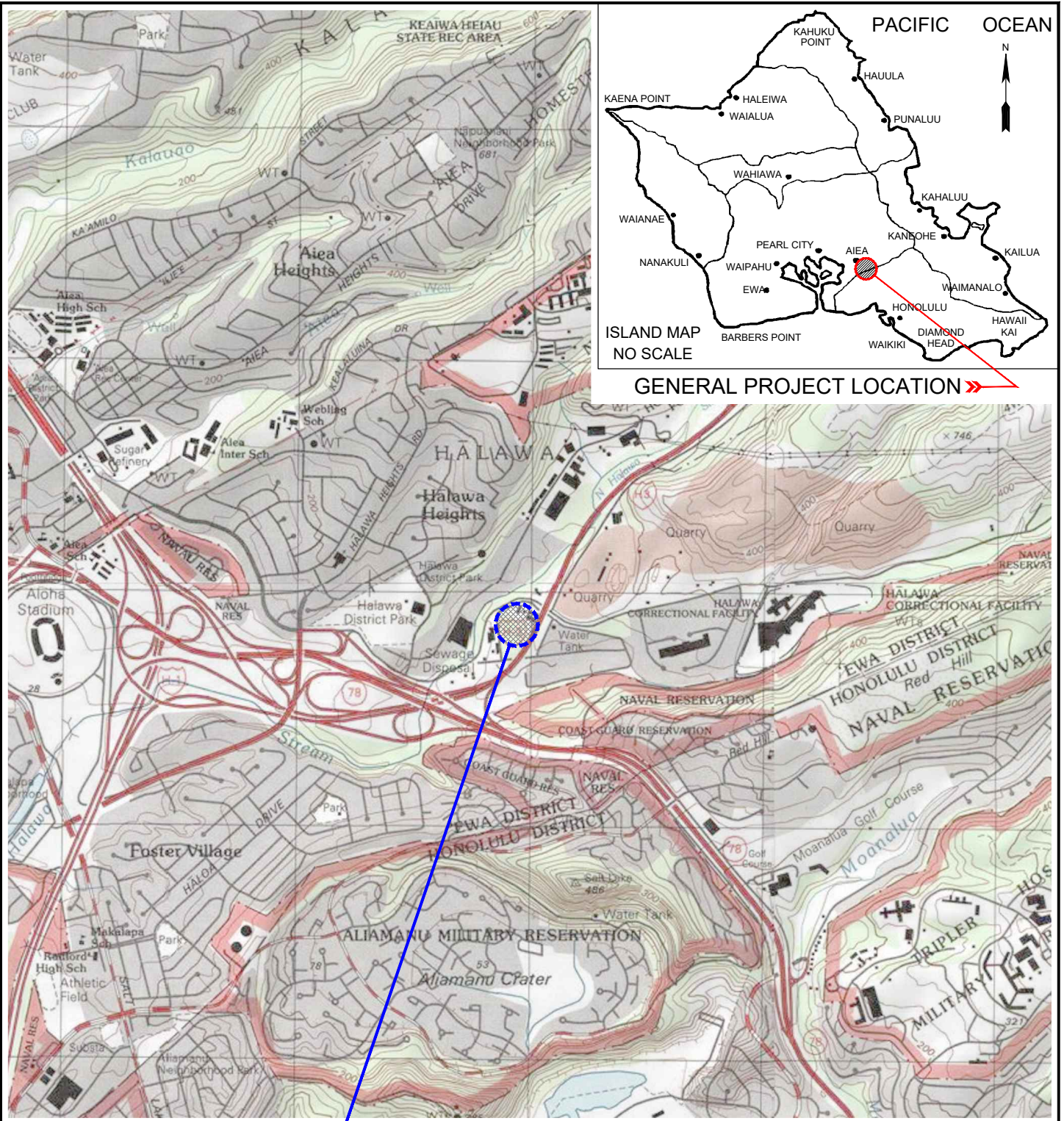
By 

Gerald Y. Seki, P.E.
Vice President

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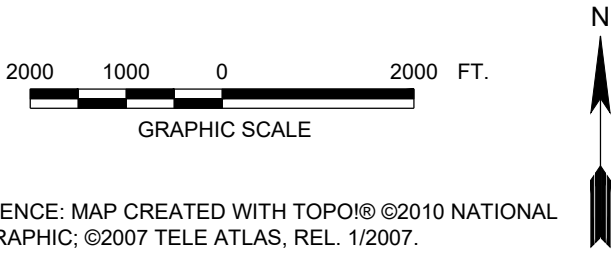
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PLATES



PROJECT LOCATION ➤

PROJECT LOCATION MAP
NEW ANIMAL QUARANTINE STATION (AQS)
HALAWA, OAHU, HAWAII

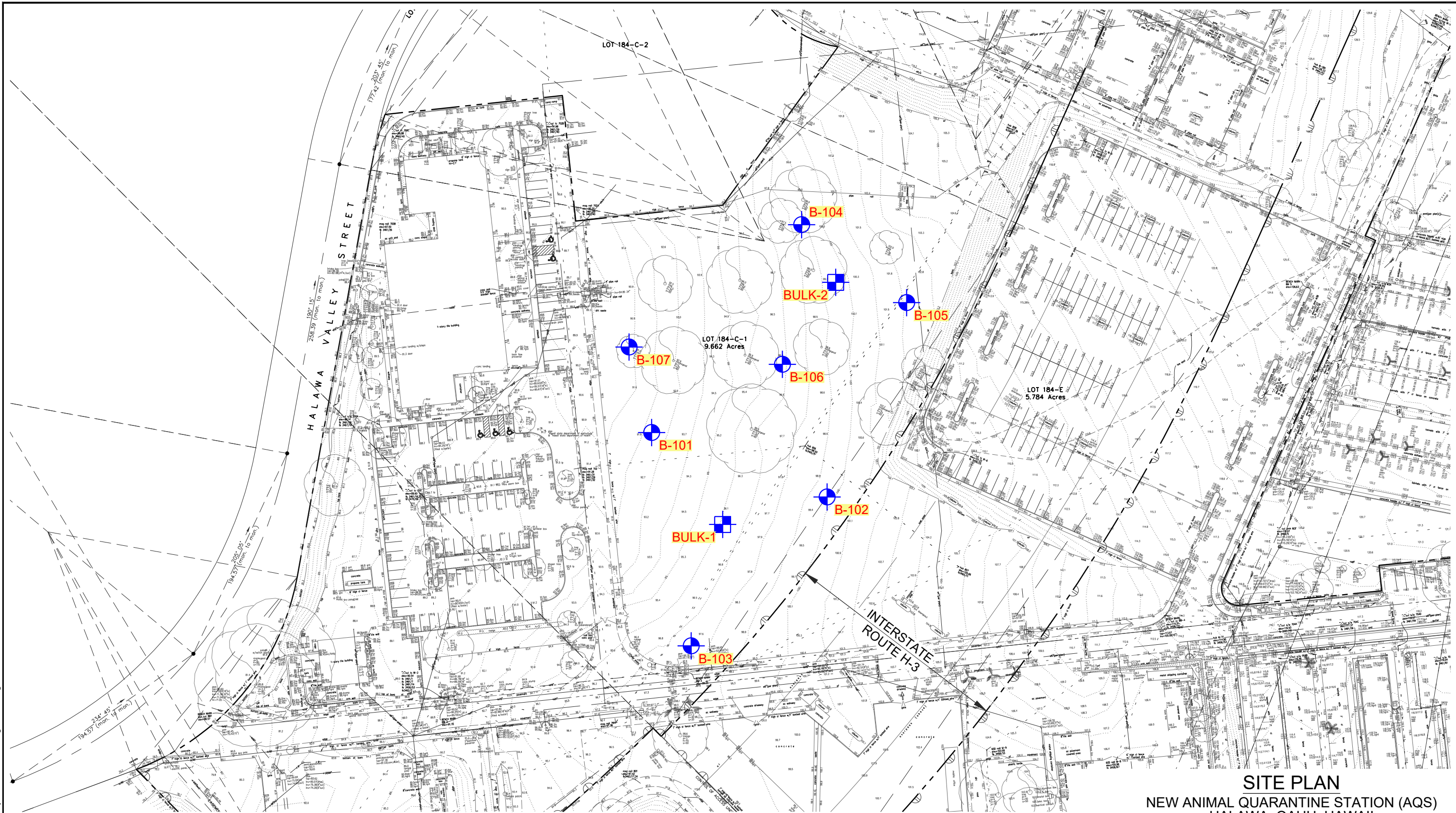


GEOLABS, INC. <i>Geotechnical Engineering</i>		
DATE	DRAWN BY	PLATE
JUNE 2021	ASP	
SCALE	W.O.	
1" = 2,000'	8052-10	1



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

CAD User: ASPASIONJR File Last Updated: June 22, 2021 6:00:29pm Plot Date: June 22, 2021 - 6:01:02pm
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LEGEND:

-  APPROXIMATE BORING LOCATION
-  APPROXIMATE BULK SAMPLE LOCATION



SITE PLAN
 NEW ANIMAL QUARANTINE STATION (AQS)
 HALAWA, OAHU, HAWAII

GEOLABS, INC.

Geotechnical Engineering

DATE	DRAWN BY	PLATE
AUGUST 2021	ASP	
SCALE	W.O.	2
1" = 80'	8052-10	

REFERENCE: TOPOGRAPHIC SURVEY MAP TRANSMITTED BY ARCHITECTS HAWAII LIMITED MAY 26, 2020.

APPENDIX A

APPENDIX A

Field Exploration

We explored the subsurface conditions at the project site by drilling and sampling seven borings, designated as Boring Nos. 1 through 7, extending to depths of about 5 to 51.5 feet below the existing ground surface. In addition, two bulk samples of the near-surface soils, designated as Bulk-1 and Bulk-2, were obtained to evaluate the pavement support characteristics of the near-surface soils. The approximate boring and bulk sample locations are shown on the Site Plan, Plate 2. The borings were drilled using a truck-mounted drill rig equipped with continuous flight augers and coring tools.

Our geologists classified the materials encountered in the borings by visual and textural examination in the field in general accordance with ASTM D2488, Standard Practice for Description and Identification of Soils, and monitored the drilling operations on a near-continuous (full-time) basis. These classifications were further reviewed visually and by testing in the laboratory. Soils were classified in general accordance with ASTM D2487, Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System), as shown on the Soil Log Legend, Plate A-0.1. Deviations made to the soil classification in accordance with ASTM D2487 are described on the Soil Classification Log Key, Plate A-0.2. Graphic representations of the materials encountered are presented on the Logs of Borings, Plates A-1.1 through A-7.

Relatively “undisturbed” soil samples were obtained in general accordance with ASTM D3550, Ring-Lined Barrel Sampling of Soils, by driving a 3-inch OD Modified California sampler with a 140-pound hammer falling 30 inches. In addition, some samples were obtained from the drilled borings in general accordance with ASTM D1586, Penetration Test and Split-Barrel Sampling of Soils by driving a 2-inch OD standard penetration sampler using the same hammer and drop. The blow counts needed to drive the sampler the second and third 6 inches of an 18-inch drive are shown as the “Penetration Resistance” on the Logs of Borings at the appropriate sample depths. The penetration resistance shown on the Logs of Borings indicates the number of blows required for the specific sampler type used. The blow counts may need to be factored to obtain the Standard Penetration Test (SPT) blow counts.

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



UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)

MAJOR DIVISIONS			USCS	TYPICAL DESCRIPTIONS		
COARSE-GRAINED SOILS	GRAVELS	CLEAN GRAVELS LESS THAN 5% FINES		GW WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES		
		GRAVELS WITH FINES MORE THAN 12% FINES		GP POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES		
				GM SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES		
			GC CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES			
	SANDS	CLEAN SANDS LESS THAN 5% FINES		SW WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES		
				SP POORLY-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES		
		SANDS WITH FINES MORE THAN 12% FINES		SM SILTY SANDS, SAND-SILT MIXTURES		
				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	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 | LL | LIQUID LIMIT (NP=NON-PLASTIC) |
| | (3-INCH) O.D. MODIFIED CALIFORNIA SAMPLE | PI | PLASTICITY INDEX (NP=NON-PLASTIC) |
| | SHELBY TUBE SAMPLE | TV | TORVANE SHEAR (tsf) |
| | GRAB SAMPLE | UC | UNCONFINED COMPRESSION OR UNIAXIAL COMPRESSIVE STRENGTH |
| | CORE SAMPLE | TXUU | UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION (ksf) |
| | WATER LEVEL OBSERVED IN BORING AT TIME OF DRILLING | | |
| | WATER LEVEL OBSERVED IN BORING AFTER DRILLING | | |
| | WATER LEVEL OBSERVED IN BORING OVERNIGHT | | |



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Soil Classification Log Key

(with deviations from ASTM D2488)

GEOLABS, INC. CLASSIFICATION*

GRANULAR 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**)

COHESIVE SOIL (- #200 ≥ 50%)

- **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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NEW ANIMAL QUARANTINE STATION (AQS)
HALAWA, OAHU, HAWAII

Log of Boring

101

Laboratory			Field				Depth (feet)	Sample	Graphic	USCS	Description
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)					
											Approximate Ground Surface Elevation (feet): 92.5 *
	18	65			56/6"+ 10/0" Ref.					CH	Reddish brown SILTY CLAY with some sand, and gravel hard, dry (older alluvium)
	8	92			50/3"		5				Brownish gray GRAVELLY BASALTIC COBBLES with a little boulders (basaltic), very dense, dry (older alluvium)
	21				25/2"		10			CH	Brown SILTY CLAY with some cobbles (basaltic), and boulders (basaltic) very stiff to hard, dry (older alluvium)
											Brownish gray GRAVELLY BASALTIC COBBLES with some basaltic boulders (older alluvium)
Consol. TXUU S _u =0.2 ksf	29	85			58	4.5	15			CL	Orangish brown SANDY CLAY with some gravel, hard, dry (older alluvium)
LL=43 PI=24	19				29		20				grades to very stiff
					30/4"		25				grades to hard
	17				32		30				
							35				

BORING LOG 8052-10.GPJ GEOLABS.GDT 10/28/21

Date Started: July 19, 2021	Water Level: ▼ 43.1 ft. 07/19/2021 1403 HRS	Plate A - 1.1
Date Completed: July 19, 2021		
Logged By: D. Gremminger	Drill Rig: CME-75DG1 (Energy Transfer Ratio = 83.9%)	
Total Depth: 51.5 feet	Drilling Method: 4" Solid-Stem Auger	
Work Order: 8052-10	Driving Energy: 140 lb. wt., 30 in. drop	



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Geotechnical Engineering

NEW ANIMAL QUARANTINE STATION (AQS)
HALAWA, OAHU, HAWAII

Log of Boring

102

Laboratory			Field				Depth (feet)	Sample	Graphic	USCS	Description
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)					
	51	70			15				CH	(Continued from previous plate)	
	50				18		40		MH	Brown with orangish mottling CLAYEY SILT with some sand, very stiff, moist (older alluvium)	
UC=2.3 ksf	47	77			16	2.5	45			grades to stiff	
	44				11		50				
Boring terminated at 51.5 feet											
55											
60											
65											
70											

BORING LOG 8052-10.GPJ GEOLABS.GDT 10/28/21

Date Started: July 14, 2021	Water Level: ∇ 43.6 07/15/2021 0954 HRS	Plate A - 2.2
Date Completed: July 15, 2015		
Logged By: A. Taeb	Drill Rig: CME-75DG1 (Energy Transfer Ratio = 83.9%)	
Total Depth: 51.5 feet	Drilling Method: 4" Solid-Stem Auger	
Work Order: 8052-10	Driving Energy: 140 lb. wt., 30 in. drop	



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NEW ANIMAL QUARANTINE STATION (AQS)
HALAWA, OAHU, HAWAII

Log of Boring

103

Laboratory			Field				Depth (feet)	Sample	Graphic	USCS	Approximate Ground Surface Elevation (feet) : 97.5 *
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)					Description
	16	73			34					CH	Brown SILTY CLAY with some sand and a little gravel, very stiff, dry (older alluvium)
LL=55 PI=28	15				29						
Direct Shear Consol.	25	80			64	4.5	5				grades to hard
										MH	Brown with grayish mottling CLAYEY SILT with some sand and a little gravel, hard, dry (older alluvium)
LL=53 PI=21	29				43		10				
Consol.	45	66			36	3.0	15			MH	Greenish brown CLAYEY SILT with some sand and saprolitic gravel, very stiff, moist (older alluvium)
LL=60 PI=27	43				21		20				grades to grayish brown
TXUU S _v =3.7 ksf	44	79			38	4.5	25				
	46				15		30				grades to stiff
							35				

BORING LOG 8052-10.GPJ GEOLABS.GDT 10/28/21

Date Started: July 15, 2021	Water Level: ∇ 41.4 07/15/2021 0000 HRS	Plate A - 3.1
Date Completed: July 15, 2021		
Logged By: D. Gremminger	Drill Rig: CME-75DG1 (Energy Transfer Ratio = 83.9%)	
Total Depth: 51.5 feet	Drilling Method: 4" Solid-Stem Auger	
Work Order: 8052-10	Driving Energy: 140 lb. wt., 30 in. drop	



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NEW ANIMAL QUARANTINE STATION (AQS)
HALAWA, OAHU, HAWAII

Log of Boring

103

Laboratory			Field				Depth (feet)	Sample	Graphic	USCS	Description
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)					
	50	69			11	1.5				MH	(Continued from previous plate)
	52				5		40				
UC=1.1 ksf	51	73			13	1.0	45			CH	Brown SILTY CLAY with some sand, medium stiff (older alluvium)
	54				7		50				
Boring terminated at 51.5 feet											
							55				
							60				
							65				
							70				

Date Started: July 15, 2021

Date Completed: July 15, 2021

Logged By: D. Gremminger

Total Depth: 51.5 feet

Work Order: 8052-10

Water Level: ∇ 41.4 07/15/2021 0000 HRS

Drill Rig: CME-75DG1 (Energy Transfer Ratio = 83.9%)

Drilling Method: 4" Solid-Stem Auger

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

Plate

A - 3.2



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NEW ANIMAL QUARANTINE STATION (AQS)
HALAWA, OAHU, HAWAII

Log of Boring

104

Laboratory			Field				Depth (feet)	Sample	Graphic	USCS	Description
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)					
											Approximate Ground Surface Elevation (feet): 98.5 *
	20	83			47/6" +50/4" Ref.				SM		Reddish brown SILTY SAND with a little gravel, dry (fill)
			78						SP		Grayish brown GRAVELLY SAND with traces of silt, very dense, dry (alluvium) BOULDER at 2.3 to 5 feet
			7						MH		Brown with orange and gray mottling CLAYEY SILT with some sand, hard, moist (older alluvium)
LL=56 PI=20	45				59						grades with some gravel
	52				36						grades with traces of clay
			57						CH		Brown SILTY CLAY with traces of sand, stiff (older alluvium)
LL=70 PI=39	46		100								Boring terminated at 26.5 feet

BORING LOG 8052-10.GPJ GEOLABS.GDT 10/28/21

Date Started: July 12, 2021	Water Level: ∇ Not Encountered	Plate A - 4
Date Completed: July 12, 2021		
Logged By: A. Taeb	Drill Rig: CME-75DG1 (Energy Transfer Ratio = 83.9%)	
Total Depth: 26.5 feet	Drilling Method: 4" Solid-Stem Auger	
Work Order: 8052-10	Driving Energy: 140 lb. wt., 30 in. drop	



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NEW ANIMAL QUARANTINE STATION (AQS)
HALAWA, OAHU, HAWAII

Log of Boring

105

Laboratory			Field				Depth (feet)	Sample	Graphic	USCS	Approximate Ground Surface Elevation (feet): 102 *
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)					Description
Direct Shear	16	81			26/6" +50/4" Ref.				SM	Light brown SILTY SAND with traces of gravel, dry (fill)	
	9								ML	Grayish brown SANDY SILT with a little gravel, very dense, dry (alluvium)	
Sieve - #200 = 51.5%	14	91			30/0" Ref.					grades to dark brown	
UC=3.8 ksf	43	68			39	2.5	15		MH	Brown with dark gray mottling CLAYEY SILT with a little sand, very stiff, moist (alluvium)	
	51	66			31	2.0	20			grades to grayish brown, hard	
	44				34		25			Boring terminated at 26.5 feet	

BORING LOG 8052-10.GPJ GEOLABS.GDT 10/28/21

Date Started: July 12, 2021	Water Level: ▼ Not Encountered	Plate A - 5
Date Completed: July 12, 2021		
Logged By: A. Taeb	Drill Rig: CME-75DG1 (Energy Transfer Ratio = 83.9%)	
Total Depth: 26.5 feet	Drilling Method: 4" Solid-Stem Auger	
Work Order: 8052-10	Driving Energy: 140 lb. wt., 30 in. drop	



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Geotechnical Engineering

NEW ANIMAL QUARANTINE STATION (AQS)
HALAWA, OAHU, HAWAII

Log of Boring

106

Laboratory			Field				Depth (feet)	Sample	Graphic	USCS	Approximate Ground Surface Elevation (feet) : 97 *
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)					Description
UC=3.2 ksf	18	82			53		0		SP	Brown GRAVELLY SAND , dense, dry (fill)	
	25				39		1			Grayish brown GRAVELLY COBBLES with a little clayey silt, medium dense to dense, dry (older alluvium)	
	7				50/2" Ref.		5			grades to gray, very dense	
	30				42		10			grades to brown, dense	
	32	79			57	4.0	15		CH	Brown with orange mottling SILTY CLAY with a little sand, hard, moist (older alluvium)	
	33				24		20			grades to very stiff	
	36	78			63	4.0	25			grades with dark gray mottling	
							26.5			Boring terminated at 26.5 feet	

BORING LOG 8052-10.GPJ GEOLABS.GDT 10/28/21

Date Started: July 14, 2021	Water Level: ▼ Not Encountered	Plate A - 6
Date Completed: July 14, 2021		
Logged By: A. Taeb	Drill Rig: CME-75DG1 (Energy Transfer Ratio = 83.9%)	
Total Depth: 26.5 feet	Drilling Method: 4" Solid-Stem Auger	
Work Order: 8052-10	Driving Energy: 140 lb. wt., 30 in. drop	



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Geotechnical Engineering

NEW ANIMAL QUARANTINE STATION (AQS)
HALAWA, OAHU, HAWAII

Log of Boring

107

Laboratory			Field				Depth (feet)	Sample	Graphic	USCS	Approximate Ground Surface Elevation (feet) : 91 *
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)					Description
LL=52 PI=27	17	73			37				CH	Reddish brown SILTY CLAY with some sand, and gravel very stiff to hard, dry (older alluvium) grades to hard	
	24				32						
	36	64			73	4.5					
							5			Boring terminated at 5 feet	
							10				
							15				
							20				
							25				
							30				
							35				

BORING LOG 8052-10.GPJ GEOLABS.GDT 10/28/21

Date Started: July 15, 2021	Water Level: ∇ Not Encountered	Plate A - 7
Date Completed: July 15, 2021		
Logged By: D. Gremminger	Drill Rig: CME-75DG1 (Energy Transfer Ratio = 83.9%)	
Total Depth: 5 feet	Drilling Method: 4" Solid-Stem Auger	
Work Order: 8052-10	Driving Energy: 140 lb. wt., 30 in. drop	

APPENDIX B

APPENDIX B

Laboratory Tests

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

Nine Atterberg Limits tests (ASTM D4318) were performed on selected soil samples to evaluate the liquid and plastic limits to aid in soil classifications. The test results are summarized on the Logs of Borings at the appropriate sample depths. Graphic presentation of the test results is provided on Plate B-1.

One Sieve Analysis test (ASTM C117 & C136) was performed on a selected soil sample to evaluate the gradation characteristics of the soils and to aid in soil classification. Graphic presentation of the grain size distribution is provided on Plate B-2.

One one-inch Ring Swell test was performed on a relatively undisturbed sample to evaluate the swelling potential of the near-surface soils. The test results are summarized on Plate B-3.

Two Unconfined Compression tests (ASTM D2166) were performed on selected in-situ samples to evaluate the unconfined compressive strength of the on-site clayey soils. The test results are shown on the Logs of Borings at the appropriate sample depths. Individual stress-strain curves of the unconfined compression tests are presented on Plates B-4 and B-5.

Three Triaxial Unconsolidated Undrained Compression tests (ASTM D2850) were performed on selected soil samples to evaluate the undrained shear strength of the in-situ soils. The approximate in-situ effective overburden pressure was used as the applied confining pressure for the relatively “undisturbed” soil sample. The test results and the stress-strain curves are presented on Plates B-6 through B-8.

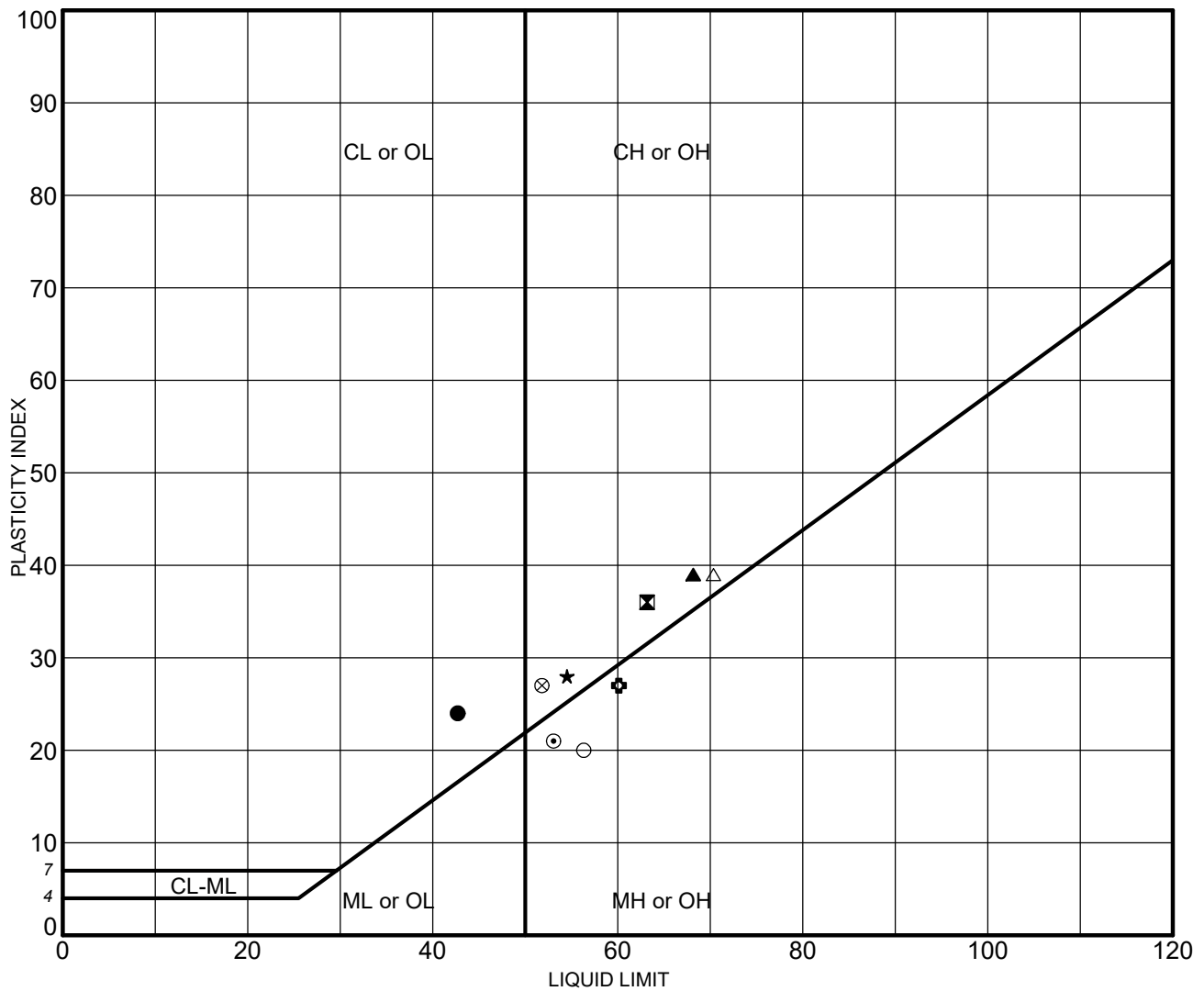
Two Direct Shear tests (ASTM D3080) were performed on selected samples to evaluate the shear strength characteristics of the material tested. The test results are presented on Plates B-9 and B-10.

Three Consolidation tests (ASTM D2435) were performed on samples of the potentially compressible soils to evaluate the compressibility characteristics of the materials encountered. Results of the consolidation tests are presented on Plates B-11 through B-13.

Two Modified Proctor compaction tests (ASTM D1557, Method C) were performed on selected bulk soil samples to evaluate the relationship between the moisture content and the dry density of the near-surface soils. The test results are presented on Plates B-14 and B-15.

Appendix B
Laboratory Tests

Two laboratory California Bearing Ratio tests (ASTM D1883) were performed on selected bulk soil samples to evaluate the pavement support characteristics of the soils. The test results are presented on Plates B-16 and B-17.



	Sample	Depth (ft)	LL	PL	PI	Description
●	B-101	20.0-21.5	43	19	24	Orangish brown sandy clay (CL) with some gravel
⊠	B-101	50.0-51.5	63	27	36	Brown with grayish mottling silty clay (CH) with a little sand
▲	B-102	10.0-11.5	68	29	39	Brown with gray mottling silty clay (CH) with a little sand
★	B-103	2.5-4.0	55	27	28	Brown silty clay (CH) with some sand and a little gravel
⊙	B-103	10.0-11.5	53	32	21	Brown with gray mottling clayey silt (MH) with some sand and a little gravel
⊕	B-103	20.0-21.5	60	33	27	Grayish brown clayey silt (MH) with some sand and gravel
○	B-104	11.5-13.0	56	36	20	Brown with orange and gray mottling clayey silt (MH) with some sand
△	B-104	21.5-23.0	70	31	39	Brown silty clay (CH) with trace of sand
⊗	B-107	0.5-2.0	52	25	27	Reddish brown silty clay (CH) with some sand and gravel

NP = NON-PLASTIC

G. ATTERBERG PL-100 LL-120 8052-10.GPJ GEOLABS.GDT 10/25/21

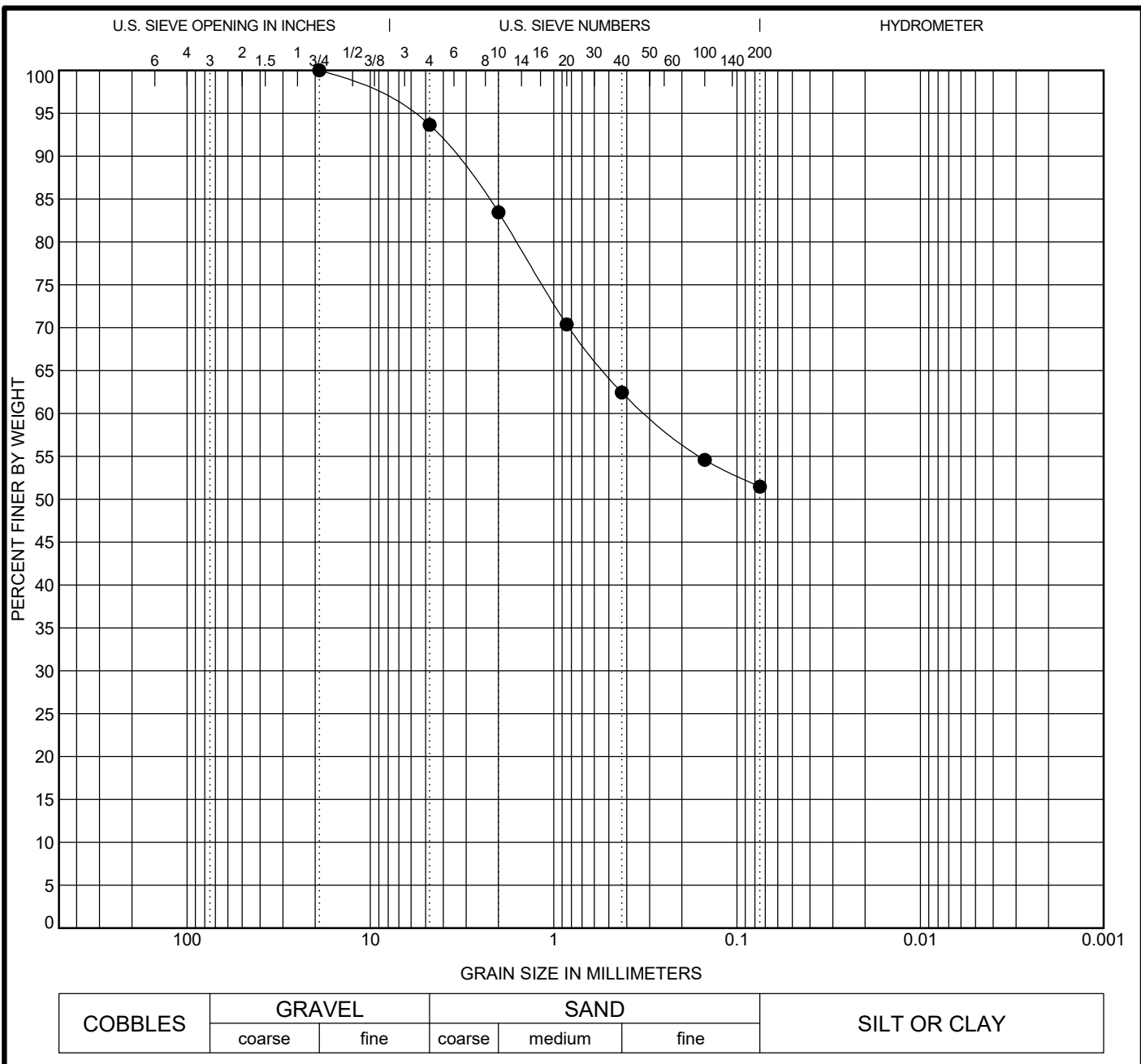


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ATTERBERG LIMITS TEST RESULTS - ASTM D4318

NEW ANIMAL QUARANTINE STATION (AQS)
 HALAWA, OAHU, HAWAII

Plate
B - 1



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Sample	Depth (ft)	Description	LL	PL	PI	Cc	Cu
● B-105	10.0-10.5	Grayish brown sandy silt (ML) with a little gravel					

Sample	Depth (ft)	D100 (mm)	D60 (mm)	D30 (mm)	D10 (mm)	%Gravel	%Sand	%Fine
● B-105	10.0-10.5	19	0.307			6.4	42.2	51.5

G GRAIN SIZE MOD 8052-10.GPJ GEOLABS.GDT 10/28/21

	GEOLABS, INC. GEOTECHNICAL ENGINEERING	GRAIN SIZE DISTRIBUTION - ASTM C117 & C136	
	W.O. 8052-10	NEW ANIMAL QUARANTINE STATION (AQS) HALAWA, OAHU, HAWAII	

Location	Depth (feet)	Soil Description	Dry Density (pcf)	Moisture Contents			Ring Swell (%)
				Initial (%)	Air-Dried (%)	Final (%)	
B-103*	1.0 - 2.5	Brown silty clay with some sand and a little gravel	67.6	18.5	13.5	47.5	0.0

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

* Relatively Undisturbed

** Remolded

G RING SWELL TEST 8052-10.GPJ GEOLABS.GDT 10/28/21



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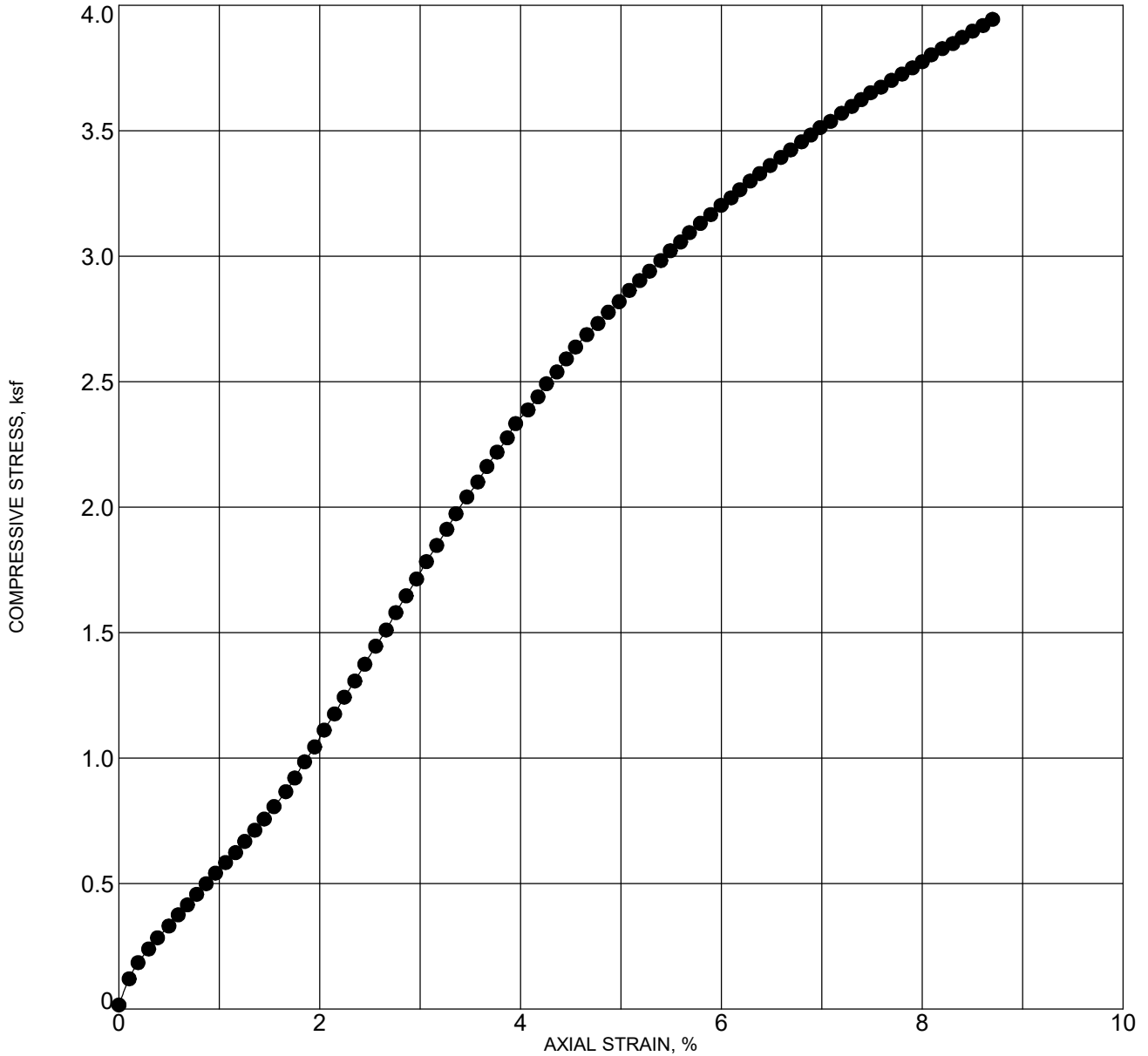
GEOTECHNICAL ENGINEERING

W.O. 8052-10

SUMMARY OF RING SWELL TESTS

NEW ANIMAL QUARANTINE STATION (AQS)
HALAWA, OAHU, HAWAII

Plate
B - 3



Unconfined Compressive Strength (ksf):	3.94
Axial Strain at Failure (%):	8.7
Strain Rate (% / minute):	1.00

Location: B-101
 Depth: 45.0 - 46.5 feet
 Description: Brown with grayish mottling silty clay with a little sand
 Test Date: 9/8/2021

Dry Density (pcf)	82.3	Sample Diameter (inches)	2.400
Moisture (%)	42.6	Sample Height (inches)	5.100

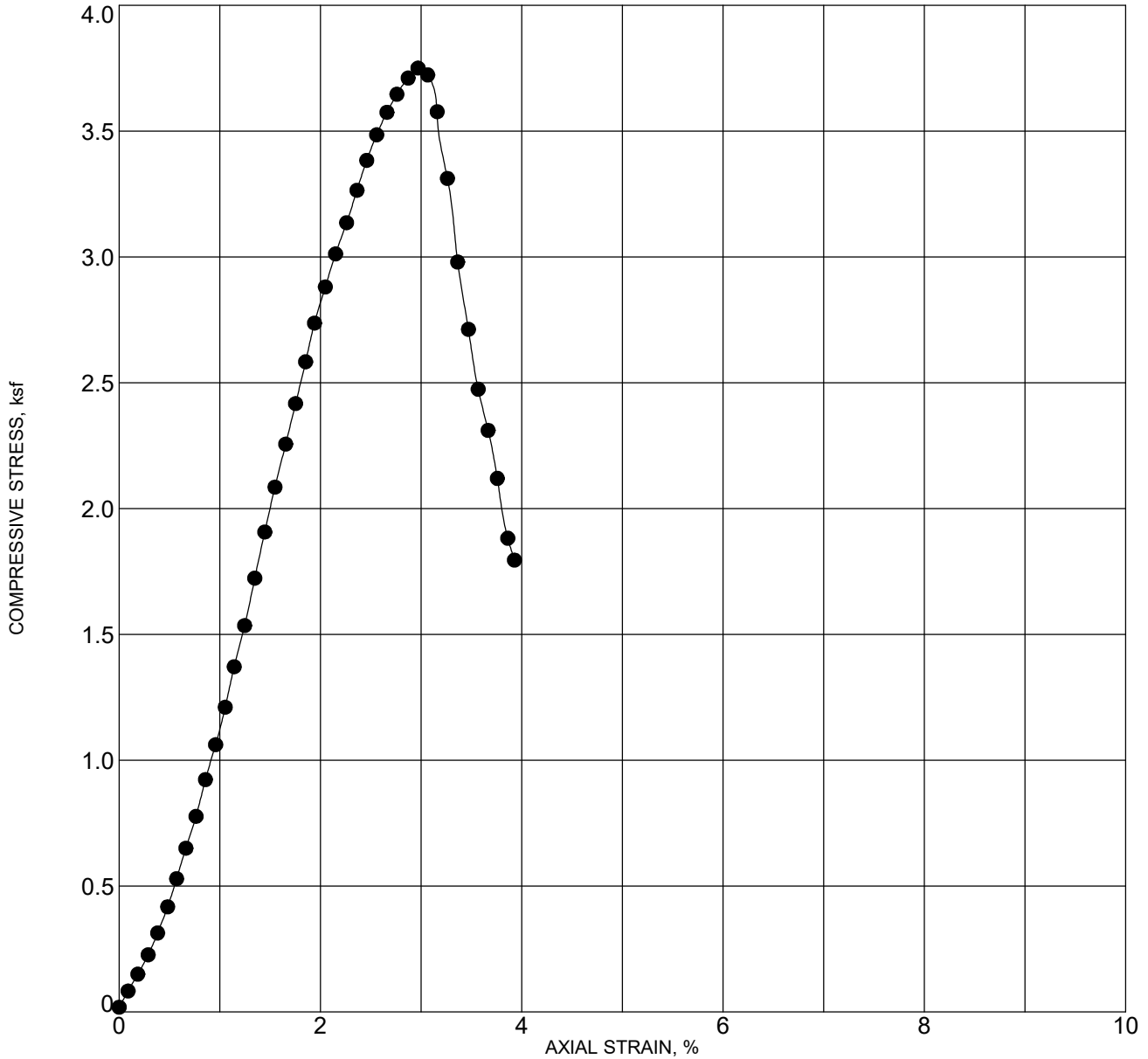


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UNCONFINED COMPRESSION TEST - ASTM D2166

NEW ANIMAL QUARANTINE STATION (AQS)
 HALAWA, OAHU, HAWAII

Plate
B - 4



Unconfined Compressive Strength (ksf):	3.75
Axial Strain at Failure (%):	3.0
Strain Rate (% / minute):	0.99

Location: B-105
 Depth: 15.0 - 16.5 feet
 Description: Brown with dark gray mottling clayey silt with a little sand
 Test Date: 9/8/2021

Dry Density (pcf)	72.4	Sample Diameter (inches)	2.400
Moisture (%)	41.1	Sample Height (inches)	5.100

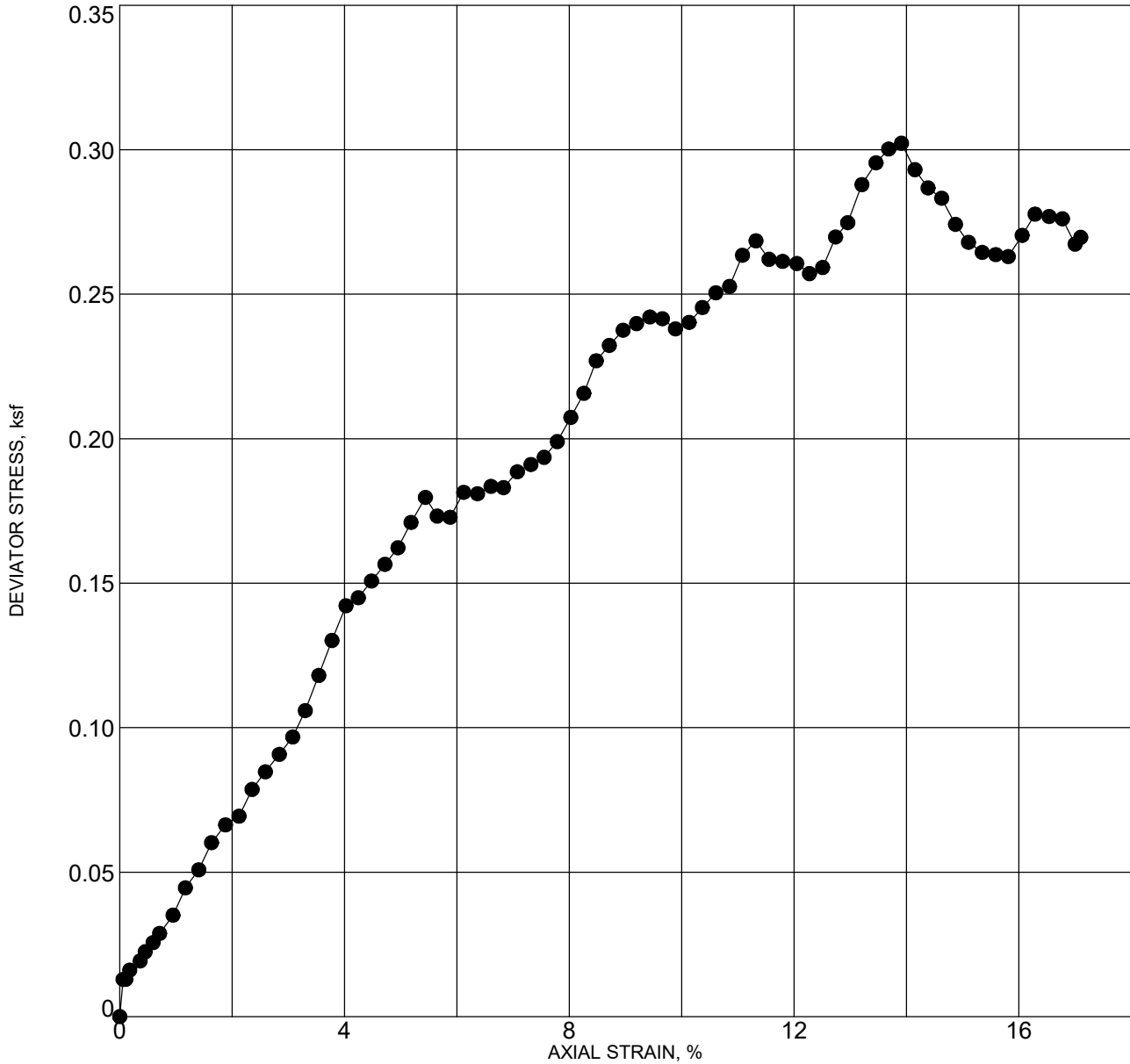


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UNCONFINED COMPRESSION TEST - ASTM D2166

NEW ANIMAL QUARANTINE STATION (AQS)
 HALAWA, OAHU, HAWAII

Plate
B - 5




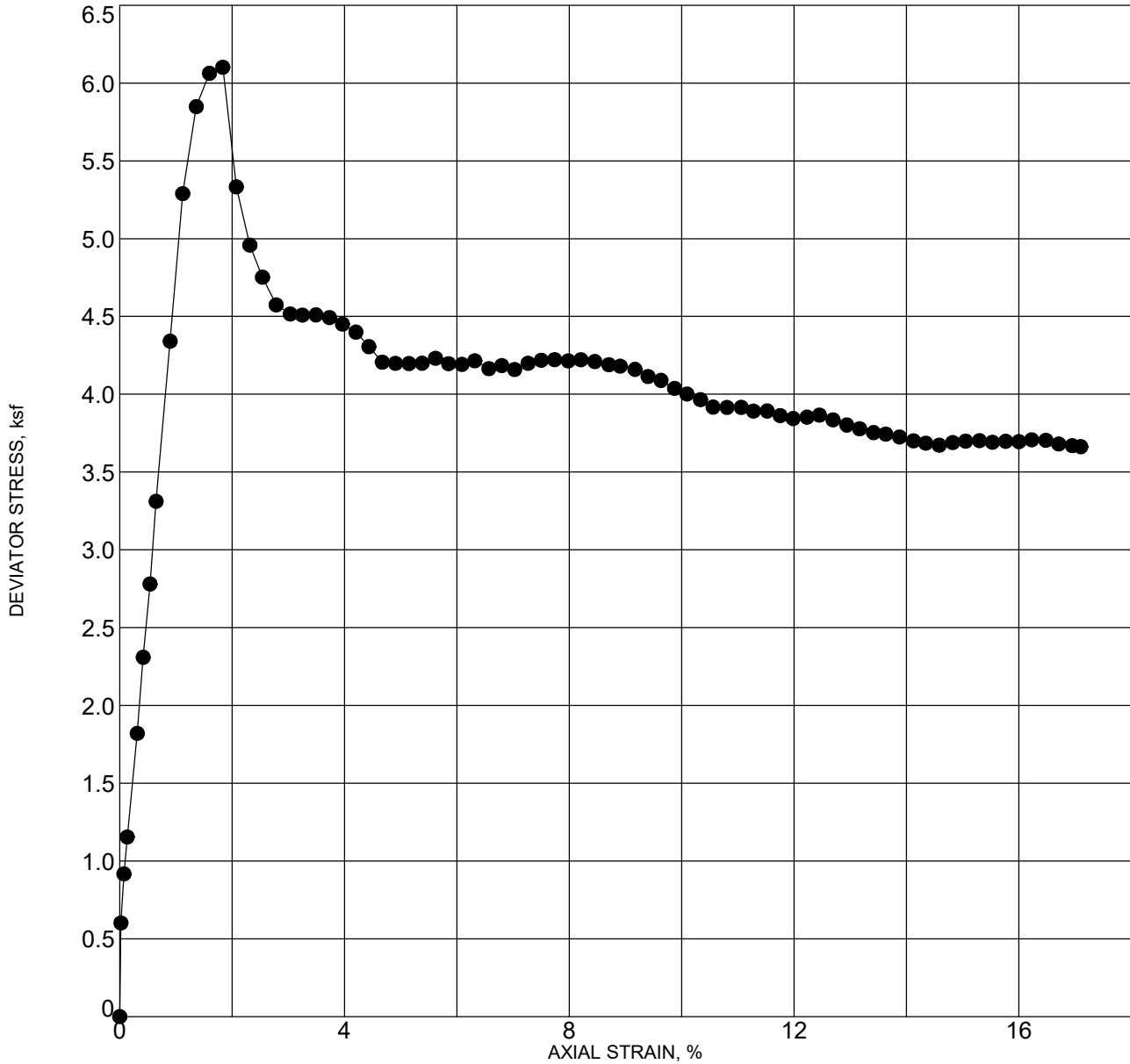
Max. Deviator Stress (ksf):	0.3
Confining Stress (ksf):	1.5

Location: B-101
 Depth: 15.0 - 16.5 feet
 Description: Orangish brown sandy clay with some gravel
 Test Date: 8/23/2021

Dry Density (pcf)	63.5	Sample Diameter (inches)	2.413
Moisture (%)	53.0	Sample Height (inches)	5.100
Axial Strain at Failure (%)	13.9	Strain Rate (% / minute)	0.71

G TXUU 8052-10.GPJ GEOLABS.GDT 10/28/21

	GEOLABS, INC. GEOTECHNICAL ENGINEERING	TRIAXIAL UU COMPRESSION TEST - ASTM D2850	
	W.O. 8052-10	NEW ANIMAL QUARANTINE STATION (AQS) HALAWA, OAHU, HAWAII	
			Plate B - 6




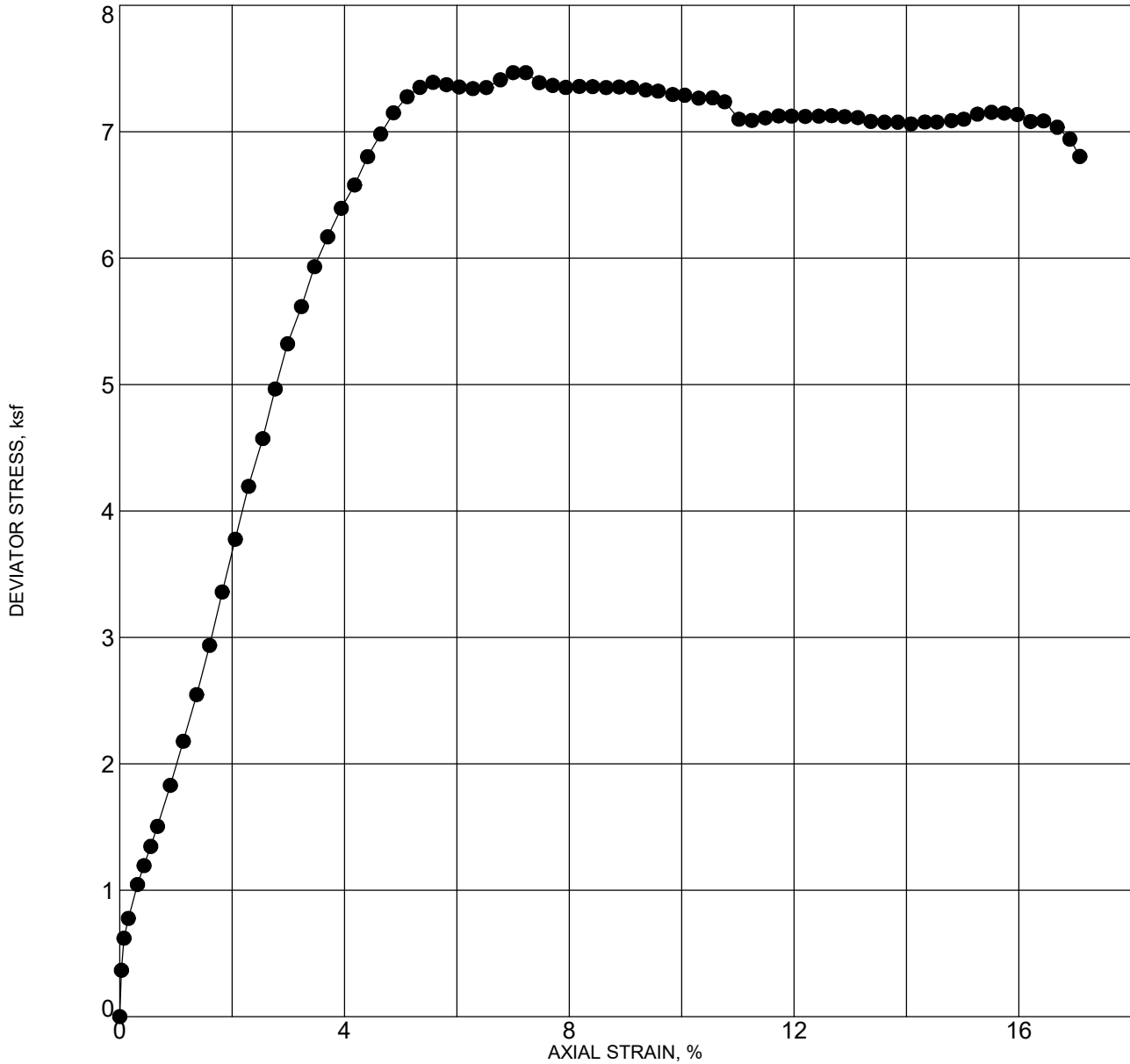
Max. Deviator Stress (ksf):	6.1
Confining Stress (ksf):	2.9

Location: B-102
 Depth: 25.0 - 26.5 feet
 Description: Brown with gray mottling silty clay with a little sand and some gravel
 Test Date: 8/23/2021

Dry Density (pcf)	73.1	Sample Diameter (inches)	2.413
Moisture (%)	40.3	Sample Height (inches)	5.100
Axial Strain at Failure (%)	1.8	Strain Rate (% / minute)	0.70

G TXUU 8052-10.GPJ GEOLABS.GDT 10/28/21

	GEOLABS, INC. GEOTECHNICAL ENGINEERING	TRIAXIAL UU COMPRESSION TEST - ASTM D2850	
	W.O. 8052-10	NEW ANIMAL QUARANTINE STATION (AQS) HALAWA, OAHU, HAWAII	
			Plate B - 7




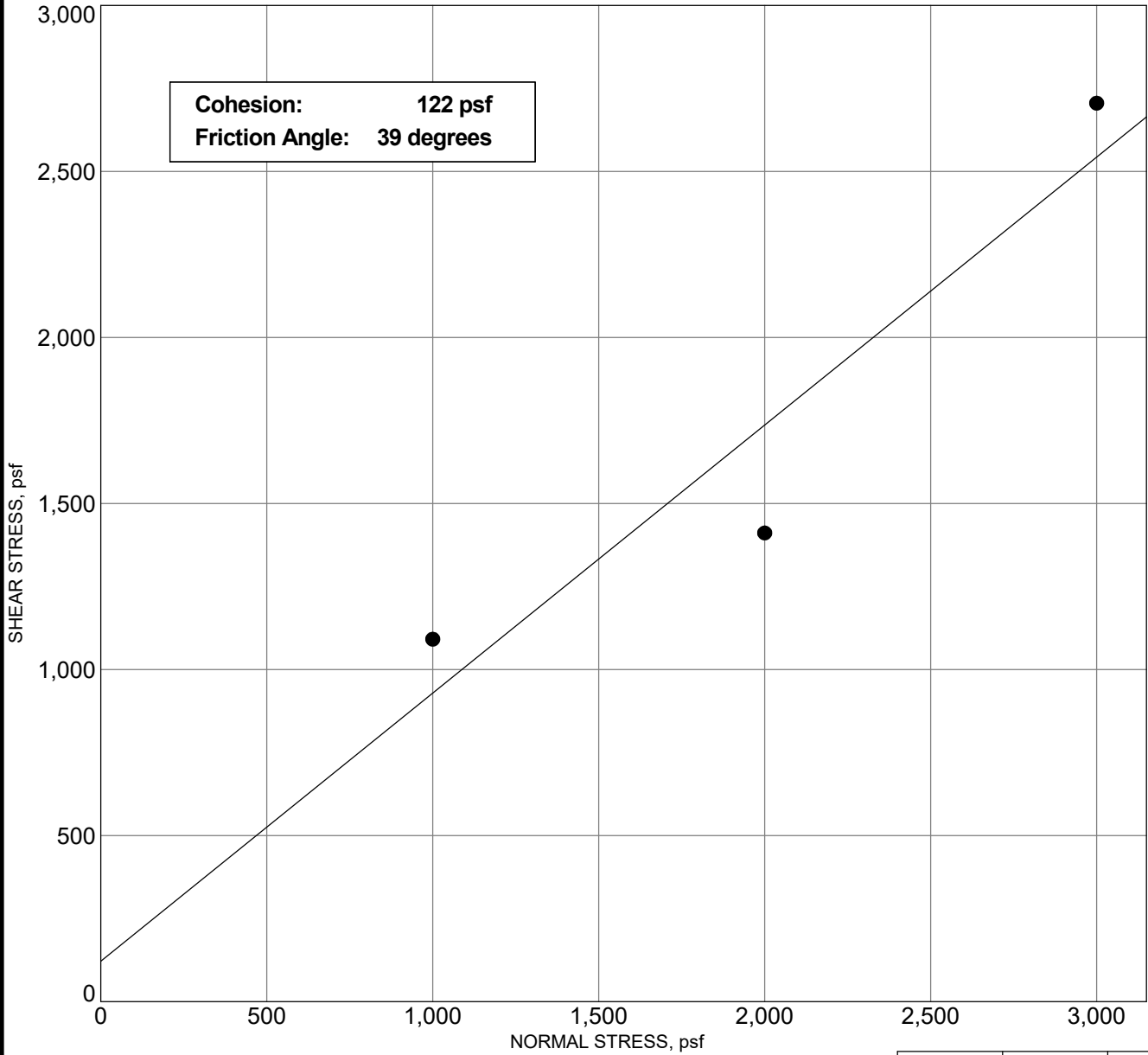
Max. Deviator Stress (ksf):	7.5
Confining Stress (ksf):	2.9

Location: B-103
 Depth: 25.0 - 26.5 feet
 Description: Grayish brown clayey silt with some sand and gravel
 Test Date: 8/23/2021

Dry Density (pcf)	78.0	Sample Diameter (inches)	2.413
Moisture (%)	42.8	Sample Height (inches)	5.100
Axial Strain at Failure (%)	7.2	Strain Rate (% / minute)	0.70

G TXUU 8052-10.GPJ GEOLABS.GDT 10/28/21

	GEOLABS, INC. GEOTECHNICAL ENGINEERING	TRIAXIAL UU COMPRESSION TEST - ASTM D2850	
	W.O. 8052-10	NEW ANIMAL QUARANTINE STATION (AQS) HALAWA, OAHU, HAWAII	
			Plate B - 8



		Sample #1	Sample #2	Sample #3
INITIAL	Moisture Content, %	24.7	24.5	24.6
	Dry Density, pcf	68.7	79.1	78.8
	Height, inches	1.00	1.00	1.00
FINAL	Moisture Content, %	43.7	39.3	38.4
	Dry Density, pcf	66.6	81.1	81.2
	Height, inches	1.031	0.975	0.971
Diameter, inches		2.42	2.42	2.42
Deformation Rate, inch/minute		0.0024	0.0021	0.0023
Normal Stress, psf		1000	2000	3000
Peak Shear Stress, psf		1091	1411	2706
Shear Displacement, inches		0.43	0.41	0.41

Sample: B-103
 Depth: 5.0 - 6.5 feet
 Description: Brown silty clay with some sand and a little gravel

G DIRECT SHEAR 8052-10.GPJ GEOLABS.GDT 10/25/21

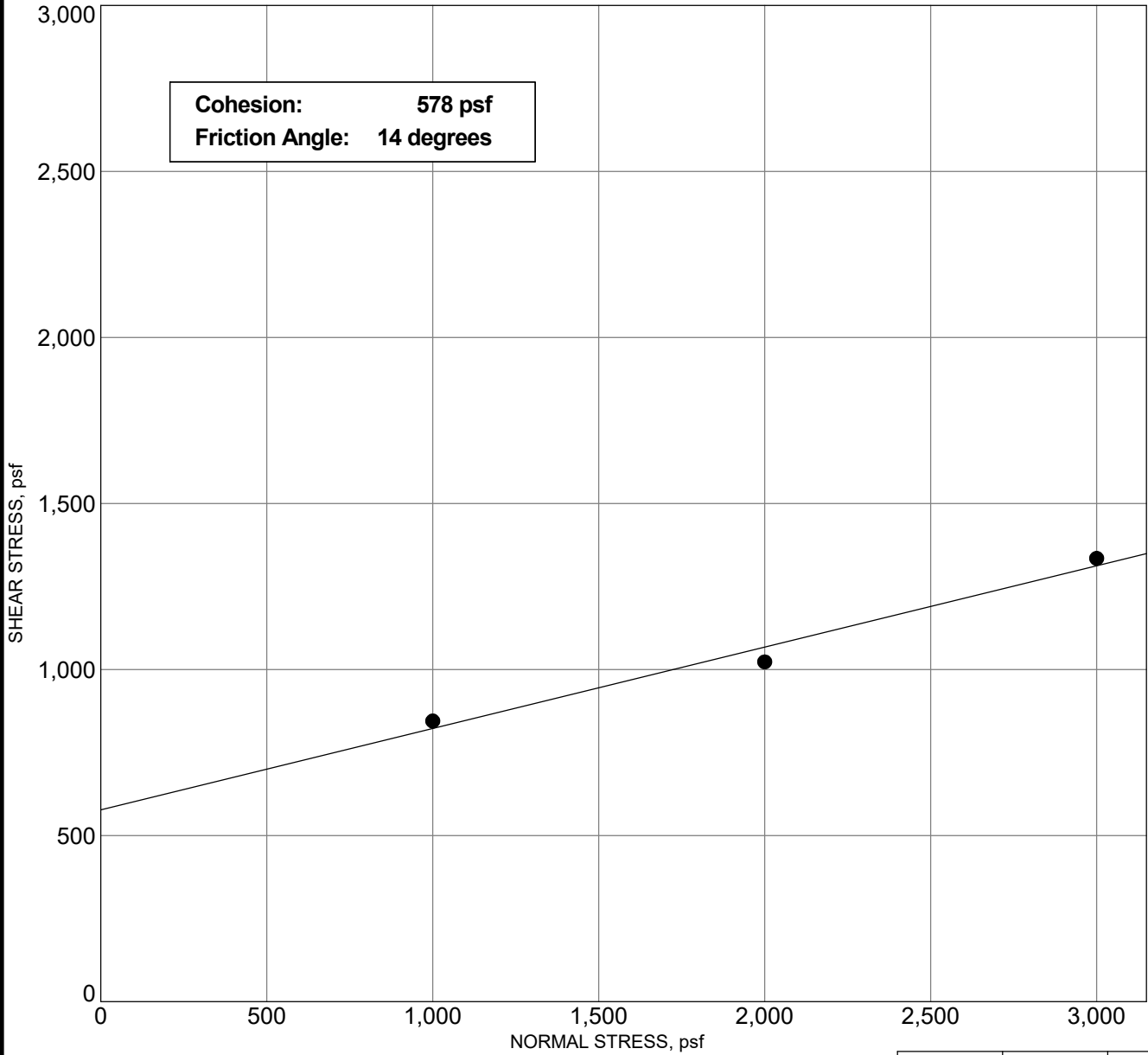


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 GEOTECHNICAL ENGINEERING
 W.O. 8052-10

DIRECT SHEAR TEST - ASTM D3080

NEW ANIMAL QUARANTINE STATION (AQS)
 HALAWA, OAHU, HAWAII

Plate
B - 9



		Sample #1	Sample #2	Sample #3
INITIAL	Moisture Content, %	18.8	17.9	19.0
	Dry Density, pcf	71.8	73.1	72.8
	Height, inches	1.00	1.00	1.00
FINAL	Moisture Content, %	37.9	36.0	34.2
	Dry Density, pcf	69.6	75.6	74.9
	Height, inches	1.032	0.967	0.972
Diameter, inches		2.42	2.42	2.42
Deformation Rate, inch/minute		0.0025	0.0023	0.0024
Normal Stress, psf		1000	2000	3000
Peak Shear Stress, psf		845	1023	1335
Shear Displacement, inches		0.43	0.42	0.42

Sample: B-105
 Depth: 1.0 - 2.3 feet
 Description: Grayish brown sandy silt with a little gravel

G DIRECT SHEAR 8052-10.GPJ GEOLABS.GDT 10/25/21

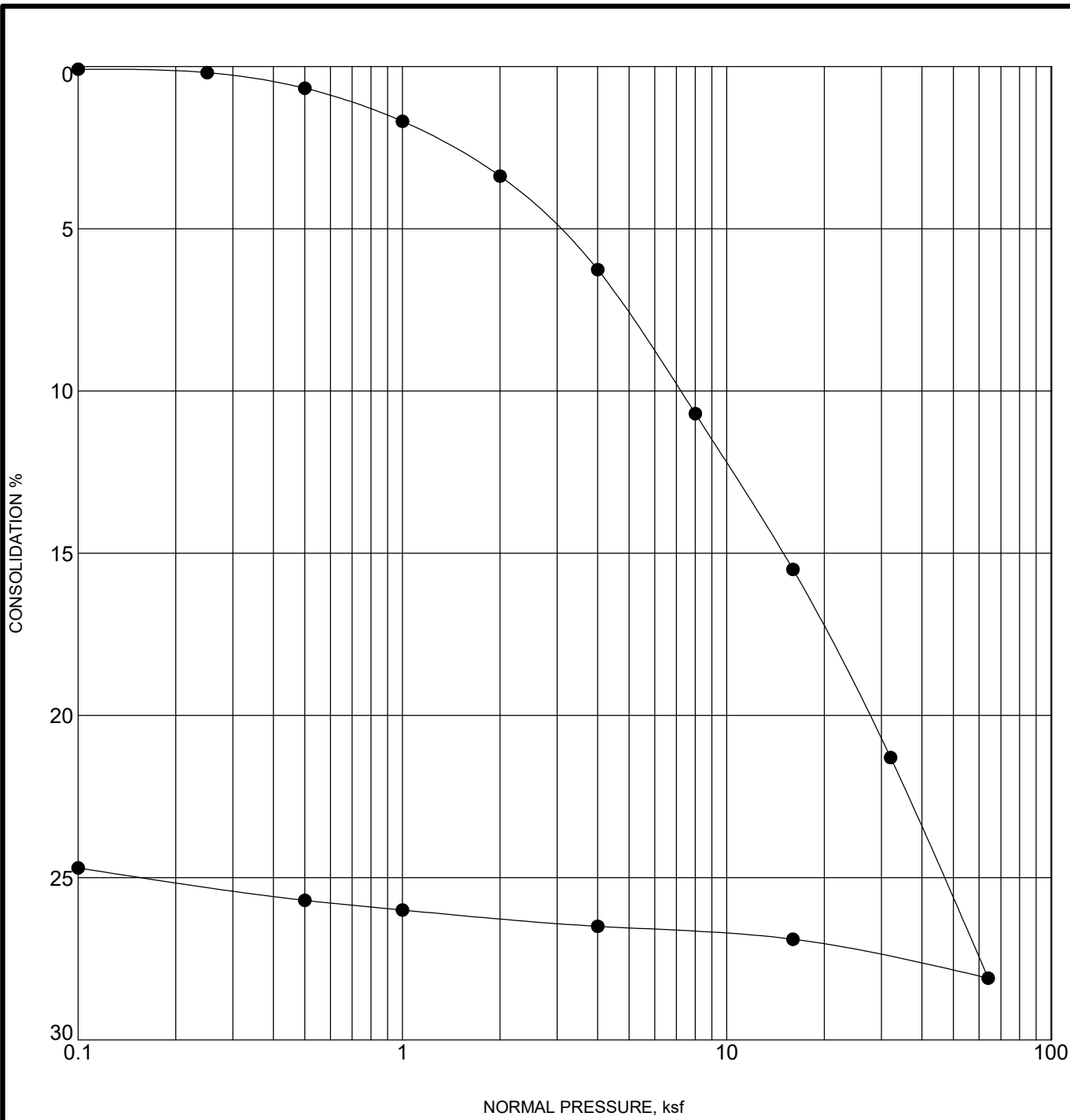


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 W.O. 8052-10

DIRECT SHEAR TEST - ASTM D3080

NEW ANIMAL QUARANTINE STATION (AQS)
 HALAWA, OAHU, HAWAII

Plate
B - 10



Sample: B-102
 Depth: 25.0 - 26.5 feet
 Description: Brown with gray mottling silty clay with a little sand and some gravel

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

	Initial	Final
Water Content, %	28.4	44.2
Dry Density, pcf:	62.0	82.4
Void Ratio	2.190	1.401
Degree of Saturation, %	41.1	100.0
Sample Height, inches	0.9800	0.7400

G CONSOL. 8052-10.GPJ GEOLABS.GDT 10/27/21

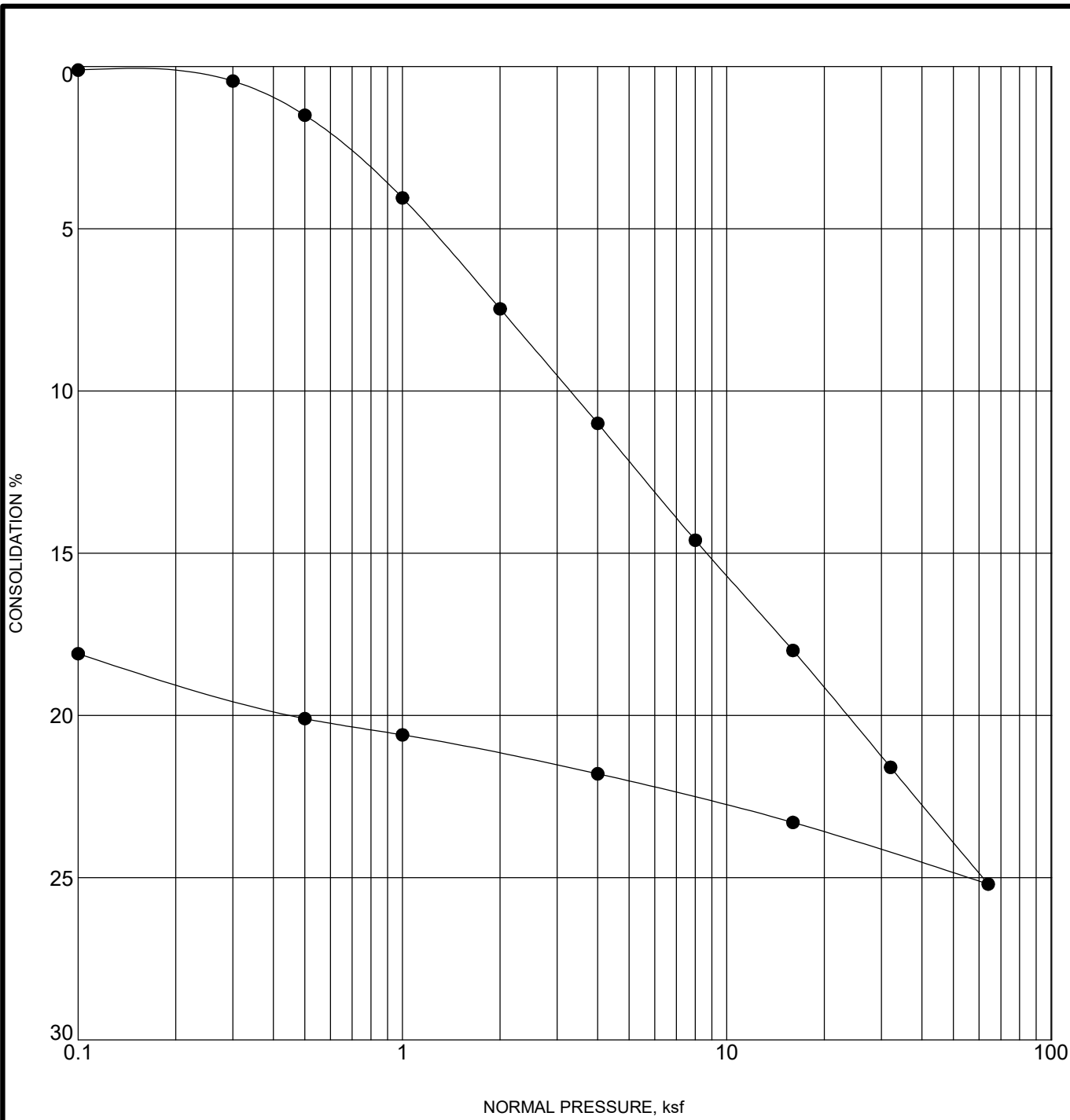


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CONSOLIDATION TEST - ASTM D2435

NEW ANIMAL QUARANTINE STATION (AQS)
 HALAWA, OAHU, HAWAII

Plate
B - 11



Sample: B-103
 Depth: 5.0 - 6.5 feet
 Description: Brown silty clay with some sand and a little gravel

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

	Initial	Final
Water Content, %	19.3	35.8
Dry Density, pcf:	77.7	94.9
Void Ratio	1.678	1.194
Degree of Saturation, %	38.3	100.0
Sample Height, inches	0.9900	0.8100

G CONSOL. 8052-10.GPJ GEOLABS.GDT 10/27/21

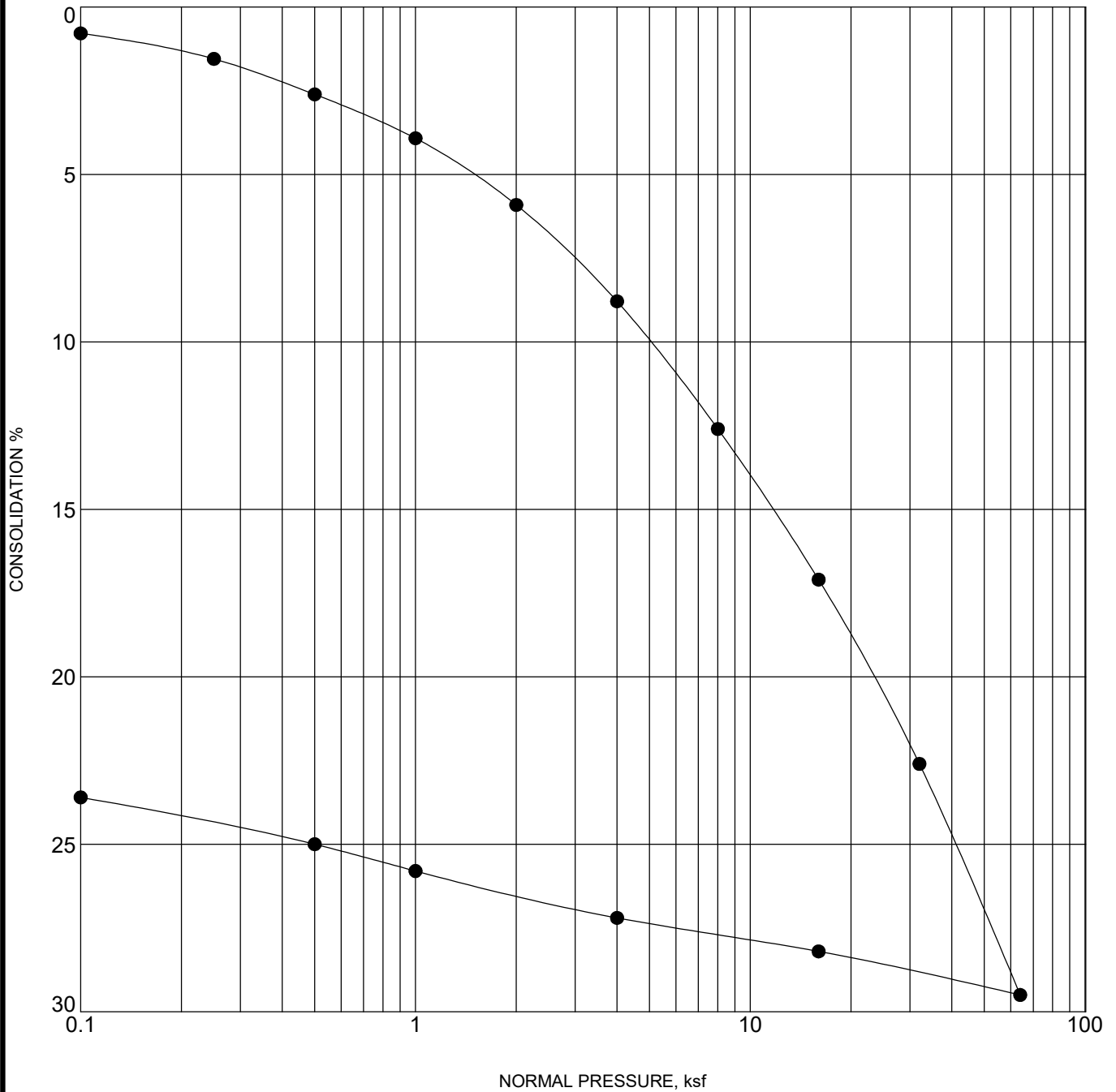


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CONSOLIDATION TEST - ASTM D2435

NEW ANIMAL QUARANTINE STATION (AQS)
 HALAWA, OAHU, HAWAII

Plate
B - 12



Sample: B-103
 Depth: 15.0 - 16.5 feet
 Description: Greenish brown clayey silt with some sand and gravel

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

	Initial	Final
Water Content, %	23.9	45.2
Dry Density, pcf:	63.3	82.9
Void Ratio	2.278	1.503
Degree of Saturation, %	34.9	100.0
Sample Height, inches	0.9800	0.7500

G CONSOL. 8052-10.GPJ GEOLABS.GDT 10/27/21



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CONSOLIDATION TEST - ASTM D2435

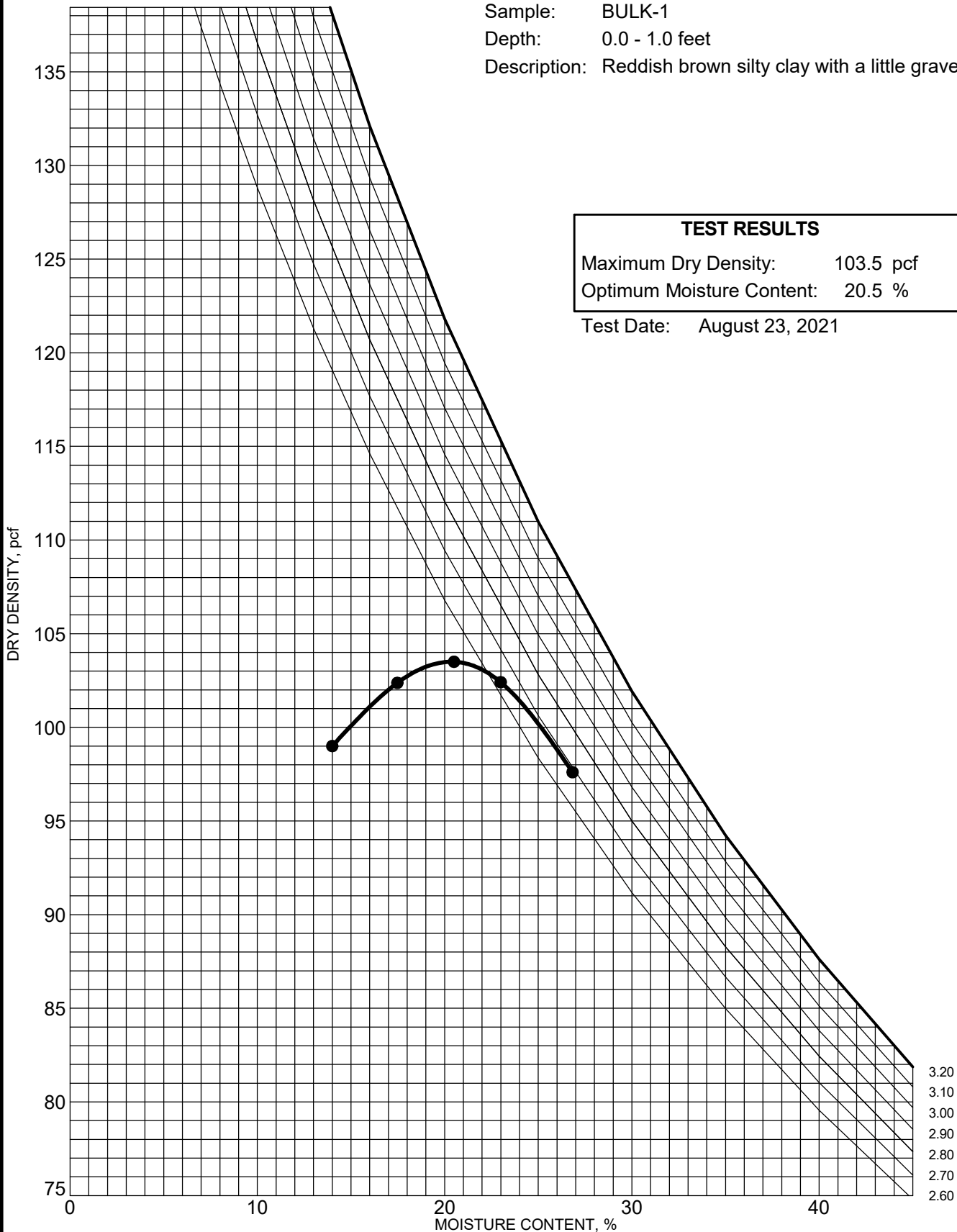
NEW ANIMAL QUARANTINE STATION (AQS)
 HALAWA, OAHU, HAWAII

Plate
B - 13

Sample: BULK-1
 Depth: 0.0 - 1.0 feet
 Description: Reddish brown silty clay with a little gravel

TEST RESULTS
 Maximum Dry Density: 103.5 pcf
 Optimum Moisture Content: 20.5 %

Test Date: August 23, 2021



G COMPACTON 8052-10.GPJ GEOLABS.GDT 10/28/21



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MOISTURE-DENSITY RELATIONSHIP - ASTM D1557 C

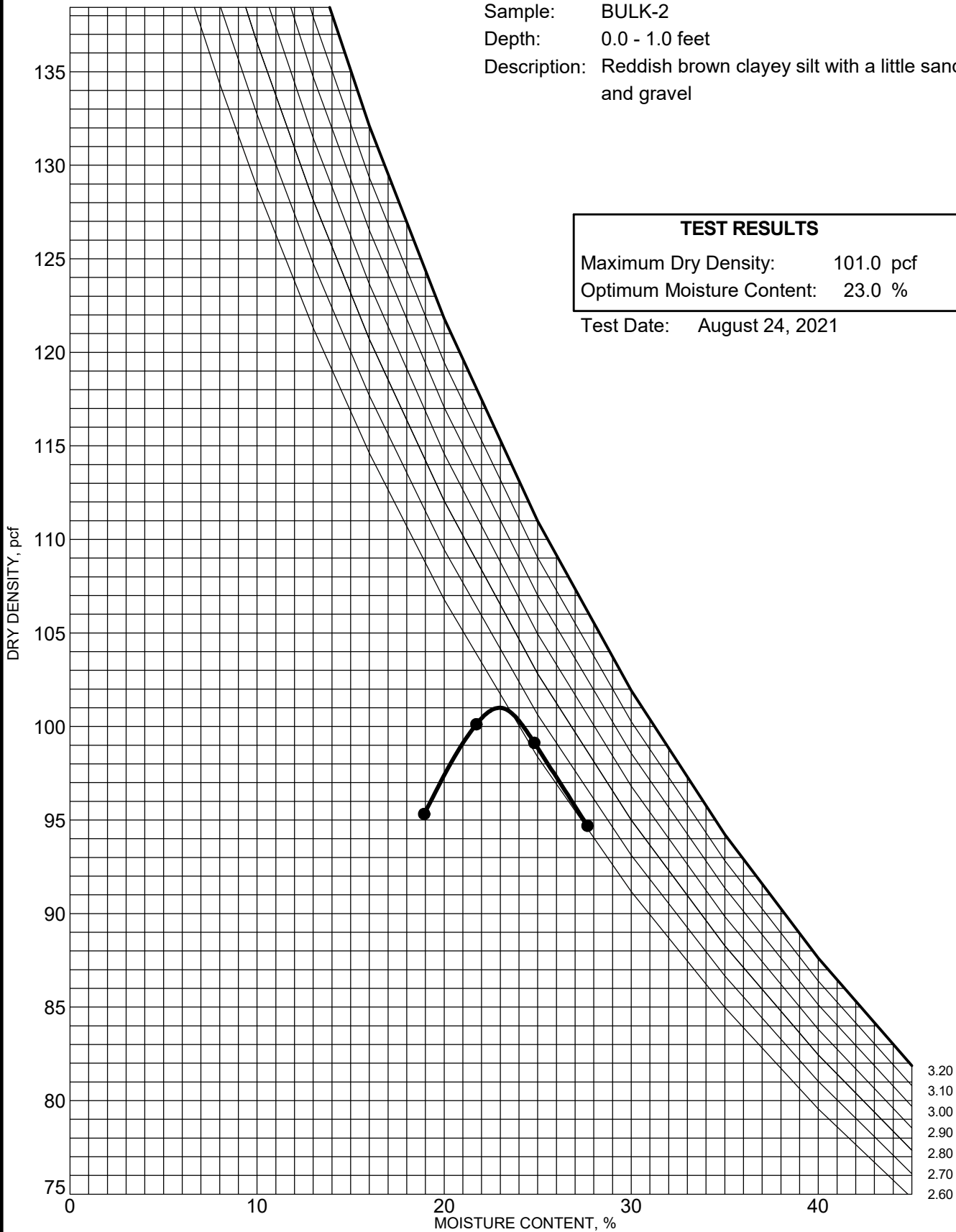
NEW ANIMAL QUARANTINE STATION (AQS)
 HALAWA, OAHU, HAWAII

Plate
B - 14

Sample: BULK-2
 Depth: 0.0 - 1.0 feet
 Description: Reddish brown clayey silt with a little sand and gravel

TEST RESULTS
 Maximum Dry Density: 101.0 pcf
 Optimum Moisture Content: 23.0 %

Test Date: August 24, 2021



G COMPACTON 8052-10.GPJ GEOLABS.GDT 10/25/21

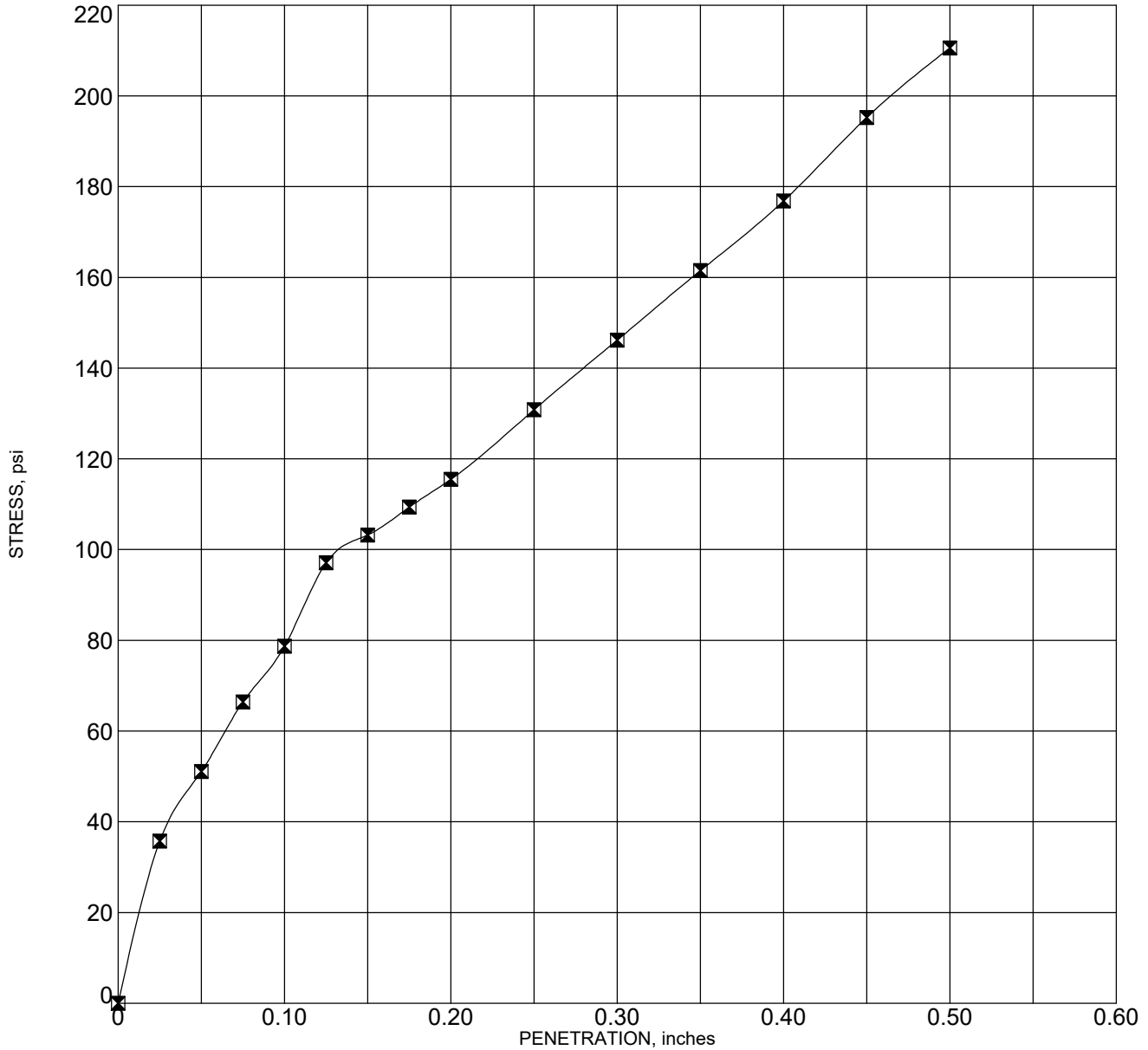


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 GEOTECHNICAL ENGINEERING
 W.O. 8052-10

MOISTURE-DENSITY RELATIONSHIP - ASTM D1557 A

NEW ANIMAL QUARANTINE STATION (AQS)
 HALAWA, OAHU, HAWAII

Plate
B - 15



Sample: BULK-1
 Depth: 0.0 - 1.0 feet
 Description: Reddish brown silty clay with a little gravel

Corr. CBR @ 0.1"	7.9
Corr. CBR @ 0.2"	7.7
Swell (%)	1.37

Molding Dry Density (pcf)	100.5	Hammer Wt. (lbs)	10
Molding Moisture (%)	24.8	Hammer Drop (inches)	18
Days Soaked	4	No. of Blows	56
Aggregate	3/4 inch minus	No. of Layers	5

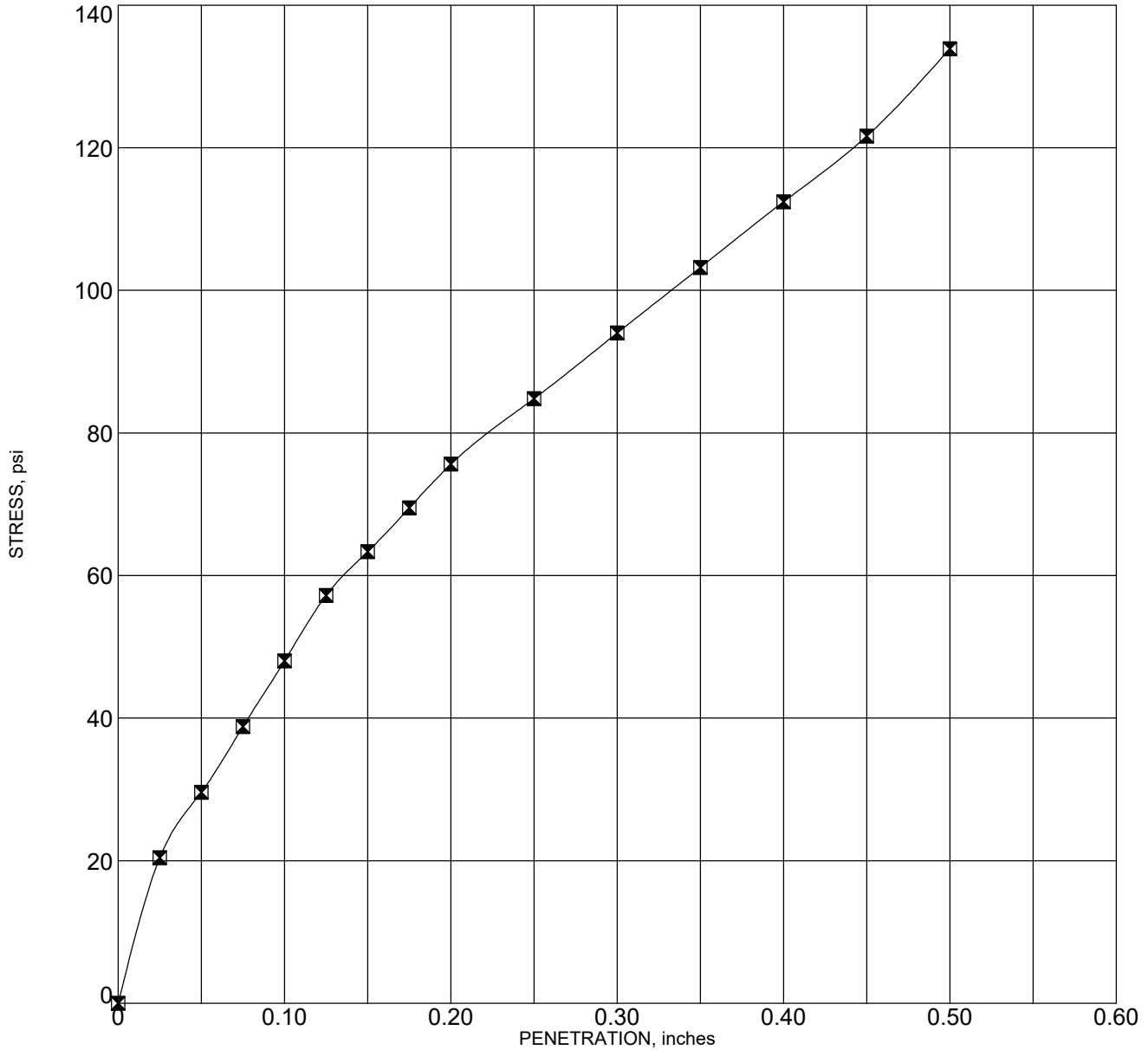


GEOLABS, INC.
 GEOTECHNICAL ENGINEERING
 W.O. 8052-10

CALIFORNIA BEARING RATIO - ASTM D1883

NEW ANIMAL QUARANTINE STATION (AQS)
 HALAWA, OAHU, HAWAII

Plate
B - 16



Corr. CBR @ 0.1"	4.8
Corr. CBR @ 0.2"	5.0
Swell (%)	0.46

Sample: BULK-2
 Depth: 0.0 - 1.0 feet
 Description: Reddish brown clayey silt with a little sand and gravel

Molding Dry Density (pcf)	96.5	Hammer Wt. (lbs)	10
Molding Moisture (%)	26.2	Hammer Drop (inches)	18
Days Soaked	4	No. of Blows	56
Aggregate	3/4 inch minus	No. of Layers	5

G. CBR. 8052-10.GPJ GEOLABS.GDT. 10/25/21

	GEOLABS, INC. GEOTECHNICAL ENGINEERING	CALIFORNIA BEARING RATIO - ASTM D1883	
	W.O. 8052-10	NEW ANIMAL QUARANTINE STATION (AQS) HALAWA, OAHU, HAWAII	
			Plate B - 17

APPENDIX C

APPENDIX C

Field Infiltration Test

As part of our field exploration program, we performed one infiltration test in Boring No. 107 at the approximate location shown on the Site Plan, Plate 2. The test was performed in general accordance with the procedures in Appendix D of the State of Maryland, Department of the Environment “Stormwater Design Manual, Volumes I and II” (rev. 2009). These procedures are consistent with other state’s procedures and may generally be considered an industry standard.

The field infiltration test was performed by advancing the boring to the selected test depth of about 5 feet below the existing ground surface. Upon reaching the test depth, a solid casing was set to the bottom of the drilled hole to allow infiltration only through the soil exposed on the bottom of the boring. Falling head infiltration tests were performed to determine the infiltration rate of the underlying subsurface materials. The test consisted of four trials of filling the casing with 24 inches of water and taking periodic readings over a one-hour trial period or until the water in the casing is drained. The test results are presented on Plate C-1.