GEOTECHNICAL ENGINEERING EXPLORATION
ALA MOANA BOULEVARD
ELEVATED PEDESTRIAN WALKWAY
FEDERAL AID PROJECT NO. BLD-092-1(029)
HONOLULU, OAHU, HAWAII

W.O. 8115-00 JUNE 1, 2021

Prepared for

VICTORIA WARD, LTD.

GEOLABS, INC.
Geotechnical Engineering and Drilling Services

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VICTORIA WARD, LTD.

ROBIN M. LIM
No. 8436-C

THIS WORK WAS PREPARED BY
ME OR UNDER MY SUPERVISION.

SIGNATURE

4-30-22
EXPIRATION DATE
OF THE LICENSE

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GEOLABS, INC.
Geotechnical Engineering and Drilling Services

June 1, 2021
W.O. 8115-00

Mr. Lee Cranmer
Victoria Ward, Ltd.
1240 Ala Moana Boulevard, Suite 200
Honolulu, HI 96814

Dear **Mr. Cranmer:**

Geolabs, Inc. is pleased to submit our report entitled “Geotechnical Engineering Exploration, Ala Moana Boulevard Elevated Pedestrian Walkway, Federal Aid Project No. BLD-092-1(029), Honolulu, Oahu, Hawaii” prepared in support of the design of the proposed elevated walkway project.

Our work was performed in general accordance with the scope of services outlined in our revised fee proposal dated April 15, 2020 and the Work Order Agreement entered into on April 24, 2020.

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

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

Very truly yours,

GEOLABS, INC.

Robin M. Lim, P.E.
President

RML:AH:rl

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SUMMARY OF FINDINGS AND RECOMMENDATIONS

Our field exploration generally encountered surface fill materials placed over lagoonal deposits overlying coralline deposits extending to the maximum depth explored of about 122.5 feet below the existing ground surface. We encountered groundwater in the drilled borings at depths ranging from about 3 to 5.5 feet below the existing ground surface. The groundwater levels encountered generally correspond to between about Elevations -1 and +2.5 feet MSL at the time of our field exploration.

Generally, we anticipate the soft and/or loose subsurface conditions underlying the project site and the relatively heavy structural load demands will require supporting the new pedestrian bridge on a deep foundation system, such as cast-in-place concrete drilled shafts. The drilled shaft foundations would extend below the surface fills and soft and/or loose lagoonal deposits and derive support principally from adhesion between the drilled shaft and the dense/hard coralline deposits encountered at greater depths. Based on the structural load demands provided for our engineering analyses, a drilled shaft diameter of 36 inches and embedment length of 60 feet may be used for design of the new bridge abutment foundations. Due to the heavier loading at the center pier of the bridge, the center pier foundations should consist of drilled shafts of 48 inches in diameter and an embedment length of 75 feet.

Based on the grading plans, we understand significant fills of up to about 20 feet in height will be required to meet the finished grade elevations of the new pedestrian walkway on the mauka and makai sides of the bridge. Appreciable ground settlements are anticipated when substantial new fills are placed over the existing ground underlain by soft silts and loose sands to raise the site to the proposed finished grades. Based on the subsurface conditions encountered and the planned fill thicknesses, we estimate potential filled ground settlements on the order of about 12 to 17 inches could occur at the planned mauka and makai embankments of the project.

Because the makai abutment is adjacent to Kewalo Basin and consists of paved areas with existing underground utilities, the estimated amount of ground settlement will impact the existing underground utilities in this area adversely and will change the drainage patterns of the paved areas. Therefore, we recommend supporting the makai abutment and makai ramp embankments on jet grout columns to reduce the amount of ground settlement at this location and to improve slope stability. We also recommend supporting the abutment fill immediately behind the mauka abutment on jet grout columns to allow for construction of the abutment structure concurrently with the abutment fill.

As for the mauka walkway embankments, we recommend building the walkway embankments and surcharging the fill area to accelerate the settlements. We also recommend implementing a ground settlement monitoring program to confirm the actual settlement rate prior to construction of the utilities embedded in the filled ground and the on-grade improvements on top of the fill. Construction of the walkway embankments will involve constructing Geosynthetically Reinforced Soil (GRS) Embankments to allow construction of the embankments with geosynthetics to improve the embankment stability and to allow ground settlements to occur.

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

END OF SUMMARY OF FINDINGS AND RECOMMENDATIONS

SECTION 1. GENERAL

This report presents the results of our geotechnical engineering exploration and engineering analyses performed in support of the design of the proposed *Ala Moana Boulevard Elevated Pedestrian Walkway* project in the Kaka`ako area of Honolulu on the Island of Oahu, Hawaii. The project location and general vicinity are shown on the Project Location Map, Plate 1.

This report summarizes the findings and presents our geotechnical recommendations based on our field explorations, laboratory testing, and engineering analyses. The recommendations presented herein are intended for the design of foundations, site grading, geosynthetic reinforced soil (GRS) embankments, soil stabilization, and retaining structures only. The findings and recommendations presented herein are subject to the limitations noted at the end of this report.

1.1 Project Considerations

The new Ala Moana Boulevard Elevated Pedestrian Walkway is located along Ala Moana Boulevard about halfway between the intersection with Ward Avenue and the intersection with Kamakee Street in the Kaka`ako area of Honolulu on the Island of Oahu, Hawaii.

The new pedestrian bridge will be a unique signature pedestrian bridge structure spanning approximately 120 feet in the north-south direction from Victoria Ward, Ltd. property across Ala Moana Boulevard to the existing landscaped green belt fronting Kewalo Basin. The new elevated walkway will connect to a pathway atop an embankment running in the east-west direction along the green belt on the makai side of Ala Moana Boulevard for a distance of approximately 300 feet.

We understand the bridge will be approximately 18 feet in width, and the alignment meanders along the east-west directions. We anticipate two columns will be used to support the new pedestrian bridge center pier structure, and we anticipate using a deep foundation system consisting of drilled shafts to support the bridge columns following FHWA guidelines for design of bridge foundations. In addition, we anticipate earth retaining structures, such as Geosynthetic Reinforced Soil (GRS) retaining structures, may be used for the ramps and abutments of the new bridge structure. Design of the new pedestrian bridge structure will follow the State of Hawaii – Department of Transportation (HDOT) bridge design guidelines using the latest version of the American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) Design Specifications (2020) along with the State of Hawaii, Department of Transportation (HDOT) amendments entitled "Design Criteria for Bridges and Structures" dated August 8, 2014.

1.2 Purpose and Scope

The purpose of our field exploration was to obtain an overview of the surface and subsurface conditions to develop a generalized subsurface data set to formulate geotechnical recommendations for the design of bridge foundations, site grading, soil stabilization, and retaining structures only. The scope of work for this exploration included the following tasks and work efforts:

1. Research and review of the readily available soil and geologic information related to the project area.
2. Compile the available subsurface information and engineering properties of the subsurface geomaterials to perform engineering analyses in support of the elevated walkway design.
3. Application and coordination of a State excavation permit and utility clearance with the applicable agencies (including Hawaii's One-Call Center) by our staff. Underground utility toning by our geologist and engineer at each borehole location prior to drilling.
4. Preparation and submittal of a traffic control plan in support of our field exploration activities on the highway.
5. Preparation and submittal of an Accident Prevention Plan with activity hazard analyses in support of our field exploration activities only and briefing of our field personnel of the plan.
6. Mobilization and demobilization of a truck-mounted drill rig, water truck, and two operators to the project site and back.
7. Drilling and sampling of six exploratory borings extending to depths of about 66.5 to 122.5 feet below the existing ground surface for a total of about 587 linear feet of exploration. In addition to the six borings, two additional borings, designated as Boring Nos. 2A and 2B, were drilled and

sampled to depths of about 42 and 11.5 feet, respectively, within the Ala Moana Boulevard median to further explore a buried concrete obstruction.

8. Performance of one seismic shear wave velocity profiling test extending to a depth of about 111.4 feet below the existing ground surface to determine the shear wave velocities of the subsurface materials and evaluate the seismic site classification at the project site.

9. Provision of traffic control devices and signs during the geotechnical field exploration activities.

10. Coordination of the field exploration and logging of the exploratory borings by our geologist.

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

12. Analyses of the field and laboratory data for the project to formulate geotechnical recommendations for design of the bridge foundations, site grading, settlement monitoring, GRS embankment, deep soil stabilization, and retaining structures.

13. Preparation of this formal geotechnical engineering report summarizing our work on the project and presenting the findings and our geotechnical recommendations for design.

14. Coordination of our overall work on the project by our project engineer.

15. Quality assurance of our work on the project by our principal engineer.

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

17. 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 seismic shear wave velocity profiling is presented in Appendix B. The laboratory test results of selected soil and core samples obtained from our field exploration are presented in Appendix C. Photographs of the core samples retrieved from our field exploration are presented in Appendix D.

END OF GENERAL

SECTION 2. SITE CHARACTERIZATION

Of interest to our geotechnical analysis for foundation design of the new pedestrian bridge structure and the geotechnical aspects of the elevated pedestrian walkway project 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 subsections provide a description of the geologic setting of the project site, the surface and subsurface conditions encountered at the site, the groundwater conditions encountered, and a discussion on the items needed for seismic design, such as soil liquefaction, soil profile for the elastic response spectrum, etc.

2.1 Regional Geology

The Island of Oahu is composed largely of the weathered remnants of two extinct shield volcanoes - Waianae and Koolau. The older Waianae Volcano forms the bulk of the western third of the island while the younger Koolau Volcano forms the majority of the eastern two-thirds of the island. It is believed that Waianae Volcano became extinct while Koolau Volcano was still active, and its eastern flank is partially buried below Koolau lavas in Central Oahu.

The project site is on the southern flank of Koolau Volcano, and its geomorphology and subsurface conditions are directly related to the glacio-eustatic fluctuations of the sea level during the Pleistocene Epoch and the genesis of the Honolulu Coastal Plain. The subsurface conditions in the Kaka`ako area are dominated by fossil Pleistocene Age coral/algal reef deposits and related deposits laid during high stands of the sea, which are intercalated with terrigenous sediments. The base of the stratigraphic section in this area is Koolau Basalt.

During the later Pleistocene Epoch (Ice Age), there were many sea level changes as a result of widespread glaciation in the continental areas of the world. These glacio-eustatic fluctuations resulted in stands of the sea that were both higher and lower relative to the present sea level on Oahu. About 15,000 years ago, a relatively rapid rise in sea level occurred. During that rise, valleys in the project area were drowned. In the last 10,000 years or so, the sea level has adjusted to its present stand.

The higher sea level stands caused the accumulation of deltas and fans of terrigenous sediments in the heads of old bays, accumulation of reef deposits at correspondingly higher elevations, and deposition of lagoonal/marine sediments in the quiet waters protected by fringing reefs. Subaerial exposure of the sediments and calcareous materials caused desiccation of the soft deltaic materials and lagoonal deposits and induration of the calcareous reef materials. The lower sea stands caused streams to carve valleys in the sediments and reef deposits. Subaerial erosion of the upper areas of the volcanic dome deposited terrigenous alluvial soils under relatively high energy conditions within and along streams. During periods of no significant sea level changes, continued stream action extended the alluvial deltas and fans seaward and deposited alluvium over the lagoonal sediments.

In the early part of the twentieth century, the Kaka`ako area consisted of low, marshy areas. As the City of Honolulu grew and the Kaka`ako area was urbanized, man-made fills were placed to reclaim the marshy areas and lagoons. The fill placed upon lagoonal deposits generally consisted of silty sands and gravel with coral fragments. Land development and reclamation projects within the last 100 years have brought the Kaka`ako area to its present form. Many of the resulting fills are of poor quality in terms of supporting heavy structural loads.

2.2 <u>Site Description</u>

The project site is located along Ala Moana Boulevard about halfway between the intersection with Ward Avenue and the intersection with Kamakee Street in the Kaka’ako area of Honolulu on the Island of Oahu, Hawaii. The project site consists of three major areas:

1. Mauka Abutment
2. Center Pier
3. Makai Abutment

The future mauka embankment is located at the open lot owned by Victoria Ward, Ltd. north of Ala Moana Boulevard. The lot consists primarily of lawn space and asphaltic concrete paved parking both commonly used for on-going construction in the area and farmer's market. Underlying the lot, an existing drainage culvert exists.

The future center pier is located within the existing Ala Moana Boulevard median. The median generally is covered by grass and trees spaced at approximately 80 feet along the boulevard alignment. Based on our field observations and the topographic plans provided, we anticipate some of the existing subsurface utilities will need to be relocated for the center pier construction.

The future makai embankment is located at the existing landscaped green belt and parking lot for Kewalo Basin. The landscaped green belt appears to have been built up by about 2 to 3 feet in reference to the Ala Moana Boulevard pavement surface.

Based on the topographic survey map provided and our field observations, the project site is generally flat with existing ground elevations between about +3 and +6 feet MSL (Mean Sea Level datum) with the exception of the existing landscaped green belt fronting Kewalo Basin. The landscaped green belt slopes down to Ala Moana Boulevard at approximately a 5H:1V slope with elevations between approximately +3 and +7.5 feet MSL.

2.3 Subsurface Conditions

Our field exploration consisted of drilling and sampling six borings extending to depths between approximately 66.5 and 122.5 feet below the existing ground surface. Based on the subsurface conditions encountered in the borings, the project site generally is underlain by surface fill materials placed over lagoonal deposits overlying coralline deposits. The surface fills encountered generally consisted of very soft to medium stiff clayey/silty soils and very loose to medium dense sands and gravel of varying silt content extending to depths of about 5 to 13.5 feet below the existing ground surface. It should be noted that Boring Nos. 1, 2A, 2B, and 3 through 5 were drilled and sampled in the archaeological trenches, which consisted of new backfill materials extending to the groundwater level at approximately 3 to 5.5 feet below the existing ground surface. It should be noted that the relative density of archaeological trench backfill materials may not be representative of the on-site fill materials present at the project site.

During the drilling of the median boring, designated as Boring No. 2, a concrete obstruction of about 6 feet in thickness was encountered starting at about 7 feet below the existing ground surface. Two additional probes, designated as Boring Nos. 2A and 2B, also were drilled within the median to further investigate the concrete obstruction. Boring No. 2A encountered the concrete obstruction between about 9 and 14 feet below the existing ground surface whereas Boring No. 2B encountered the concrete obstruction starting at 8.5 feet and extending to the maximum depth explored of about 11.5 feet below the existing ground surface. Based on the depth and thickness of the concrete obstructions, it is suspected that an abandoned pier deck may have been located beneath the Ala Moana Boulevard median.

Lagoonal deposits were encountered underlying the surface fill layer extending to depths of about 25 to 31.5 feet below the existing ground surface. The lagoonal deposits generally consisted of very loose to medium dense silty sands and gravel, and very soft to medium stiff sandy silts and clays. It should be noted that the lagoonal deposits encountered at the project site are highly compressible when subjected to structural loads and are potentially liquefiable during a moderate to strong seismic event.

Below the highly compressible lagoonal deposits, our field exploration generally encountered coralline deposits consisting of medium dense to very dense sandy and gravelly coralline detritus with pockets of loose sands and intermittent layers of soft to medium hard sandstone and coral formations extending down to the maximum depth explored of about 122.5 feet below the existing ground surface. It should be noted that in Boring Nos. 1 through 3, volcanic tuff and weathered volcanic tuff consisting of clayey sands were encountered within the coralline deposits between depths of about 110.5 and 114.5 feet below the existing ground surface.

We encountered groundwater in the drilled borings at depths between about 3 and 5.5 feet below the existing ground surface at the time of our field exploration. The groundwater levels encountered generally correspond to between about Elevations -1 and +2.5 feet MSL. Due to the area geology and close proximity to the Pacific Ocean, it should be noted that groundwater levels are expected to vary due to tidal fluctuation. Other factors that may affect groundwater levels at the project site include seasonal rainfall, time of year, surface runoff, and other factors.

Detailed descriptions of the field exploration methodology are presented in Appendix A of this report. Descriptions and graphic representations of the materials encountered in the drilled borings are provided on the Logs of Borings, Plates A-1.1 through A-8.2. Laboratory tests were performed on selected soil and rock core samples, and the test results are presented in Appendix C. Photographs of the core samples retrieved from the borings are presented in Appendix D.

2.4 Seismic Design Considerations

The project site will be subjected to seismic activity and should be evaluated for the potential for soil liquefaction. Seismic design of the proposed project will be based on the AASHTO LRFD Bridge Design Specifications, 9th Edition (AASHTO LRFD 2020) and the "Design Criteria for Bridges and Structures" prepared by DOT Highways Division dated August 8, 2014. The following subsections provide discussions on the seismicity, the potential for liquefaction at the project site, and the soil profile for seismic design.

2.4.1 Earthquakes and Seismicity

In general, earthquakes that occur throughout the world are caused solely 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 of the earthquakes are so small that they can be detected only by sensitive instruments. However, some of the earthquakes are strong enough to be felt, and a few cause minor to moderate damage.

In general, earthquakes (associated with volcanic activity) are most common on the Island of Hawaii. Earthquakes that are directly associated with the movement of magma are concentrated beneath the active Kilauea and Mauna Loa Volcanoes on the Island of Hawaii. Because the majority of the earthquakes in Hawaii (over 90 percent of earthquakes) are related to volcanic activity, the risk of high 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 numerous 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, several small landslides occurred on the Island of Oahu as a result of the Maui Earthquake of 1938 (M6.8). In addition, some houses on the Island of Oahu were reportedly damaged as a result of the Lanai Earthquake of 1871 (M7+).

Due to the relatively short period of documented earthquake monitoring in the State of Hawaii, information pertaining to earthquakes that were felt on the Island of Oahu may not be complete. In general, over the last 150 years of recorded history, we are not aware of reported earthquakes greater than Magnitude 6 occurring on the Island of Oahu. Based on available information, we understand that an earthquake of about Magnitude 5.6 occurred on June 28, 1948 in the vicinity of the Island of Oahu, possibly along the hypothesized and controversial Diamond Head Fault feature.

The Diamond Head Fault feature is believed to extend northeasterly away from the southeastern tip of the Island of Oahu. The Diamond Head Fault feature may be related to the widely documented Molokai Fracture Zone located on the sea floor in the vicinity of the Hawaiian Islands. Despite only the moderate tremor intensity, the resulting damage was reportedly widespread and included broken windows, ruptured masonry building walls, and a broken underground water main. In addition, some areas on the Island of Oahu, including the Tantalus, Iwilei, and Tripler areas, reported more intense ground shaking, severe enough to have cracked reinforced concrete.

2.4.2 Liquefaction Potential

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

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

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

Based on our analyses, it appears that the loose to very loose lagoonal deposits below the surface fill generally have a factor of safety of less than 1.0 against soil liquefaction and is susceptible to potential soil liquefaction. Therefore, the project site could be subjected to appreciable seismically induced ground settlements (on the order of about 3 to 13 inches) in the event of soil liquefaction during a strong earthquake (M6+).

Based on the subsurface conditions encountered during our field exploration and our analyses, it appears the makai side of the project is more susceptible to liquefaction with appreciable seismically induced ground settlements (on the order of about 7 to 13 inches) in the event of soil liquefaction during a strong earthquake (M6+). We envision settlement of the makai embankment in combination with lateral spread associated with liquefaction may result in surcharging the Kewalo Basin bulkheads approximately 50 feet south of the makai ramps. Therefore, we recommend designing the makai embankment to resist liquefaction and associated lateral spreading.

2.4.3 Soil Profile

Seismic shear wave velocity profiling was performed in an effort to analyze the subsurface conditions more closely for seismic design. We performed seismic shear wave velocity profiling using seismic piezocone penetration testing (SCPTu) equipment at discrete depths extending to a depth of approximately 111.7 feet below the existing ground surface at Boring No. 1. Based on the subsurface conditions, the weighted average shear wave velocity for the materials within the upper 100 feet of the soil profile is on the order of about 1,500 feet per second at the test location.

Based on a weighted average shear wave velocity of 1,500 feet per second for the materials within the upper 100 feet and the subsurface conditions encountered in our field exploration, the project site may be classified from a seismic analysis standpoint as a “Very Dense Soil and Soft Rock Profile.” Therefore, we believe the seismic design of the walkway structures may be designed based on a Site Class C soil profile based on Section 3 of the AASHTO LRFD 2020. Based on Site Class C, the following seismic design parameters were estimated and may be used for the seismic analysis of this project based on

AASHTO LRFD 2020 and the design criteria prepared by HDOT Highway Divisions.

SEISMIC DESIGN PARAMETERS AASHTO 2020 LRFD BRIDGE DESIGN SPECIFICATIONS 1,000-YEAR RETURN PERIOD (~7% PROBABILITY OF EXCEEDANCE IN 75 YEARS)	
Parameter	**Value**
Peak Bedrock Acceleration, PBA (Site Class B)	0.174g
Spectral Response Acceleration, S_S	0.398g
Spectral Response Acceleration, S_1	0.109g
Site Class	“C”
Site Coefficient, F_a	1.20
Site Coefficient, F_v	1.691
Site Coefficient, F_{PGA}	1.2
Design Peak Ground Acceleration, PGA (Site Class C) or A_s	0.209g
Design Spectral Response Acceleration, S_{DS}	0.477g
Design Spectral Response Acceleration, S_{D1}	0.185g

END OF SITE CHARACTERIZATION

SECTION 3. DISCUSSION AND RECOMMENDATIONS

Our field exploration generally encountered surface fill materials placed over lagoonal deposits overlying coralline deposits extending to the maximum depth explored of about 122.5 feet below the existing ground surface. We encountered groundwater in the drilled borings at depths ranging from about 3 to 5.5 feet below the existing ground surface. The groundwater levels encountered generally correspond to approximately Elevations -1 and +2.5 feet MSL at the time of our field exploration.

Generally, we anticipate the soft and/or loose subsurface conditions underlying the project site and the relatively heavy structural load demands will require supporting the new pedestrian bridge on a deep foundation system, such as cast-in-place concrete drilled shafts. The drilled shaft foundations would extend below the surface fills and soft and/or loose lagoonal deposits and derive support principally from adhesion between the drilled shaft and the dense/hard coralline deposits at greater depths. Based on the structural load demands provided for our engineering analyses, a drilled shaft diameter of 36 inches and embedment length of 60 feet may be used for the design of the new bridge abutment foundations. Due to the heavier loading at the center pier of the bridge, the center pier foundations should consist of drilled shafts of 48 inches in diameter and an embedment length of 75 feet.

Based on the grading plans, we understand significant fills of up to about 20 feet in height will be required to meet the finished grade elevations of the new pedestrian bridge. Ground settlements are anticipated when substantial new fills are placed over the existing ground underlain by soft and/or loose lagoonal deposits to raise the site to the proposed finished grades. Based on the subsurface conditions encountered and the planned fill thicknesses, we estimate potential filled ground settlements on the order of about 12 to 17 inches could occur at the planned mauka and makai embankments of the project. Therefore, a ground settlement monitoring program should be implemented to confirm the actual settlement rate prior to construction of the on-grade improvements on top of and within the filled ground areas.

Due to the close proximity of the makai ramp embankments to the Kewalo Basin bulkhead walls, we understand appreciable surcharge loads and liquefaction induced lateral spreading must be mitigated to avoid surcharging the existing bulkhead walls. Therefore, we recommend supporting the makai abutment and makai ramp embankments on jet grout columns to reduce the amount of ground settlement at this location and to improve slope stability. We also recommend supporting the abutment fill immediately behind the mauka abutment on jet grout columns to allow for construction of the abutment structure concurrently with the abutment fill.

Detailed discussions and recommendations for design of foundations, site grading, geosynthetic reinforced soil embankments, soil stabilization, and other geotechnical aspects of the project are presented in the following sections.

3.1 Structure Foundations

We envision various types of new structures, such as new bridges, embankment fills, jet-grouted column supported embankments and retaining walls, will be required for the new elevated pedestrian bridge project. Generally, we anticipate both shallow and deep foundation systems will be utilized for support of the planned structures for the project. Where ground improvements (including surcharge filling and soil stabilization) are implemented, the new structures would be supported on shallow foundations, such as mat, spread, and/or continuous strip footings. Deep foundations such as drilled shafts may be required for heavier structures underlain by weak subsurface conditions.

In general, we believe the pedestrian bridge structure will need to be supported on a deep foundation system consisting of concrete drilled shafts. Based on the subsurface conditions encountered, we believe drilled shaft foundations with nominal diameters of 36 and 48 inches may be used to support the abutment and center pier locations, respectively. The drilled shaft foundations of the pedestrian bridge would derive support principally from adhesion between the drilled shaft concrete and the coralline materials encountered in our borings.

Detailed geotechnical recommendations pertaining to the design of the planned structures are presented in the following subsections of this report.

3.2 Drilled Shaft Foundations

As mentioned above, we anticipate the soft and/or loose subsurface conditions and the relatively heavy structural load demands dictate supporting the bridge on a drilled shaft foundation system. Based on the information provided by the project structural engineer, each of the drilled shafts at the abutments for the pedestrian bridge will be subjected to a Strength I Limit State load demand of 425 kips. The drilled shafts at the center pier of the pedestrian bridge will be subjected to a Strength I Limit State load demand of 625 kips.

Based on the subsurface conditions encountered at the project site and the anticipated structural loads, we recommend supporting the new bridge structures on drilled shafts having a diameter of 36 and 48 inches for the abutments and center pier, respectively. Our recommendations pertaining to the drilled shaft foundation support system are presented in the following table.

PEDESTRIAN BRIDGE FOUNDATIONS			
	Mauka Abutment	**Center Pier**	**Makai Abutment**
Existing Ground Surface (feet MSL)	+4.5	+4.5	+5.5
Drilled Shaft Cutoff Elevation (feet MSL)	+2.5	+1.5	+2.5
Drilled Shaft Diameter (inches)	36	48	36
Drilled Shaft Length (feet)	60	75	60
Drilled Shaft Tip Elevation (feet MSL)	-57.5	-73.5	-57.5
Drilled Shaft Capacity (Resistance)			
Strength Limit State (kips)	425	625	425
Extreme Event Limit State (kips)	950	1,400	950
Nominal Single Shaft Capacity (kips)	1,700	2,500	1,700

In general, drilled shafts in groups should be spaced a minimum of three times the drilled shaft diameter center-to-center to avoid reduction in vertical load capacity due to group action and to facilitate drilling of the shaft holes.

The load bearing capacities of the drilled shafts will depend largely on the consistency and relative density of the soils and the quality of the coralline materials within the bearing strata. Because local variations in the subsurface materials likely will occur at the site, it is imperative that a Geolabs representative be present during the shaft drilling operations to confirm the subsurface conditions encountered during the drilled shaft construction and to observe the installation of the drilled shafts. In addition, contract documents should include provisions (unit prices) for additional drilling and extension of the drilled shaft during construction to account for unforeseen subsurface conditions.

Based on our evaluation of the subsurface conditions and the foundation design parameters, we anticipate the drilled shaft installation will require an experienced drilled shaft subcontractor to install the drilled shaft foundations. Therefore, consideration should be given to requiring pre-qualification of the drilled shaft subcontractor. The succeeding subsections address the design and construction of the drilled shaft foundations:

1. Lateral Load Resistance
2. Foundation Settlements
3. Drilled Shaft Construction Considerations
4. Bi-Directional Load Tests
5. Non-Destructive Integrity Testing

3.2.1 Lateral Load Resistance

In general, lateral load resistance of the drilled shafts is a function of the stiffness of the surrounding soil, the stiffness of the shaft, allowable deflection at the top of the shaft, and induced moment in the shaft. In general, we recommend spacing the drilled shafts at a minimum of three times the diameter of the shaft from center-to-center. The lateral load analyses were performed using the program LPILE-plus for Windows, which is a microcomputer adaptation of a finite difference, laterally loaded pile program originally developed at the University of Texas at Austin.

The lateral loads acting at the top of the shaft, the maximum induced moments, the depths at which the maximum moments occur, and the flexural length of the drilled shaft may be calculated using the computer program. The input parameters for the lateral load resistance was provided to the project Structural Engineer for their use. The effect of group action will need to be considered in the lateral load analysis for drilled shaft foundations based on a center-to-center spacing of at least three times the drilled shaft diameter.

3.2.2 Foundation Settlements

Settlement of the drilled shaft foundation will result from elastic compression of the shaft and subgrade response of the foundation embedded in the coralline materials. Total settlements of the abutment and center pier drilled shafts are estimated to be less than 0.5 inches. Therefore, differential settlements between the drilled shafts may be about 0.25 inches or less. We believe a significant portion of the settlement is elastic and should occur as the loads are applied.

3.2.3 Drilled Shaft Construction Considerations

In general, the performance of drilled shafts depends significantly upon the contractor's method of installation and construction procedures. The following conditions would have a significant effect on the effectiveness and cost of the drilled shaft foundations.

The load bearing capacities of drilled shafts depend, to a significant extent, on the friction between the shaft and the coralline materials. Therefore, proper construction techniques especially during the drilling operations are important. The contractor should exercise care in drilling the shaft holes and in placing concrete into the drilled holes.

As mentioned above, we encountered medium hard coral formation within the borings drilled for the pedestrian bridge structure. Therefore, some difficult drilling conditions likely will be encountered and should be expected. The drilled shaft subcontractor will need to have the appropriate equipment and tools to drill through these types of natural obstructions or harder zones, where encountered. The drilled shaft subcontractor will need to demonstrate that the proposed drilling equipment (and coring tools, where appropriate) will be capable of installing the drilled shafts to the recommended depths and dimensions.

Drilling by methods utilizing drilling fluids may have a significant effect on the supporting capacity of the drilled shaft; therefore, use of drilling fluids would require prior evaluation and acceptance by Geolabs. If drilling fluids are proposed by the drilled shaft subcontractor, the same type and quantity of drilling fluids should be used to construct the dedicated load test shaft for load testing purposes to evaluate the effect of the drilling fluid on the capacity of the drilled shaft.

We recommend concrete placement by tremie methods during drilled shaft construction due to the depth of the drilled shafts and the presence of groundwater. The concrete should be placed in a suitable manner in an upward fashion from the bottom of the drilled hole. A low-shrink concrete mix with high slump (7 to 9-inch slump range) should be used to provide close contact between the drilled shafts and the surrounding soils. The concrete should be placed in a suitable manner to reduce the potential for segregation of the aggregates from the concrete mix.

In addition, the concrete should be placed promptly after drilling (within 24 hours after drilling of the holes) to reduce the potential for softening of the sides of the drilled holes. Furthermore, drilling adjacent to a recently constructed shaft (within five shaft diameters of the recently constructed drilled shaft) should not commence until the concrete for the recently constructed drilled shaft has cured for a minimum of 24 hours.

It should be noted that some cavities and voids may be encountered in the coral formation in the project vicinity. Therefore, the actual volume of concrete required to fill the drilled shaft foundation may be appreciably more than the theoretical concrete volume.

It is imperative for a Geolabs representative to be present during construction to observe the drilling and installation of drilled shafts. Although the drilled shaft designs are primarily based on skin friction, the bottom of the drilled hole should be relatively free of loose materials prior to placement of concrete. Therefore, Geolabs observation of the drilled shaft installation operations is necessary to confirm the assumed subsurface conditions and should be designated a “Special Inspection” item.

3.2.4 Bi-Directional Load Tests

As part of the pre-construction activities, we recommend conducting one static load test for the elevated pedestrian walkway project. The load test should be conducted on a 48-inch diameter dedicated drilled shaft extending to a depth of about 75 feet below the existing ground surface. The results of the load tests will be used to confirm or modify the estimated tip elevations of the production drilled shafts. The load test shafts should be structurally reinforced and instrumented with embedment strain gauges for load testing purposes. As a minimum, two embedment strain gauges should be placed at each level, starting near the load cell location at an elevation of about 5 feet above and below the load cell and subsequently at about 6 to 10-foot intervals, as shown on the Drilled Shaft Load Test Detail (Plate 3).

Due to the high capacities recommended for the drilled shafts, a conventional load test would not be practical and would be costly to conduct. Therefore, we recommend conducting bi-directional axial load test using an expandable load cell (Osterberg Load Cell). The bi-directional load test separately tests the shear resistance and end-bearing components of the drilled shaft by loading the shaft in two directions (upward for shear resistance, and downward for end-bearing and shear resistance).

The expandable load cell should be capable of applying a load of at least 1,500 kips in each direction for the load test shaft. The expandable load cell will need to be attached to the reinforcing steel cage prior to lowering the cage into the drilled hole.

The drilled shaft load test should be performed in general accordance with the Quick Load Test Method of ASTM D1143. The load test shaft should be loaded to failure to evaluate the ultimate side shear resistance and end-bearing components of the shaft. Installation of the expandable load cells, installation of the embedment strain gauges, performance of the bi-directional axial load tests, and presentation of the load test data should be performed by a professional experienced in these types of load testing procedures. The load test shaft should be loaded at increments of about 150 kips and should be held for a minimum of 4 hours (each hold) at the 1,200-kip, 1,800-kip, and 2,400-kip load intervals for the load test shaft to evaluate the potential for creep effects.

A Geolabs representative should observe the installation and performance of the instrumented load test on the drilled shaft. It should be noted that the drilled shaft design was developed from our analysis using the field exploration data. Therefore, Geolabs observation of the drilled shaft installation operations is a vital part of the foundation design to confirm the design assumptions.

3.2.5 Non-Destructive Integrity Testing

Based on the critical nature of the drilled shaft foundations for the new bridge structure, we recommend that non-destructive integrity testing be conducted on the production drilled shafts for the project. One of the non-destructive integrity testing methods, such as Crosshole Sonic Logging (CSL), has been gaining widespread use and acceptance for integrity testing of drilled shafts.

Crosshole Sonic Logging techniques are based on the propagation of sound waves through concrete. In general, the actual velocity of sound wave propagation in concrete is dependent on the concrete material properties, geometry of the element and wave length of the sound waves. When ultrasonic frequencies are generated, sound waves travel though the concrete. If anomalies are contained in the concrete, the anomalies will reduce the wave travel velocity in the concrete. Anomalies in the drilled shaft concrete may include soil particles, gravel, water, voids, contaminated concrete, and highly segregated constituent particles.

The transit time of an ultrasonic P-wave signal may be measured between an ultrasonic transmitter and receiver in two parallel water-filled access tubes placed into the concrete during construction. The P-wave velocity can be obtained by dividing the measured transit time from the distance between the transmitter and receiver. Therefore, anomalies may be detected (if they exist).

To reduce the potential de-bonding between the access tube and the surrounding concrete, we recommend that the access tubes consist of standard steel pipe with a minimum inside diameter of 2 inches. In addition, the access tube should be equipped with watertight coupling. In general, the access tubes should be securely attached to the interior of the reinforcing cage as near to parallel as possible in the drilled shaft. We recommend that a minimum of four access tubes be cast into the concrete of the 48-inch diameter drilled shafts and a minimum three access tubes be cast into the concrete of the 36-inch diameter drilled shaft. Details pertaining to the configuration of the access tubes for crosshole sonic logging tests are presented on Plate 4.

In addition, the access tubes should be extended from the bottom of the drilled shaft reinforcing cage to at least 3.5 feet above the top of the shaft. The bottom of the access tube should be permanently capped. It is imperative that joints required to achieve the full length of the access tubes be watertight. It is the responsibility of the contractor to take extra care to prevent damaging the access tubes during the placement of the reinforcing cage into the drilled hole. The tubes should be filled with potable water as soon as possible, but not later than 4 hours after the concrete placement. Subsequently, the top of the access tubes should be capped with watertight caps.

The CSL testing of the drilled shafts should be conducted after at least 5 days of curing time, but no later than 28 days after concrete placement. In addition, the CSL test of drilled shafts should be performed in general accordance with ASTM D6760. In the event that a drilled shaft is found to have significant anomalies and/or is suspected to be defective based on the CSL testing and/or field observations, the drilled shaft should be cored to evaluate the integrity of the concrete in the drilled shaft. The coring location within the drilled shaft should be determined by a representative from Geolabs, who should be present to observe the installation of the drilled shafts. After completion of the crosshole sonic logging of the drilled shafts, all the access tubes should be filled with grout of the same strength as the drilled shaft concrete.

3.3 Site Grading

We anticipate that site grading consisting primarily of fills of up to about 20 feet and relatively minor cuts will be required to achieve the design finished grades. In addition, excavations and subsequent backfills of up to about 5 feet deep will be required for construction of the abutment and center pier foundations. In general, grading work should conform to the Hawaii Standard Specifications for Road Bridge Construction (2005) and the site-specific recommendations contained in this report. Items of site grading that are addressed in the subsequent subsections include the following:

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

A Geolabs representative should monitor the grading operations to observe whether undesirable materials are encountered during the excavation and scarification process, and to confirm whether the exposed soil conditions are similar to those encountered in our field exploration.

3.3.1 Fill Slope Design

In general, permanent embankments constructed of compacted imported select granular fill materials may be designed with a slope inclination of two horizontal to one vertical (2H:1V) or flatter. Fills to be placed on existing slopes with inclinations steeper than 5H:1V should be keyed and benched into the existing slope to provide stability of the new fill against sliding. The keyway at the bottom of fill slopes should be embedded at least 2 feet below the lowest adjacent grade and should have a minimum base width of 10 feet.

Excessive surface water runoff over the slope face may cause erosion of the exposed soils, thus jeopardizing the long-term stability and performance of the cut and fill slopes. Therefore, it is our opinion that slopes should be protected by appropriate slope planting or by other means, such as placement of erosion control matting on the slope face, as soon as practical after the slope is constructed.

3.3.2 Site Preparation

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

Soft and yielding areas encountered during clearing and grubbing below areas designated to receive fill should be over-excavated to expose firm natural material, and the resulting excavation should be backfilled with well-compacted fill. The excavated soft soils should be properly disposed of off-site.

Existing structures and pavements that are to be demolished should be completely removed. Over-excavations resulting from demolition should be backfilled with compacted select granular fill material. Existing utilities to be abandoned should be removed, and the resulting excavation should be properly backfilled with select granular fill material placed in 8-inch loose lifts, moisture-conditioned to above the optimum moisture content, and compacted to a minimum of 90 percent relative compaction. Utilities to be abandoned in-place under the proposed structures should be backfilled by pumping lean concrete or Controlled Low Strength Material (CLSM) under low pressure.

In general, the over-excavated subgrades and areas designated to receive fills (exposing soils) should be scarified to a depth of about 8 inches, moisture-conditioned to above the optimum moisture content, and recompacted to a minimum of 90 percent relative compaction. Due to the presence of shallow groundwater and the low elevations of the project site, the existing subgrade soils may be wet. Therefore, the subgrades may be proof-rolled using a drum roller (10-ton minimum static weight) at least for six passes to obtain a firm and unyielding surface to place compacted fills if the existing ground is wet but in a firm condition.

3.3.3 Fills and Backfills

The abutment and center pier footings will be located at depths of up to about 5 feet below the existing ground surface. In general, backfills from the tops of footings to the finished grades may consist of compacted general fills. In general, the near-surface sandy soils encountered during our field exploration should be suitable for use as general fill materials, provided that the maximum particle size is less than 3 inches in largest dimension. Excavated materials generated from excavations into granular fill materials may be used as general fill or backfill materials, provided that they are screened of the over-sized materials and/or processed to meet the above gradation requirements (less than 3 inches in largest dimension).

Fill or backfill below the water level should consist of free-draining granular materials, such as open-graded gravel (AASHTO M43 Size No. 67), up to a minimum of 12 inches above the groundwater level. We also recommend wrapping the open-graded gravel in a non-woven filter fabric, such as a permeable separator.

Imported materials to be used as select granular fill should be non-expansive granular material, such as crushed coral or basalt. The select granular fill should be well-graded from coarse to fine with particles no larger than 3 inches in largest dimension. The material should have a California Bearing Ratio (CBR) value of 20 or higher and a swell potential of 1 percent or less when tested in accordance with AASHTO T-193 (ASTM D1883). The material also should contain between 10 and 30 percent particles passing the No. 200 sieve. Aggregate subbase course meeting the requirements of Section 703.17 of the Hawaii Standard Specifications for Road and Bridge Construction, 2005 (HSS) also may be used as a source of imported fill materials for the project. Imported fill materials should be tested for conformance with these recommendations prior to delivery to the project site for the intended use.

3.3.4 Fill Placement and Compaction Requirements

In general, fills and backfills should be moisture-conditioned to above the optimum moisture content, placed in level lifts not exceeding 8 inches in loose thickness, and compacted to at least 90 percent relative compaction. Fills and backfills within 3 feet of the pavement grade elevation should 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 AASHTO T-180. Optimum moisture is the water content (percentage by dry weight) corresponding to the maximum dry density. Compaction should be accomplished by sheepsfoot rollers, vibratory rollers, or other types of acceptable compaction equipment. Water tamping, jetting, or ponding should not be allowed to compact the fills.

It should be noted that some of the on-site soils generally exist in a relatively moist to wet condition. Therefore, some moisture reduction may be required to achieve the minimum 90 percent compaction criteria, especially for materials primarily consisting of silts and clays. Aeration to lower the soil moisture and more compaction effort to achieve the specified compaction would generally reduce the rate of fill placement for this project. In addition, adequate stockpile areas may not be readily available on-site. Contractors proposing to work on this project should be encouraged to examine the site conditions and its limitations.

3.3.5 Excavations

Our field exploration program disclosed that the near-surface fills generally consist of very soft to medium stiff clayey/silty soils and very loose to medium dense sands and gravel of varying silt content. Boulders may be encountered in deeper excavations and in localized areas along the project alignment. In general, it is our opinion that conventional heavy excavation equipment, such as a large bulldozer, excavator, or similar heavy construction equipment, may achieve the excavations into these materials.

The method and equipment to be used for excavation should be determined by the contractor, subject to practical limits and safety considerations. The excavations should comply with all applicable local safety requirements. The above discussions regarding the rippability of the surface materials are based on field data obtained from our field reconnaissance and the borings performed at the subject site. Contractors proposing to work on this project should be encouraged to examine the site conditions to make their own interpretation.

3.4 Ground Settlements

Ground settlements are anticipated when substantial new fills are placed over the existing ground overlying soft and/or loose lagoonal deposits to raise the site to the proposed finished grades. These ground settlements would affect the construction schedule and the earthwork quantity estimates for the project. In general, the anticipated ground settlements are primarily the result of the following two processes:

- compression of the compacted fill material under its own weight; and
- consolidation and/or compression of the underlying in-situ soils induced by the new fill loads, especially where new fills are placed over soft and/or loose lagoonal deposits.

As indicated above, new fills (consisting of generally select granular fill materials and up to about 20 feet thick) will be placed over the existing surface fills and underlying soft and/or loose lagoonal deposits. Therefore, some settlement of the new fill materials imposed on the underlying soft and/or loose lagoonal deposits should be expected. In order to reduce the effects of the anticipated settlements on the slabs-on-grade constructed on the fills, a settlement waiting period should be implemented (in areas with substantial fills over soft soils) after placement of the fills and prior to construction of the on-grade improvements on the fills.

In general, the settlement rates for the potentially compressible soils could be slower and would require longer settlement waiting periods to reduce the effects of settlement on the on-grade improvements constructed in and on the fills. The settlement waiting period for fills constructed over the soft and/or loose soils may take 2 to 4 months depending on the nature and thickness of the soft soils.

Based on the subsurface conditions encountered and the planned fill thicknesses, we estimate potential filled ground settlements on the order of about 12 to

17 inches could occur at the planned mauka and makai embankments of the project. In addition, we estimate the majority of the primary settlement would occur in about 2 to 4 months after the new fills are placed. Therefore, we believe the new fills should be placed as soon as practical to allow the anticipated ground settlements to occur prior to construction of the on-grade improvements on top of the fill. However, the drilled shaft foundations and associated substructure elements (foundation caps) for the bridge structure may commence within the settlement waiting period because the drilled shaft foundations will extend through the fill and soft and/or loose lagoonal deposits and into the underlying competent coralline deposits.

Based on the information provided, we understand the existing drainage box culvert traversing below the mauka fill embankment below the elevated pedestrian walkway will be removed and replaced with a new HDPE pipe designed for the future embankment loads as part of the future Victoria Ward park project. Construction of this project will occur before the commencement of the elevated pedestrian walkway project.

It should be recognized that it is difficult to accurately predict the exact time required for the filled ground to settle because the settlement rates are affected by variations in the subsurface soil structure and the history of the soil deposition. For the soft and/or loose soils anticipated at the project site, we believe the estimated settlement period could vary by as much as 50 to 100 percent from the actual settlement period. Therefore, the actual settlement rates should be monitored, and a settlement monitoring program should be established to evaluate the magnitude and rate of the estimated settlements during the settlement waiting period prior to construction of improvements on the fills. In addition, provisions should be made for potential delays in the construction schedule if a longer settlement waiting period is required.

To monitor the actual settlement rate, we recommend that settlement gauges (minimum six gauges) be installed in areas where new fills are placed over the soft and/or loose soils. A typical settlement gauge detail is shown on Plate 5. The settlement gauges should be read optically by a qualified professional surveyor, and the readings should be transmitted for review in a timely manner. We recommend taking two readings (minimum 24 hours apart) for each settlement gauge 10 days prior to any site filling to establish a baseline. Subsequent readings of the settlement gauges should be taken on a weekly basis for the entire settlement waiting period. Geolabs should review the settlement readings to evaluate if the settlement waiting period may be shortened or extended depending on the settlement readings and type of construction activity.

It should be noted that a seismic event could induce liquefaction settlement even after completion of the settlement monitoring period. Embankments sensitive to differential settlements should consider applying geosynthetic reinforced soil (GRS) and/or soil stabilization.

3.5 Geosynthetic Reinforced Soil Embankments

As discussed previously, we anticipate geosynthetic reinforced soil (GRS) embankments would be utilized as abutments to provide additional lateral load resistance to the bridge structure during a seismic event. In general, we believe the GRS embankments may be designed with near vertical faces provided that the earth materials are reinforced with adequate layers of geotextiles to strengthen the fill soils.

Reinforced soil geotextiles generally are high-density polyethylene sheets with a good tensile strength characteristics. It generally provides a cost-effective solution to slope stability problems, which may include the following: insufficient right-of-way, high surcharge loads, poor-quality fills, high seismic forces, steep slopes, or difficult landslide repairs. When reinforced slope geotextiles are placed in soil, the reinforced slope geotextiles interlocks with the adjacent soil, creating a soil-geotextile composite with greatly enhanced engineering properties. Different combinations of reinforced slope geotextiles are available to provide optimum soil-geotextile interaction for a range of soil types and slope reinforcement applications. The construction methodology is briefly described as follows.

As the embankment is constructed, near-horizontal layers of geotextiles are placed in the compacted fill at predetermined levels. The lengths of the geotextile layers are designed to anchor potential failure zones into stable interior sections of the embankment. As forces develop within a soil mass, the high-modulus geotextiles are immediately pulled into tension. The geotextiles transfer this tensile force from the unstable soil back into less-stressed portions of the slope, and stability is thus maintained. Based on the current design concept, we recommend the geotextiles extend the entire lengths and widths of the GRS embankments for each horizontal layer.

We envision that imported select granular fill soils will be used for the reinforced earth fill embankment. Therefore, we believe that a friction angle of at least 38 degrees and a wet density of about 130 pounds per cubic foot (pcf) may be used for the design analyses of the geosynthetically reinforced soil (GRS) embankments. The select granular fill soils should be in placed accordance with requirements provided in the “Site Grading” section of this report. A friction angle of 30 degrees and a wet density of 110 pcf may also be used for the foundation soils consisting of the loose to medium dense sands and gravel underlying the project site.

The reinforced fill material (select granular fill) should have an angle of internal friction of at least 38 degrees when tested in accordance with the standard direct shear test (ASTM D 3080). The sample to be tested should be compacted to 95 percent relative compaction at a moisture content above the optimum moisture content. Fill materials for the reinforced earth slopes should be placed in loose lifts not exceeding 8 inches thick, moisture-conditioned to above the optimum moisture content, and compacted to at least 95 percent of the maximum dry density established in accordance with AASHTO T-180 test methods (ASTM D 1557).

3.6 Soil Stabilization

In order to stabilize the soft and/or loose lagoonal deposits from potential liquefaction and associated lateral spreading and to reduce the potential for significant ground settlement in the future, we recommend the soft and/or loose lagoonal deposits below the GRS embankments and makai embankment be stabilized. Based on the current design concept, we understand only areas that are highly sensitive to potential settlement will be stabilized. Three soil stabilization methods were considered for this project including the following:

- Compaction Grouting
- Stone Columns
- Jet Grouting

Due to the close proximity of the Kewalo Basin's harbor, high pressures associated with compaction grouting could laterally surcharge the harbor's seawall and should be avoided. In addition, the process of stone column installation will induce some ground settlement, which would adversely affect the adjacent underground utilities and alter the drainage patterns of the surrounding paved areas. Therefore, we recommend that soil stabilization by the jet-grouting method be used to stabilize the very soft and/or loose lagoonal deposits at the site.

In general, jet grouting is a technique utilizing a special drill bit and injection monitor with radial horizontal nozzles to produce stabilized soil-cement columns. The jet-grouting technique is a process that produces soil-cement columns by pumping neat cement grout slurry through horizontal jets injected at high pressures. The horizontal jets of cement grout slurry cuts and mixes the surrounding in-situ materials with the neat cement slurry grout as the drill bit is slowly rotated and withdrawn to form a soil-cement column.

In order to provide support for the new abutment and makai embankments, we recommend installing jet-grouted columns under the abutment and makai ramp embankment fills. The jet-grouted columns would derive vertical support primarily from bearing on the dense coralline deposits. Based on our evaluation of the subsurface conditions and the load supporting capacity of the jet-grouted columns, we recommend the soil stabilization consist of 42-inch diameter jet-grouted columns. Each jet-grouted column would be able to support 220 kips of load (weight of the overlying embankment fill). Items of the jet-grouted columns that are addressed in the succeeding subsections include the following:

1. Jet-Grouted Columns
2. Jet Grouting Equipment
3. Jet Grouting Test Program
4. Quality Control
5. Construction Considerations

3.6.1 Jet-Grouted Columns

Based on experience, the jet-grouted columns should have a minimum average diameter of 42 inches. Due to the nature of jet grouting, deviations from the specified minimum average diameter of the jet grout column is anticipated depending on the subsurface conditions. However, the jet grout column should not have a diameter less than 36 inches. In addition, the grout mix should have a specific gravity of at least 1.6 and should be able to produce jet-grouted columns with a 7-day unconfined compressive strength of at least 200 pounds per square inch (psi) and a 28-day unconfined compressive strength of at least 600 psi.

The 42-inch diameter jet-grouted columns should be spaced at grids of up to approximately 10 feet by 12 feet. In general, the jet-grouted columns should be extended until dense/hard materials are encountered at each jet-grouted column location. We also recommend the tip of the jet grout column extend a minimum of 2 feet into the dense/hard materials encountered at each jet-grouted column location. Due to the specialized nature of the jet grouting work, a representative from Geolabs should be present at the site to observe the jet-grouted column installation operations. Based on our field exploration, we estimate the length of the jet grout columns will be about 25 to 35 feet extending from the bottom of the load transfer platform.

3.6.2 Jet Grouting Equipment

Based on the subsurface conditions at the site, we believe the single-fluid method of jet grouting would be able to produce the jet-grouted columns meeting the recommended diameter for support of the abutments and makai ramps for this project. The drilling equipment should be capable of advancing the jetting rods to the depth required for this project. The drilling equipment also should be equipped with automated controls necessary to slowly rotate and withdraw the jetting rods at those rates determined necessary for the formation of the jet-grouted columns. We also recommend that an automatic recording system equipped with the drilling equipment along with a digital readout that will provide instantaneous, simultaneous records of the jet grouting parameters for each jet grout column at vertical intervals no greater than 0.5 feet. Rates of rotation and withdrawal of the jetting rods for each column should be recorded by the contractor and confirmed by a representative from Geolabs.

Grout mixers, holding tanks, and associated equipment should be capable of continuously producing a uniform grout mixture required for the formation of the jet-grouted columns. Uniformity of the grout mixture should be measured and recorded by the contractor by taking unit weight (density) measurements of the mixed grout by mud balance at least once every 1,000 gallons of grout mixed and pumped.

High-pressure pumps for the jet grouting operations should be capable of delivering grout at a minimum pressure of 4,000 psi. The high-pressure pumps should be equipped with the necessary gauges to measure and record grout pumping pressures, flow rate, and total grout used for each column.

3.6.3 Jet Grouting Test Program

We recommend that a jet grouting test program be undertaken to evaluate the proposed grouting methods and the ability of the proposed grout mix to produce jet grout columns meeting the depth, diameter, and material property requirements for the project. Test program should be conducted and evaluated, including the results of 28-day unconfined compressive strength tests, prior to starting production jet grouting work.

To achieve these objectives, we recommend that at least one test section consisting of a minimum of three jet grouted columns be constructed using the same procedures proposed for the production jet grouting work. In general, the jet grout columns for the test section should extend down to the dense/hard materials encountered in our borings at the approximate elevations indicated in above table. The test columns should be installed up to near the existing ground surface to allow for later excavation for physical inspection. Excavation to expose the grout columns of the test section should not be sooner than 10 days after the jet grout columns have been constructed.

After the jet grout test columns have set up sufficiently, at least four continuous core samples should be obtained from the full depth of the test columns. In general, we recommend that triple tube core barrels with thin walls be employed to obtain a continuous core sample of the jet grout columns. The core barrel should have a nominal inside diameter of at least 3.3 inches or greater.

The core samples should be inspected and checked for segregation. Compression tests should be performed on a minimum of four cores retrieved from each of the continuous core samples to determine the 28-day compressive strengths. The compressive strength of the core samples should be determined in accordance with ASTM D 1633 or ASTM D 2850, as appropriate. If the results of the test program are not satisfactory, modifications to the jet grout column construction procedures and additional test sections may be required.

We recommend that a representative from Geolabs be present during the jet grouting test program to observe and evaluate the field performance of the proposed jet grouting equipment and methods. Therefore, observation of the jet grouting operations by Geolabs is necessary and should be designated a "Special Inspection" item.

3.6.4 Quality Control

The type of jet grouting system and grouting parameters for grout mix, grout pressures, rotational speed, lifting rate, grout flow rate, number and size of jet nozzles, and drilling methods greatly affect the performance of the jet grouted columns. Therefore, an adequate quality control program should be implemented during the production jet grouting operations.

In general, grout mix uniformity should be verified by unit weight (density) measurements of the mixed grout by mud balance, Marsh Viscosity, and/or bleed from samples taken from the grout return line, in accordance with API Standard 13B test method. At least one group of tests should be conducted for every 2 hours that the grout is mixed and pumped.

A minimum of six cement grout samples should be fabricated in accordance with ASTM C109 every day that the grout is mixed and pumped. Two grout samples should be subjected to compressive strength tests at 7 days in accordance with ASTM C39 or C109 and ASTM D1633, respectively. The remaining samples should be subjected to compressive strength tests at 28 days following the applicable ASTM testing procedures.

In addition, core samples should be taken after the production jet grouted columns have reached sufficient strength. We recommend that the vertical core samples be taken from the full depth of the treated columns of about 6 percent of the total number of jet-grouted columns or a minimum of eight core borings. The core samples at each location should be tested for unconfined compressive strength as described in the "Jet Grouting Test Program" subsection. If the samples tested do not meet the specified strength requirements, then additional replacement jet grout columns may be required, or other provisions should be implemented, to compensate for the lower strength columns.

3.6.5 Construction Considerations

The subsurface conditions generally consist of fills and loose to medium dense lagoonal deposits. It should be noted that cobbles and boulders may be present in the surface fill materials at the site. In addition, some cemented zones are commonly encountered above the lagoonal deposits at the project site. Therefore, potentially difficult drilling conditions may be encountered and should be expected by the contractor. The jet grouting contractor will need to have the appropriate equipment and tools to drill through these obstructions, where encountered.

In addition, grout, soil, and water spoil returns produced during the jet grouting operations should be contained and disposed of properly by the contractor.

3.7 Load Transfer Platform

Differential settlements in the embankments supported on jet-grouted columns could result in structure distress and uneven walkway surfaces. Therefore, we recommend providing a load transfer platform to adequately transfer the load embankment loads to the jet-grouted columns. The load transfer platform should consist of 30 inches of No. 2 Rock (AASHTO M43 Size No. 4 or ASTM C33, No. 4 gradation) wrapped in a non-woven filter fabric (Permeable Separator). The stabilization layer should extend beyond the sides of the jet-grouted columns and overlying embankment by a minimum of 2 feet.

Geogrids (Tensar TriAx Grid TX-190L or equivalent) should be installed within the load transfer platform to increase stiffness of the stabilization layer and load transfer to the jet-grouted columns. In general, the geogrids will interlock with the No. 2 Rock, resulting in two benefits consisting of increasing the modulus of the stabilization layer by providing lateral confinement and enhancing the subgrade bearing capacity. Three layers of geogrids should be placed horizontally throughout the load transfer platform: the first layer above the filter fabric prior to rock placement and the second and third layers placed at 8 to 12-inch vertical spacing.

To promote interlocking of the No. 2 rock with the geogrid in the Load Transfer Platform, we recommend densifying each layer of the open graded gravel (No. 2 rock) between the geogrid layers with a vibratory plate tamper a minimum of four passes per layer.

3.8 Structure Approach Slab

As previously indicated, we anticipate that a relatively substantial embankment of about 20 feet high will be required in order to construct the abutment structures of the new pedestrian bridge. To reduce the potential for appreciable abrupt differential settlements between the jet-grouted column-supported embankments and the newly placed fills (with temporary surcharge fills), we recommend providing structure approach slabs at the column-supported embankment transitions to the filled ground. In general, the structure approach slabs should be at least 10 feet in length.

The structure approach slabs should be supported on a minimum of 6 inches of aggregate subbase course placed on a prepared subgrade. The subgrade should be scarified to a depth of about 8 inches, moisture-conditioned to above the optimum moisture content, and compacted to no less than 95 percent relative compaction. The aggregate subbase course also should be moisture conditioned to above the optimum moisture content and compacted to at least 95 percent relative compaction. Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same soil established in accordance with AASHTO T-180 (or ASTM D1557). Optimum moisture is the water content (percentage by dry weight) corresponding to the maximum dry density.

3.9 Retaining Structures

Based on the information provided, we understand that retaining structures, such as abutment walls and grade separation walls, will be required. Therefore, the following general guidelines are provided and may be used for design of retaining structures at the project site.

3.9.1 Retaining Structure Foundations

Parameters for design of foundations for the abutment walls supported on drilled shafts have been provided in the “Bridge Foundations” section of this report. Design of foundations for the retaining walls and other walls (not structurally connected to the bridge structure) should be based on the parameters presented in the following subsections of this report.

Based on the information provided, we understand that retaining walls of up to about 20 feet high may be required for the grade separation structures for the new bridge structure. In general, we anticipate that shallow foundations bearing on jet-grouted columns constructed for the embankment fills at the project site may be utilized for support of the planned retaining walls. Areas with less embankment fills (such as some landscaping areas) may be supported on a load transfer platform without jet-grouted column support. Based on our field exploration, we believe that the following values may be used to evaluate the bearing support, sliding resistance, and passive pressure resistance of the planned retaining walls supported on a load transfer platform based on LRFD design methods.

RETAINING WALL FOUNDATIONS			
Description	**Extreme Event Limit State**	**Strength Limit State**	**Service Limit State**
Bearing Pressure (Jet-Grouted Column Supported)	12,000 psf	6,000 psf	4,000 psf
Bearing Pressure (Only Load Transfer Platform Supported)	7,500 psf	3,750 psf	2,500 psf
Coefficient of Friction	0.46	0.39	N/A
Passive Pressure Resistance	360 pcf	180 pcf	N/A

In general, foundations should be embedded a minimum of 18 inches below the lowest adjacent finished grades. 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 they should extend 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.

Based on a service limit state bearing pressure of 4,000 psf, we estimate that foundation settlements under the anticipated design loads for foundations bearing on the jet-grouted columns to be less than 0.5 inches.

Lateral loads acting on the structure may be resisted by frictional resistance between the base of the foundation and the load transfer platform and by passive earth pressure developed against the near-vertical faces of the embedded portion of the foundation. The passive pressure resistance values presented in the table above, expressed in pounds per square foot per foot of embedment (pcf), may be used to evaluate the passive resistance for footings embedded in medium dense sandy soils. Unless covered by pavements or slabs, the passive resistance in the upper 12 inches should be neglected.

3.9.2 Static Lateral Earth Pressures

Retaining structures, including the abutment walls and grade separation walls, should be designed to resist the lateral earth pressures due to the adjacent soils and surcharge effects caused by loads adjacent to the retaining structures. The recommended lateral earth pressures for design of retaining structures, expressed in equivalent fluid pressures, are presented in the following table.

LATERAL EARTH PRESSURES FOR DESIGN OF RETAINING STRUCTURES

Backfill Condition	**Earth Pressure Component**	**Active** (pcf)	**At-Rest** (pcf)
Level Backfill	Horizontal	31	50
	Vertical	None	None
Maximum 2H:1V Sloping Backfill	Horizontal	38	54
	Vertical	18	27

The values provided above assume that Type A Structure Backfill Material conforming to Section 703.20 of the Hawaii Standard Specifications for Road and Bridge Construction, 2005 (HSS) or select granular fill will be used to backfill behind the retaining structures. It is assumed that the backfill behind retaining structures will be compacted to at least 90 percent relative compaction. In general, an active condition may be used for gravity retaining walls or walls that are free to deflect by as much as 0.5 percent of the wall height. If the tops of walls are not free to deflect beyond this degree or are restrained, the walls should be designed for the at-rest condition. These lateral earth pressures do not include hydrostatic pressures that might be caused by groundwater trapped behind the walls.

Surcharge stresses due to areal surcharges, line loads, and point loads within a horizontal distance equal to the depth of the wall should be considered in the design. For uniform surcharge stresses imposed on the loaded side of the wall, a rectangular distribution with uniform pressure equal to 24 percent of the vertical surcharge pressure acting over the entire height of the wall, which is free to deflect (cantilever), may be used in design. For walls that are restrained, a rectangular distribution equal to 39 percent of the vertical surcharge pressure acting over the entire height of the wall may be used for design. Additional analyses during design may be needed to evaluate the surcharge effects of point loads and line loads.

3.9.3 Dynamic Lateral Earth Pressures

Dynamic lateral earth forces due to seismic loading (A_s = 0.209g) may be estimated by using $3.7H^2$ pounds per linear foot of wall length for level backfill conditions, where H is the height of the wall in feet. It should be noted that the dynamic lateral earth forces provided assume that the wall will be allowed to move laterally by up to about 1 to 2 inches in the event of a strong earthquake. The resultant force should be assumed to act through the mid-height of the wall. An appropriately reduced factor of safety may be used when dynamic lateral earth forces are accounted for in the design of the retaining structures.

If the estimated amount of lateral movement is not acceptable, the retaining structure should be designed with higher dynamic lateral forces for a restrained condition. For a restrained condition (less than 0.5 inches of lateral movement), dynamic lateral forces due to seismic loading may be estimated using $8.1H^2$ pounds per linear foot of wall (H measured in feet) for level backfill conditions.

3.9.4 Lightweight Planter Soil Lateral Earth Pressures

We understand lightweight planter soil mix is designated in some landscaping areas to be laterally supported by retaining walls. Based on a unit weight of up to 75 pounds per cubic foot (pcf) and anticipated minimal to no compaction, the recommended lateral earth pressures for design of lightweight planter soil retained structures are presented in the following table.

LATERAL EARTH PRESSURES FOR DESIGN OF LIGHTWEIGHT PLANTER SOIL RETAINED STRUCTURES			
Backfill Condition	**Earth Pressure Component**	**Active** (pcf)	**At-Rest** (pcf)
Level Backfill	Horizontal	27	40
	Vertical	None	None
Maximum 2H:1V Sloping Backfill	Horizontal	44	55
	Vertical	22	28

Based on the planter designation and limited space of these areas, we anticipate the area will restrict access of pedestrians and heavy equipment. Therefore, surcharge stresses were not a design consideration.

3.9.5 Drainage

Retaining walls should be well drained to reduce the potential for the build-up of hydrostatic pressures. A typical drainage system would consist of a 12-inch wide zone of permeable material, such as drain rock (AASHTO M43 Size No. 67), immediately adjacent to the wall with 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 or EnkaDrain, may be used instead of the drainage material. The prefabricated drainage product should also be hydraulically connected to a perforated pipe at the base of the wall.

Backfill behind the permeable drainage zone should consist of Type A Structure Backfill Material conforming to Section 703.20 of the HSS or select granular fill (a minimum of 90 percent relative compaction). Unless covered by concrete slabs or pavements, the upper 12 inches of backfill should consist of relatively impervious material to reduce the potential for water infiltration behind the walls. In addition, the backfill below the drainage outlet (or weepholes) should consist of the relatively impervious material to reduce the potential for water infiltration into the footing subgrade. The relatively impervious material should be compacted to not less than 90 percent relative compaction.

3.10 Light Pole Foundations

Based on the information provided, we understand new light poles are to be included in the construction to illuminate the new walkways. The bridge and makai walkway embankment light poles will be affixed to the elevated walkway structure whereas the mauka walkway embankment light poles are to be individually supported on independent foundation systems. In addition to the new light poles, a decorative light pole within the Ala Moana Boulevard median and another light pole within Kewalo Basin are to be relocated.

In order to develop the required bearing and lateral load resistances, the proposed light poles may individually be supported by a deep foundation system consisting of a single cast-in-place concrete drilled shaft. The cast-in-place concrete drilled shaft would derive vertical support principally from skin friction between the shaft and the surrounding soils. It should be noted that the top of drilled shaft will vary greatly due to the changes in grade at the project site. The loads imposed onto the light pole foundations provided by the project structural engineer are summarized in the table below.

STRUCTURAL LOADS AT LIGHT POLE BASE

Location	**Axial** (kips)	**Shear** (kips)	**Bending Moment** (kip-feet)
Makai Walkway Embankment	0.255	0.22	1.51
Ala Moana Boulevard Median	0.5	0.793	19.6
Kewalo Basin	0.5	0.793	19.6

Based on the structural loads provided and the subsurface conditions encountered at the project site, we recommend using drilled shaft foundations with an embedment length of no less than 4 feet below the lowest adjacent finished grade. Due to the higher structural loads and relatively shallow lagoonal deposits at the relocated light poles within the Ala Moana Boulevard median and the Kewalo Basin, we recommend increasing the drilled shaft diameter and embedment length of these foundations to 30 inches and no less than 7.5 feet, respectively. Our recommendations pertaining to the drilled shaft foundation support system are presented in the following table.

LIGHT POLE FOUNDATIONS PER LOCATION			
	Mauka Walkway	**Ala Moana Blvd Median**	**Kewalo Basin**
Existing Ground Surface (feet MSL)	+4.5	+4.5	+5.5
Drilled Shaft Cutoff Elevation (feet MSL)	Varies	+4.5	+5.5
Drilled Shaft Diameter (inches)	24	30	30
Drilled Shaft Length (feet)	4	7.5	7.5
Drilled Shaft Tip Elevation (feet MSL)	Varies	-3.0	-2.0
Drilled Shaft Capacity (Resistance)			
Strength Limit State (kips)	1.2	3.2	3.2
Extreme Event Limit State (kips)	2.6	7.2	7.2
Nominal Single Shaft Capacity (kips)	4.7	12.8	12.8

Based on our evaluation of the subsurface conditions and the foundation design parameters, we anticipate the drilled shaft installation will require an experienced drilled shaft subcontractor to install the drilled shaft foundations. Therefore, consideration should be given to requiring pre-qualification of the drilled shaft subcontractor. The succeeding subsections address the design and construction of the drilled shaft foundations:

1. Lateral Load Resistance
2. Foundation Settlements
3. Hard Coral Formation and Obstructions
4. Loose Sandy Soils
5. Workmanship

3.10.1 Lateral Load Resistance

The lateral load resistance of the drilled shaft is a function of the stiffness of the surrounding soils, the stiffness of the drilled shaft, allowable deflection at the top of the drilled shaft, and the induced moment in the drilled shaft. The lateral load analyses were performed using the program LPILE-plus for Windows, which is a microcomputer adaptation of a finite difference, laterally loaded deep foundation program originally developed at the University of Texas at Austin. The program solves for deflection and bending moment along a deep foundation under lateral loads as a function of depth. The analysis was carried out with the use of nonlinear "p-y" curves to represent soil moduli. The lateral deflection was then computed using the appropriate soil moduli at various depths.

Based on the loading conditions provided and the anticipated subsurface soil conditions, the results of our lateral load analyses are summarized in the following table:

LATERAL LOAD ANALYSES FOR LIGHT POLE FOUNDATIONS			
Location	**Maximum Horizontal Deflection** (inches)	**Maximum Moment** (kip-feet)	**Depth to Maximum Moment** (feet)
Makai Walkway Embankment	0.2	1.7	1.1
Ala Moana Boulevard Median	0.2	20.6	1.1
Kewalo Basin	0.2	20.6	1.8

3.10.2 Foundation Settlement

Settlement of the light pole foundation will result from elastic compression of the shaft and subgrade response of the foundation embedded in the soils encountered at the site. The total settlement of the drilled shaft is estimated to be less than 0.5 inches. We believe that a significant portion of the settlement will be elastic and should occur as the loads are applied.

3.10.3 Hard Coral Formation and Obstructions

Based on our field exploration and experience in the area, the proposed light poles could be underlain by hard coral formation and/or concrete obstructions at relatively shallow depths. Therefore, coring into potentially hard materials should be anticipated during the drilled shaft construction. Appropriate drilling equipment (and coring tools) should be utilized by the drilled shaft contractor to install the drilled shafts to the depths and dimensions recommended herein.

3.10.4 Loose Sandy Soils

As mentioned above, loose to medium dense sands were encountered within the existing fill materials. Therefore, loose sandy soils could be encountered within the depth of the drilled shaft foundations. In order to provide safe access by the workers and to reduce the potential for caving-in of the drilled holes, temporary/permanent steel casing may be required for the drilled shaft foundation construction work.

3.10.5 Workmanship

The load-bearing capacities of the drilled shafts depend, to a large extent, on the contact between the drilled shafts and the surrounding soils. Therefore, proper construction techniques are important. The contractor should exercise care when drilling the shaft holes and placing concrete into the holes.

A low-shrink concrete mix with a high slump (6 to 9-inch range) should be used to provide close contact between the drilled shafts and the surrounding soils. The concrete should be placed in a suitable manner to reduce the potential for segregation of the aggregates from the concrete mix. In addition, the concrete should be placed promptly after drilling (within 24 hours after drilling of the holes) to reduce the potential for softening of the sides of the drilled hole.

3.11 Corrosion Potential

Design of metallic substructures, such as metallic piping, should consider the effects of the corrosive environment on the substructure. Resistivity is generally recognized as one of the most significant soil characteristics regarding the corrosivity of the soil to buried metallic objects. In general, the lower the resistivity, the greater the potential for corrosion of the buried metallic structure. Conversely, the higher the resistivity, the less likely the soil will contribute to corrosion of metallic objects.

Two sets of corrosivity test, including pH (ASTM G51), Minimum Resistivity (ASTM G57), Chloride Content (EPA 300.0), and Sulfate Content (EPA 300.0), were performed by our office and Eurofins TestAmerica Laboratories, Inc. on selected soil samples obtained from our field exploration. A summary of the corrosivity tests is presented in Appendix C of the referenced geotechnical data report.

Based on the results of corrosivity laboratory testing, the subsurface soils at the project site exhibit minimum resistivity values of about 2,000 and 5,300 ohm-cm. Therefore, the on-site near-surface soils may be considered very to moderately corrosive (Corrosion Ratings of 2 to 4) to buried metallic structures based on the Board of Water Supply, City and County of Honolulu Water System External Corrosion Control Standards dated 1991.

The method used to control the corrosion of underground concrete pipelines and structures is dependent on the pH value, chloride content, and sulfate content found in the soil. In general, soils with a chloride content of less than 300 parts per million (ppm), sulfate content of less than 2,000 ppm, and a pH of greater than 5.0 may be considered “non-corrosive” to underground concrete pipelines and structures. Based on the laboratory tested values of pH, chloride content, and sulfate content of the in-situ soils, we believe that the near-surface soils at the project site may be considered “non-corrosive”, therefore, either Type I or Type II (Type I/II) cement may be used for the concrete in contact with the ground.

3.12 Design Review

Final drawings and specifications for the proposed construction should be forwarded to Geolabs for review and written comments prior to bid solicitation and/or construction. This review is necessary to evaluate conformance of the plans and specifications with the intent of the foundation and earthwork recommendations provided herein. If this review is not made, Geolabs cannot assume responsibility for misinterpretation of the recommendations presented herein.

3.13 Post-Design Services/Services During Construction

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

1. Review of drilled shaft foundation installation submittals
2. Review of jet grout column installation submittals
3. Observation of the load test shaft installation and load testing
4. Observation of the production drilled shaft installation
5. Observation of the jet grouting test section
6. Observation of the production jet grout columns
7. Observation of the GRS embankment construction
8. Observation of the subgrade soil preparation
9. Observation of fill placement and compaction

A Geolabs representative should monitor the other aspects of the earthwork construction. This is to observe compliance with the intent of the design concepts, specifications, or recommendations and to expedite suggestions for design changes that may be required in the event that subsurface conditions differ from those anticipated at the time this report was prepared. The recommendations provided herein are contingent upon such observations.

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

END OF DISCUSSION AND RECOMMENDATIONS

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 data points may occur, and the nature and extent of these variations may not become evident until construction is underway. If variations then appear evident, it will be necessary to re-evaluate the recommendations presented herein.

The field boring locations indicated in this report are approximate, having been staked out in the field using a hand-held Global Positioning System (GPS) unit. Elevations noted on the borings were interpolated based on the Ala Moana Boulevard Elevated Pedestrian Walkway Topographical Survey provided by WSP USA, Inc. on January 4, 2021. The locations and elevations of the field borings should be considered accurate only to the degree implied by the methods used.

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

This report has been prepared for the exclusive use of Victoria Ward, Ltd. and their project consultants for specific application to the *Ala Moana Boulevard Elevated Pedestrian Walkway* 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 design engineers in the preparation of the design documents for the highway improvements 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 or contract bidding. A contractor wishing to bid on this project should retain a competent geotechnical engineer to assist in the interpretation of this report and/or performance of site-specific exploration for bid estimating purposes.

The owner/client should be aware that unanticipated subsurface conditions are commonly encountered. Unforeseen subsurface conditions, such as 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 for presence of hazardous materials existing at the project site. It should be noted that the equipment, techniques, and personnel used to conduct a geo-environmental exploration differ substantially from those applied in geotechnical engineering.

END OF LIMITATIONS

CLOSURE

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

-ΩΩΩΩΩΩΩΩΩ-

Respectfully submitted,

GEOLABS, INC.

By Hignite
Andrew Hignite, P.E.
Project Engineer

By
Robin M. Lim, P.E.
President

RML:AH:rl

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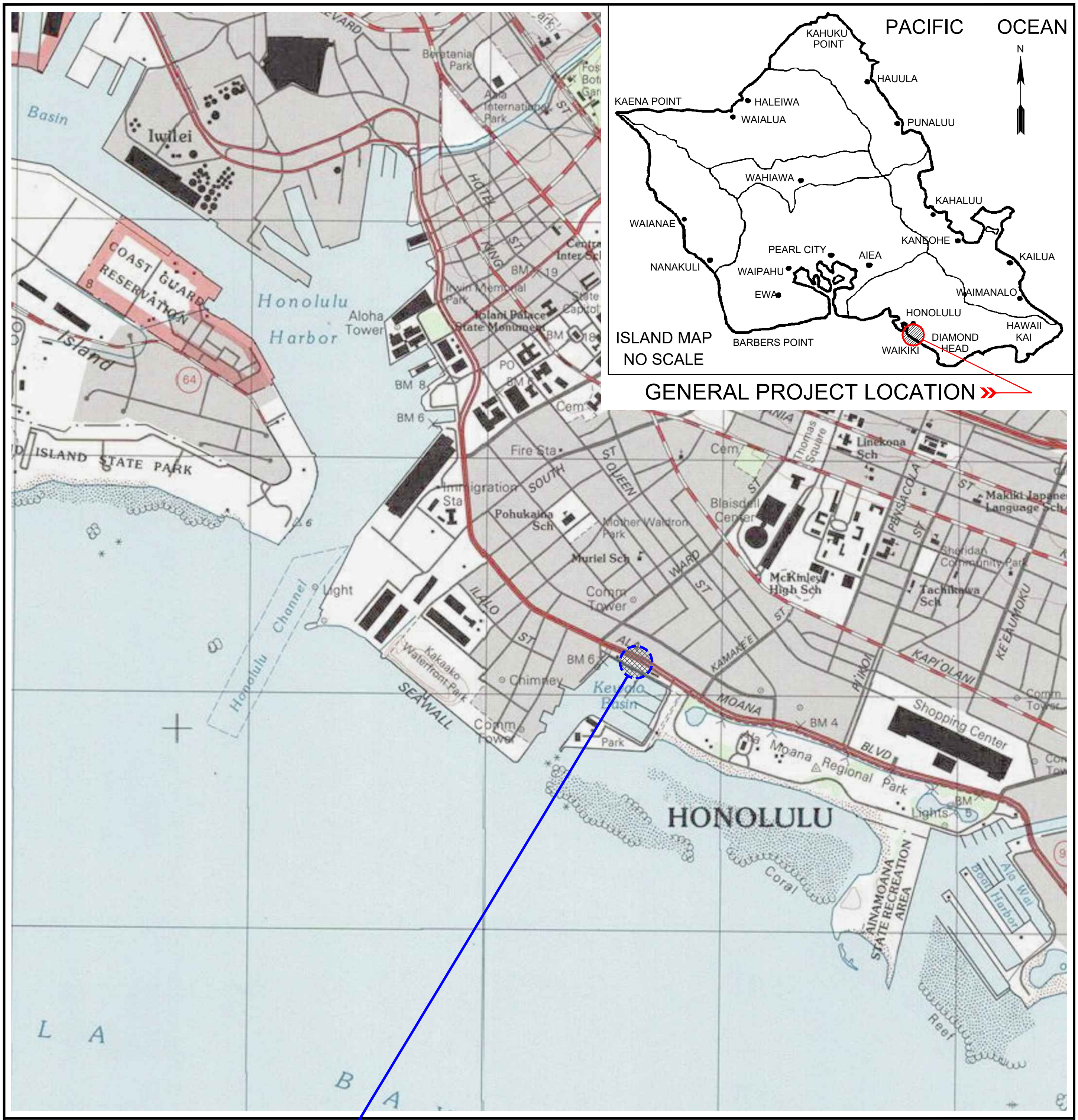
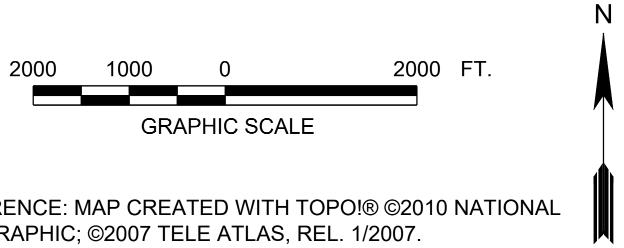
PLATES



PROJECT LOCATION »

PROJECT LOCATION MAP

ALA MOANA BOULEVARD
ELEVATED PEDESTRIAN WALKWAY
FEDERAL AID PROJECT NO. BLD-092-1(029)
HONOLULU, OAHU, HAWAII



GRAPHIC SCALE

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

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Geotechnical Engineering

DATE	DRAWN BY	PLATE
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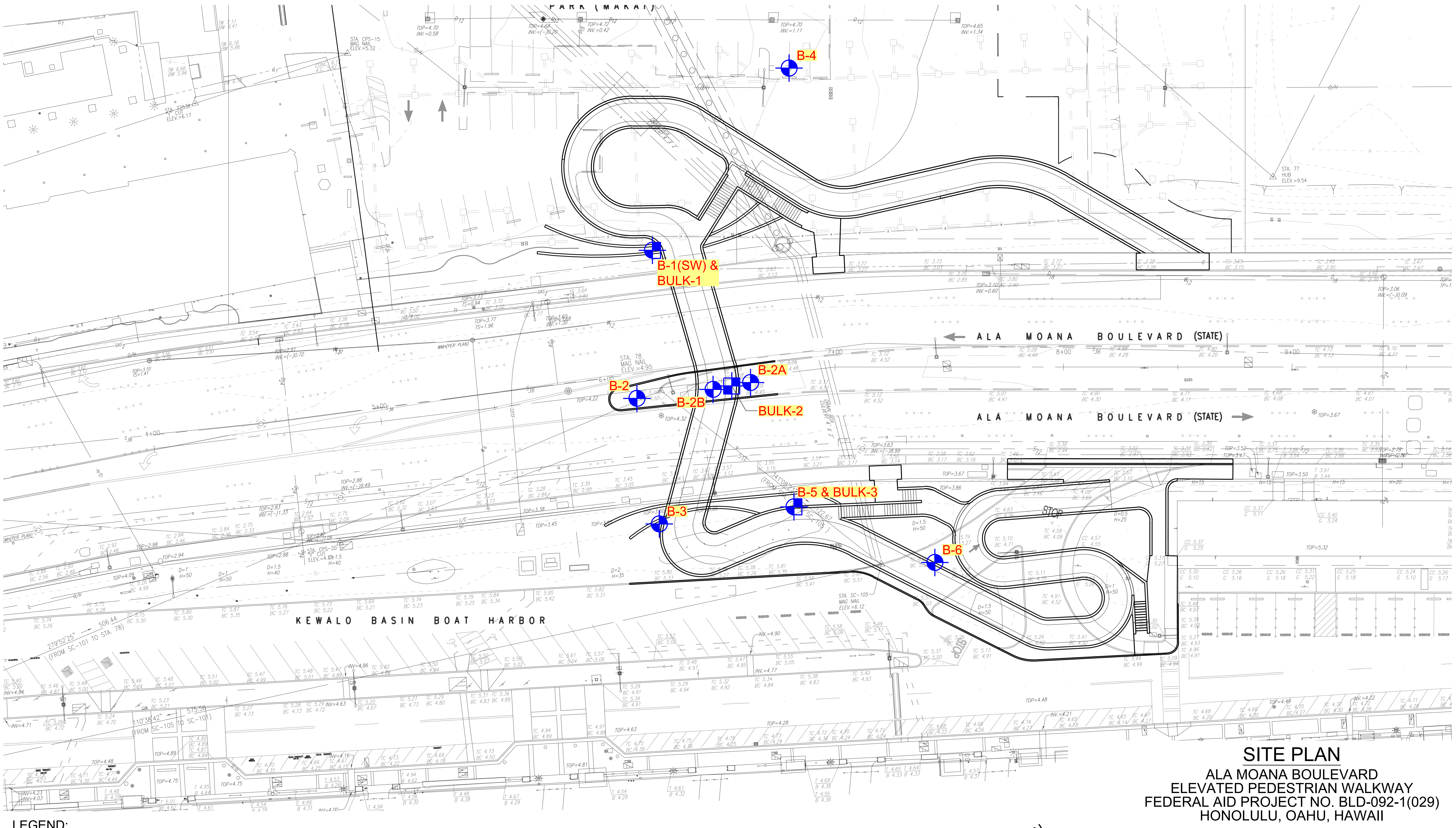

SITE PLAN

ALA MOANA BOULEVARD
ELEVATED PEDESTRIAN WALKWAY
FEDERAL AID PROJECT NO. BLD-092-1(029)
HONOLULU, OAHU, HAWAII

LEGEND:

⊕	APPROXIMATE BORING LOCATION
■	APPROXIMATE BORING LOCATION
SW	SEISMIC SHEAR WAVE VELOCITY TESTING

0 20 40 80 120 FT.

GRAPHIC SCALE

REFERENCE: ALA MOANA BOULEVARD ELEVATED PEDESTRIAN WALKWAY FOUNDATION AND GROUND DISTURBANCE SURVEY PROVIDED BY WSP USA, INC. ON JANUARY 4, 2021.

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Geotechnical Engineering

DATE	DRAWN BY	PLATE
DECEMBER 2020	ASP	
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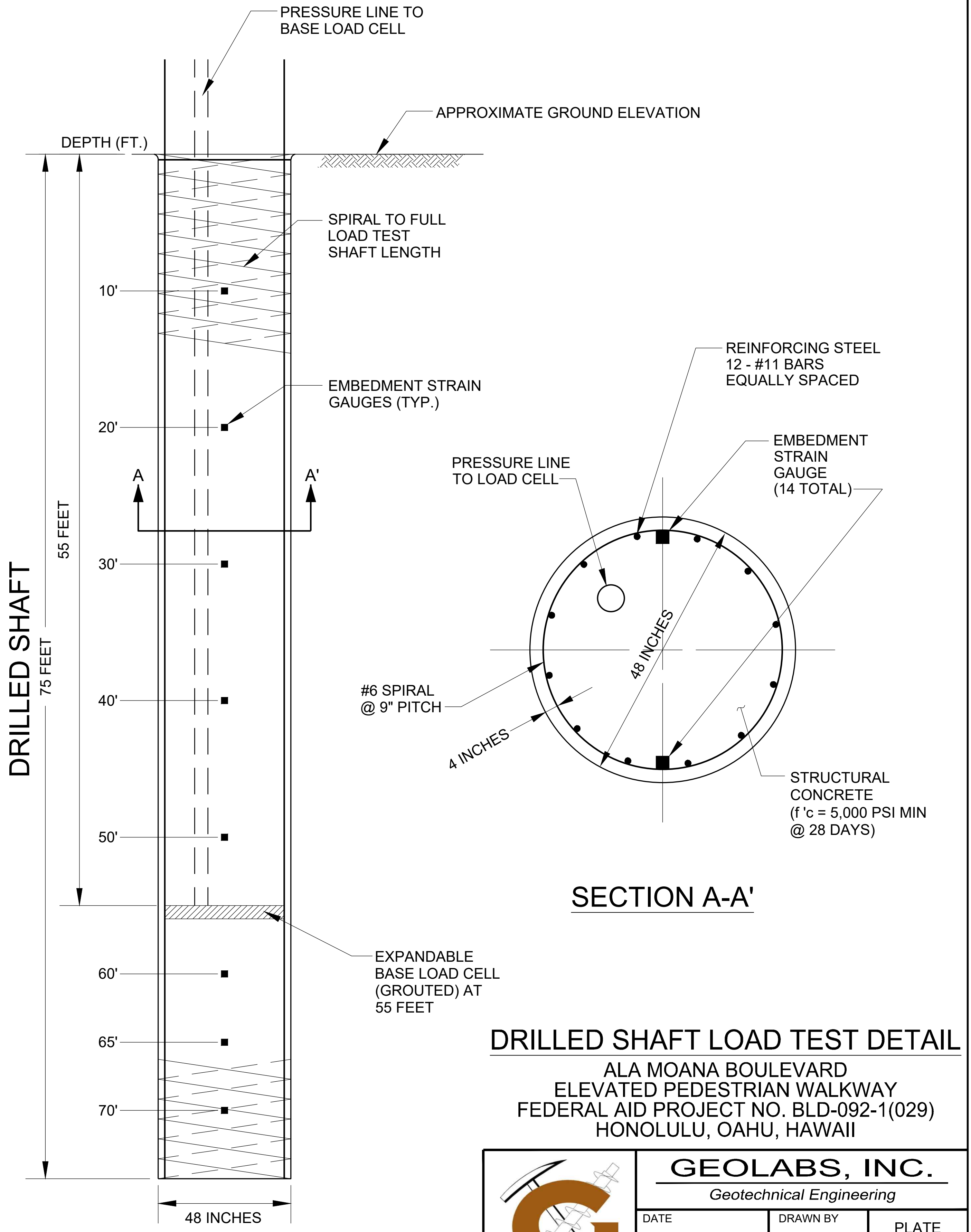

SECTION A-A'

DRILLED SHAFT LOAD TEST DETAIL

ALA MOANA BOULEVARD
ELEVATED PEDESTRIAN WALKWAY
FEDERAL AID PROJECT NO. BLD-092-1(029)
HONOLULU, OAHU, HAWAII

GEOLABS, INC.
Geotechnical Engineering

DATE MARCH 2021	DRAWN BY ASP	PLATE
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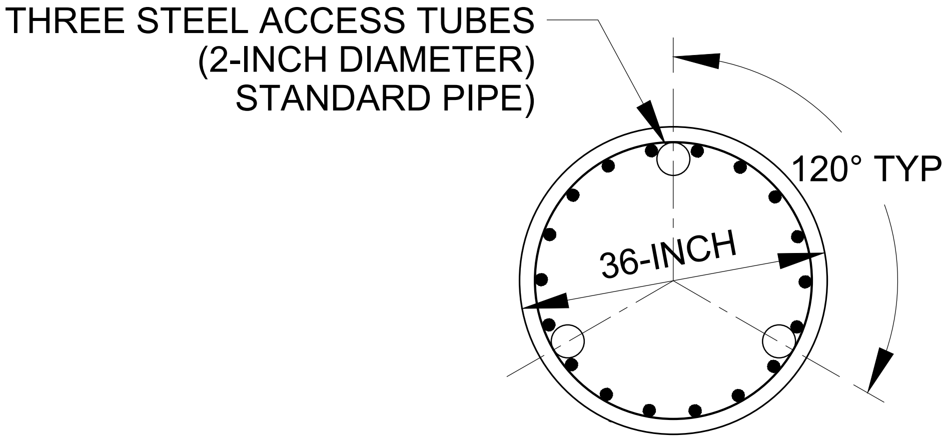

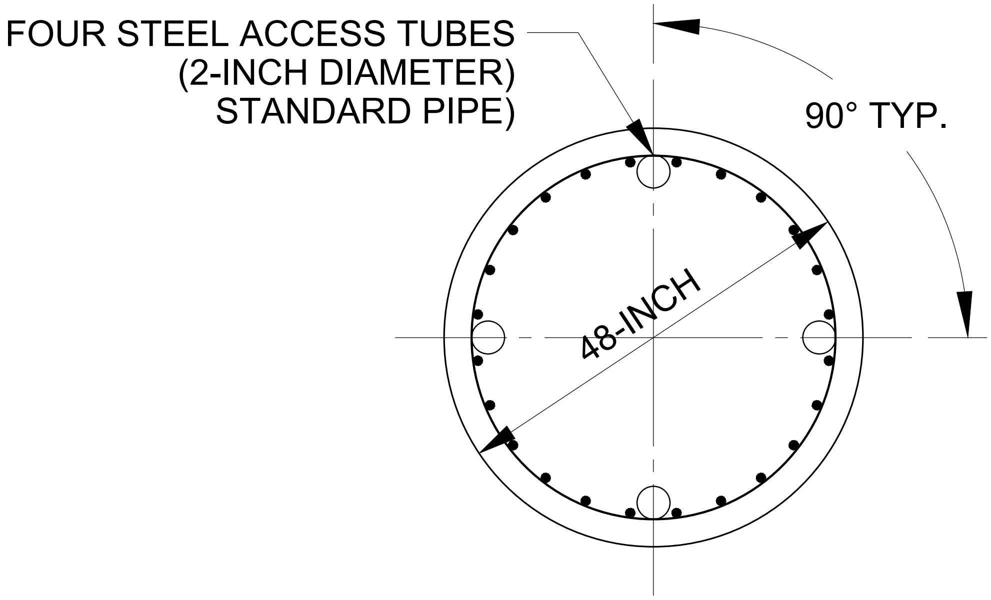

ACCESS TUBES DETAIL FOR CROSSHOLE SONIC LOGGING TEST

ALA MOANA BOULEVARD
ELEVATED PEDESTRIAN WALKWAY
FEDERAL AID PROJECT NO. BLD-092-1(029)
HONOLULU, OAHU, HAWAII

GEOLABS, INC.
Geotechnical Engineering

DATE MARCH 2021	DRAWN BY ASP	PLATE
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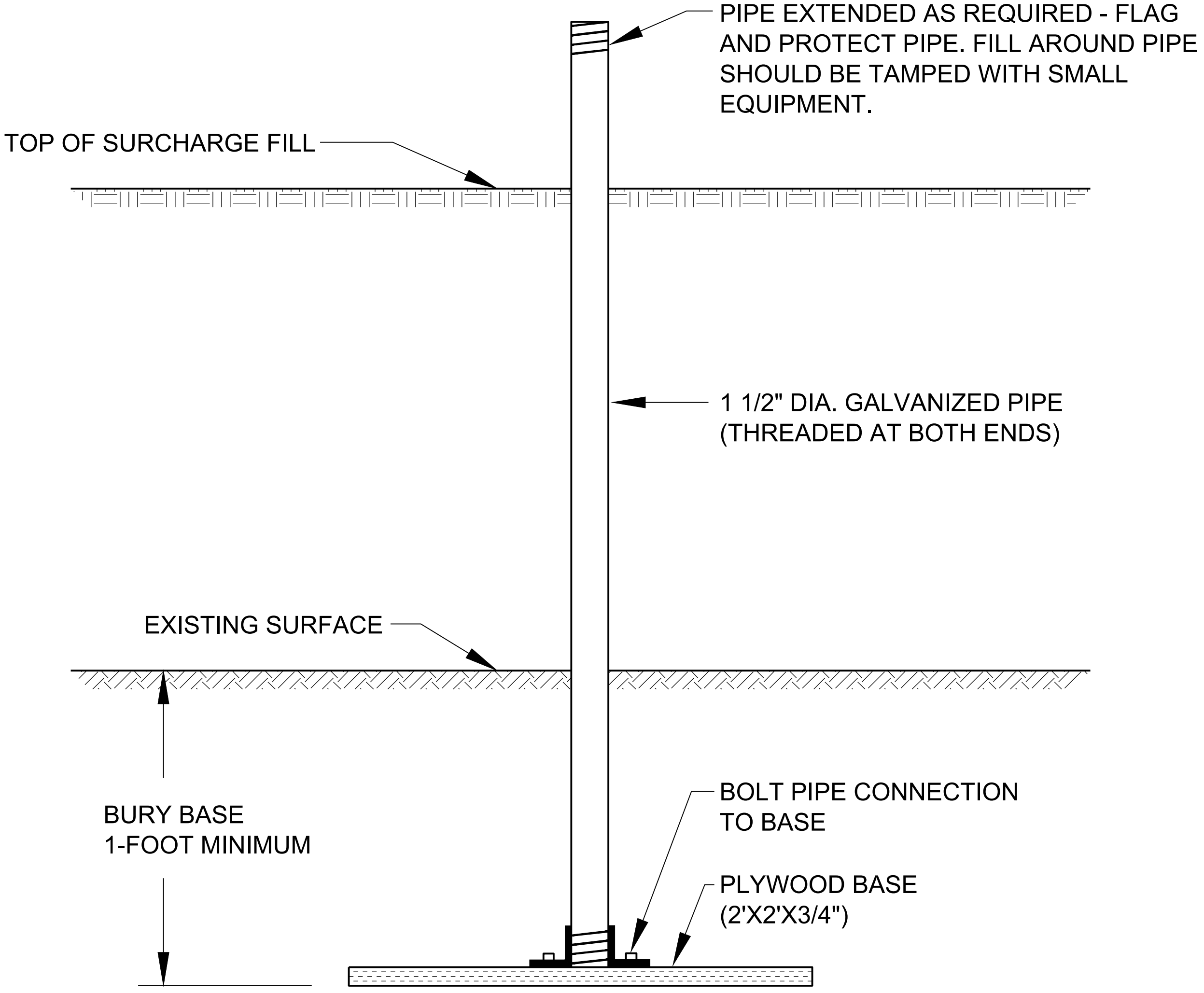

TYPICAL SETTLEMENT GAUGE

ALA MOANA BOULEVARD
ELEVATED PEDESTRIAN WALKWAY
FEDERAL AID PROJECT NO. BLD-092-1(029)
HONOLULU, OAHU, HAWAII

GEOLABS, INC.
Geotechnical Engineering

DATE MARCH 2021	DRAWN BY ASP	PLATE
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APPENDIX A

APPENDIX A

Field Exploration

We explored the subsurface conditions at the proposed Ala Moana Boulevard Elevated Pedestrian Walkway project site by drilling and sampling six borings, designated as Boring Nos. 1 through 6. The borings were extended to depths ranging from about 66.5 to 122.5 feet below the existing ground surface using a truck mounted drill rig equipped with continuous flight augers and rotary coring tools. In addition to the six borings, two additional borings, designated as Boring Nos. 2A and 2B, were drilled and sampled to depths of about 42 and 11.5 feet, respectively, within the Ala Moana Boulevard median to further explore a buried concrete obstruction. Three bulk samples, designated as Bulk Samples No. 1 through 3, of the near-surface soils were obtained at selected locations. The approximate boring and bulk sample locations are shown on the Site Plan, Plate 2. The borings were drilled using truck-mounted drill rigs equipped with continuous flight augers and coring tools.

Our geologist classified the materials encountered in the borings by visual and textural examination in the field in general accordance with ASTM D2488, Standard Practice for Description and Identification of Soils, and monitored the drilling operations on a near-continuous (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 in the Soil Classification Log Key, Plate A-0.2. Graphic representations of the materials encountered are presented on the Logs of Borings, Plates A-1.1 through A-8.2.

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

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

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

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

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

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

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Soil Log Legend

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)

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

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

LEGEND

◩	(2-INCH) O.D. STANDARD PENETRATION TEST
⧗	(3-INCH) O.D. MODIFIED CALIFORNIA SAMPLE
S	SHELBY TUBE SAMPLE
G	GRAB SAMPLE
▮	CORE SAMPLE
▽	WATER LEVEL OBSERVED IN BORING AT TIME OF DRILLING
▼	WATER LEVEL OBSERVED IN BORING AFTER DRILLING
▼	WATER LEVEL OBSERVED IN BORING OVERNIGHT

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

Plate

A-0.1

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Soil Classification Log Key

(with deviations from ASTM D2488)

GEOLABS, INC. CLASSIFICATION*

GRANULAR SOIL (- #200 <50%)

- **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)

**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).*

SOIL_CLASS_LOG_KEY 8115-00.GPJ GEOLABS.GDT 12/7/20

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Rock Log Legend

ROCK DESCRIPTIONS

⁄	**BASALT**	∘	**CONGLOMERATE**
○	**BOULDERS**	▤	**LIMESTONE**
△	**BRECCIA**	⁙	**SANDSTONE**
×	**CLINKER**	×	**SILTSTONE**
⬠	**COBBLES**	▩	**TUFF**
☼	**CORAL**		**VOID/CAVITY**

ROCK DESCRIPTION SYSTEM

ROCK FRACTURE CHARACTERISTICS

The following terms describe general fracture spacing of a rock:

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

DEGREE OF WEATHERING

The following terms describe the chemical weathering of a rock:

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

HARDNESS

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

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

LOG LEGEND FOR ROCK 8115-00.GPJ GEOLABS.GDT 12/7/20

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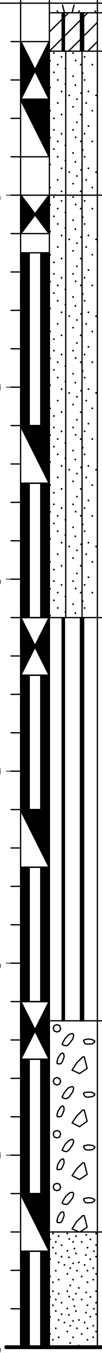
ALA MOANA BOULEVARD
ELEVATED PEDESTRIAN WALKWAY
FEDERAL AID PROJECT NO. BLD-092-1(029)
HONOLULU, OAHU, HAWAII

Log of Boring

1

Approximate Ground Surface Elevation (feet): 4.5 *



Laboratory			Field						
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)	Depth (feet)	USCS	Description
								MH	3-inch **TOPSOIL**
	38	74			3			SM	Brown **CLAYEY SILT** with some sand, very soft, moist (fill)
	31				2				Grayish brown **SILTY SAND (CORALLINE)** with some gravel, very loose, moist (fill)
							▽		
Consol.	24	89			19/6" +10/0" Ref.		▼ 5	SM	Light gray **SILTY SAND (CORALLINE)** with some gravel and cobbles (coralline), medium dense, wet (lagoonal deposit)
			0						
							10		
Sieve - #200 = 13.1%	35				4				grades to loose
			0						
							15		
	30	70			2			ML	Light gray **SANDY SILT** with a little gravel (coralline), very soft (lagoonal deposit)
			0						
							20		
LL=28 PI=3	43				0				
			0						
							25		
	32	81			98			GP	Tan **SANDY GRAVEL (CORALLINE)** with a little cobbles (coralline), very dense (coralline detritus)
			0						
							30		
	36				52				
			29					SP-SM	Tan **GRAVELLY SAND (CORALLINE)** with a little silt and cobbles (coralline), medium dense (coralline detritus)
							35		

Date Started: June 19, 2020
Date Completed: June 20, 2020
Logged By: D. Gremminger
Total Depth: 122.5 feet
Work Order: 8115-00

Water Level: ▽ 4.3 ft. 06/19/2020 0912 HRS
▼ 5.5 ft. 06/20/2020 0820 HRS
Drill Rig: CME-75DR *(Energy Transfer Ratio = 77.3%)*
Drilling Method: 4" Solid-Stem Auger & PQ Coring
Driving Energy: 140 lb. wt., 30 in. drop

Plate

A - 1.1

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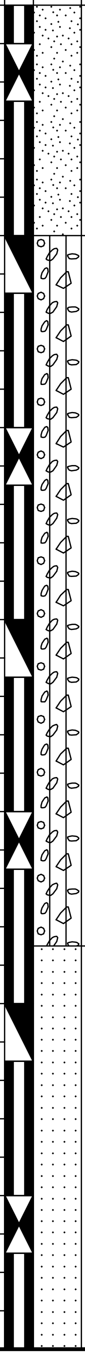
ALA MOANA BOULEVARD
ELEVATED PEDESTRIAN WALKWAY
FEDERAL AID PROJECT NO. BLD-092-1(029)
HONOLULU, OAHU, HAWAII

Log of Boring
1

(Continued from previous plate)



Laboratory			Field								
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)	Depth (feet)	Sample	Graphic	USCS	Description
										SP-SM	
	26	81			22						
			0								
							40				
										GM	Tannish white **SILTY GRAVEL (CORALLINE)** with some sand (coralline), medium dense (coralline detritus)
	26				11						
			0								
							45				
Sieve - #200 = 14.7%	25	90			22						
			0								
							50				
	25				3						grades to very loose locally
			0								
							55				
	26	86			23						
			24	12							
											Tannish white **SANDSTONE**, closely fractured, highly weathered, soft (sandstone formation)
							60				
	21				11						
UC= 250 psi			36	17							
							65				
	27	91			9						
			40	14							
							70				

Date Started:	June 19, 2020
Date Completed:	June 20, 2020
Logged By:	D. Gremminger
Total Depth:	122.5 feet
Work Order:	8115-00

Water Level: ▽ 4.3 ft. 06/19/2020 0912 HRS
▼ 5.5 ft. 06/20/2020 0820 HRS

Drill Rig: CME-75DR *(Energy Transfer Ratio = 77.3%)*

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

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

Plate

A - 1.2

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ALA MOANA BOULEVARD
ELEVATED PEDESTRIAN WALKWAY
FEDERAL AID PROJECT NO. BLD-092-1(029)
HONOLULU, OAHU, HAWAII

Log of Boring
1

(Continued from previous plate)

Laboratory			Field								
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)	Depth (feet)	Sample	Graphic	USCS	Description
	26				7						
			0								
							75				
	24	82			10						
			0								
							80				
Sieve - #200 = 10.4%	20				20					GP-GM	Tannish white **SANDY GRAVEL (CORALLINE)** with a little silt, medium dense (coralline detritus)
			0								
							85				
	25	87			30						
			0								
							90				
	30				10					SP-SM	Tannish white **GRAVELLY SAND (CORALLINE)** with a little silt, medium dense (coralline detritus)
			0								
							95				
	34	81			11						grades to loose
			0								
							100				
	20				46					SP-SM	Tan **GRAVELLY SAND (CORALLINE)** with a little silt and cobbles (coralline), very dense (weathered sandstone)
			0								
							105				

Date Started:	June 19, 2020	Water Level: ▽	4.3 ft. 06/19/2020 0912 HRS
Date Completed:	June 20, 2020	▼	5.5 ft. 06/20/2020 0820 HRS
Logged By:	D. Gremminger	Drill Rig:	CME-75DR *(Energy Transfer Ratio = 77.3%)*
Total Depth:	122.5 feet	Drilling Method:	4" Solid-Stem Auger & PQ Coring
Work Order:	8115-00	Driving Energy:	140 lb. wt., 30 in. drop

Plate

A - 1.3

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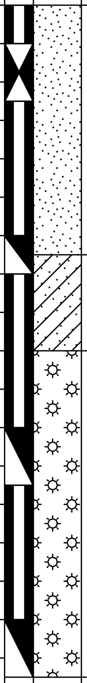
ALA MOANA BOULEVARD
ELEVATED PEDESTRIAN WALKWAY
FEDERAL AID PROJECT NO. BLD-092-1(029)
HONOLULU, OAHU, HAWAII

Log of Boring
1

(Continued from previous plate)



Laboratory			Field						
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)	Depth (feet)	USCS	Description
	20	106			83			SP-SM	grades to dense
			0						
							110		
UC= 1740 psi	36		42	0	40/6" +10/0" Ref.			SC	Brown **CLAYEY SAND** with some gravel, very dense (weathered volcanic tuff)
							115		Whitish tan **CORAL**, closely fractured, moderately weathered, medium hard (coral formation)
	23				22				grades to severely fractured, soft
			0	0					
							120		
	23				18				
									Boring terminated at 122.5 feet * Elevations estimated from Ala Moana Boulevard Elevated Pedestrian Walkway Foundation and Ground Disturbance Survey provided by WSP USA, Inc. on January 4, 2021.
							125		
							130		
							135		
							140		

Date Started:	June 19, 2020	Water Level: ∇	4.3 ft. 06/19/2020 0912 HRS
Date Completed:	June 20, 2020	▼	5.5 ft. 06/20/2020 0820 HRS
Logged By:	D. Gremminger	Drill Rig:	CME-75DR *(Energy Transfer Ratio = 77.3%)*
Total Depth:	122.5 feet	Drilling Method:	4" Solid-Stem Auger & PQ Coring
Work Order:	8115-00	Driving Energy:	140 lb. wt., 30 in. drop

Plate

A - 1.4

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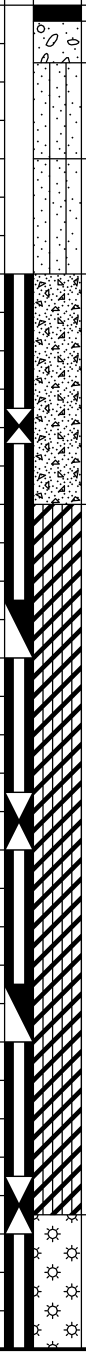
ALA MOANA BOULEVARD
ELEVATED PEDESTRIAN WALKWAY
FEDERAL AID PROJECT NO. BLD-092-1(029)
HONOLULU, OAHU, HAWAII

Log of Boring

2

Approximate Ground Surface Elevation (feet): 4.5 *



Laboratory			Field						
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)	Depth (feet)	USCS	Description
									5-inch **ASPHALT**
								GW-GM	Dark grayish brown **SANDY GRAVEL (BASALTIC)** with a little silt, moist (fill)
							▼	SM	Tannish brown **SILTY SAND** with some gravel, moist (fill)
							▽ 5	SM	Gray **SILTY SAND (CORALLINE)** with some gravel, moist (fill) grades to wet
			100	100					72-inch **CONCRETE**
					30/5"		10		
			16	0					
								CL	Light gray **SANDY CLAY (CORALLINE)** with some gravel (coralline), very soft (lagoonal deposit)
Sieve - #200 = 55.2%	32				2		15		
			0						
	33	96			8		20		grades with a little cobbles (coralline), medium stiff
			0						
	39				2		25		grades to very soft
			0						
	34	85			39		30		
UC= 550 psi			64	0					Tan **CORAL**, closely fractured, moderately weathered, medium hard (coral formation)
							35		

Date Started:	August 17, 2020	Water Level: ▽	5.0 ft. 08/17/2020 1021 HRS
Date Completed:	August 24, 2020	▼	3.0 ft. 08/24/2020 0902 HRS
Logged By:	D. Gremminger	Drill Rig:	CME-75DG1 *(Energy Transfer Ratio = 80.3%)*
Total Depth:	122 feet	Drilling Method:	4" Solid-Stem Auger & PQ Coring
Work Order:	8115-00	Driving Energy:	140 lb. wt., 30 in. drop

Plate

A - 2.1

GEOLABS, INC.

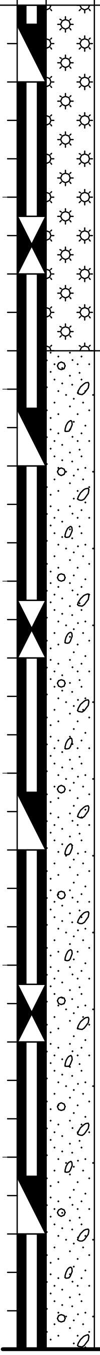
Geotechnical Engineering

ALA MOANA BOULEVARD
ELEVATED PEDESTRIAN WALKWAY
FEDERAL AID PROJECT NO. BLD-092-1(029)
HONOLULU, OAHU, HAWAII

Log of Boring

2

(Continued from previous plate)



Laboratory			Field								
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)	Depth (feet)	Sample	Graphic	USCS	Description
	17				62						
			52	0							grades to severely fractured, highly weathered
							40				
	22	102			65						
			38	0							
										SW	Tan **GRAVELLY SAND (CORALLINE)** with traces of silt, medium dense (coralline detritus)
							45				
	20				28						
			0								
							50				
Direct Shear	15	111			36						
			0								
							55				
	16				21						
			0								
							60				
	17	115			27						
			0								
							65				
LL=NP PI=NP Sieve - #200 = 0.8%	19				7						grades to loose
			0								
							70				

Date Started:	August 17, 2020	Water Level: ▽	5.0 ft. 08/17/2020 1021 HRS
Date Completed:	August 24, 2020	▼	3.0 ft. 08/24/2020 0902 HRS
Logged By:	D. Gremminger	Drill Rig:	CME-75DG1 *(Energy Transfer Ratio = 80.3%)*
Total Depth:	122 feet	Drilling Method:	4" Solid-Stem Auger & PQ Coring
Work Order:	8115-00	Driving Energy:	140 lb. wt., 30 in. drop

Plate

A - 2.2

GEOLABS, INC.

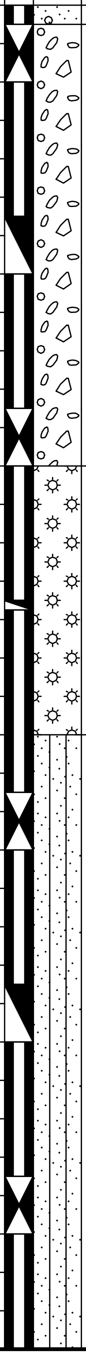
Geotechnical Engineering

ALA MOANA BOULEVARD
ELEVATED PEDESTRIAN WALKWAY
FEDERAL AID PROJECT NO. BLD-092-1(029)
HONOLULU, OAHU, HAWAII

Log of Boring

2

(Continued from previous plate)



Laboratory			Field						
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)	Depth (feet)	USCS	Description
	15	106			43			GP-GM	Whitish tan **SANDY GRAVEL (CORALLINE)** with a little silt, medium dense (coralline detritus)
			0						
							75		
	12				8				grades to loose
			0						
							80		
	18	101			27				grades to medium dense
			67	12					Tannish white **CORAL**, severely fractured, highly to moderately weathered, medium hard (coral formation)
							85		
			18	18	40/3"				grades to moderately fractured, hard locally
								SM	Tan **SILTY SAND (CORALLINE)** with some gravel, loose (coralline detritus)
							90		
	34	78			8				
			0						
							95		
	23				7				
			0						
							100		
					11				
			0						
							105		

Date Started:	August 17, 2020
Date Completed:	August 24, 2020
Logged By:	D. Gremminger
Total Depth:	122 feet
Work Order:	8115-00

Water Level: ▽ 5.0 ft. 08/17/2020 1021 HRS
▼ 3.0 ft. 08/24/2020 0902 HRS

Drill Rig: CME-75DG1 *(Energy Transfer Ratio = 80.3%)*
Drilling Method: 4" Solid-Stem Auger & PQ Coring
Driving Energy: 140 lb. wt., 30 in. drop

Plate

A - 2.3

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Geotechnical Engineering

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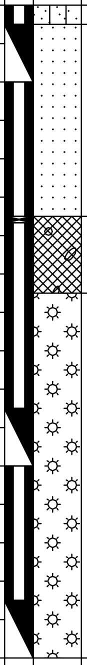
ELEVATED PEDESTRIAN WALKWAY
FEDERAL AID PROJECT NO. BLD-092-1(029)
HONOLULU, OAHU, HAWAII

Log of Boring
2

(Continued from previous plate)



Laboratory: Other Tests	Laboratory: Moisture Content (%)	Laboratory: Dry Density (pcf)	Field: Core Recovery (%)	Field: RQD (%)	Field: Penetration Resistance (blows/foot)	Field: Pocket Pen. (tsf)	Depth (feet)	USCS	Description
	24				10				Tan cemented **SANDSTONE**, severely to closely fractured, moderately weathered, medium hard (sandstone formation)
			57	19					
							110		
			57	10	30/2"				Brown **TUFF**, closely fractured, slightly weathered, medium hard to hard (volcanic tuff)
									Tan **CORAL**, severely fractured, moderately to highly weathered, medium hard (coral)
							115		
	20				65				
			71	0					grades to white
							120		
	26				11				
									Boring terminated at 122 feet
							125		
							130		
							135		
							140		

Date Started: August 17, 2020
Date Completed: August 24, 2020
Logged By: D. Gremminger
Total Depth: 122 feet
Work Order: 8115-00

Water Level: ∇ 5.0 ft. 08/17/2020 1021 HRS
▼ 3.0 ft. 08/24/2020 0902 HRS
Drill Rig: CME-75DG1 *(Energy Transfer Ratio = 80.3%)*
Drilling Method: 4" Solid-Stem Auger & PQ Coring
Driving Energy: 140 lb. wt., 30 in. drop

Plate
A - 2.4

GEOLABS, INC. Geotechnical Engineering	ALA MOANA BOULEVARD ELEVATED PEDESTRIAN WALKWAY FEDERAL AID PROJECT NO. BLD-092-1(029) HONOLULU, OAHU, HAWAII		Log of Boring 2A

Approximate Ground Surface Elevation (feet): 4.5 *

Laboratory			Field								
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)	Depth (feet)	Sample	Graphic	USCS	Description
										CH	2-inch **TOPSOIL**
	30	106			48						Brown **SILTY CLAY** with a little sand and gravel, medium stiff, moist (fill)
										SP	Brownish gray **SAND (BASALTIC)**, medium dense, moist (fill)
						∇	5				
	22	99			11					SM	Gray **SILTY SAND (CORALLINE)** with some gravel (coralline), loose (fill)
			100	0							18-inch **CONCRETE**
							10				
	35				37					SM	Black **SILTY SAND** with some gravel, dense (fill)
			83	48							24-inch **CONCRETE**
										CL	Light gray **SANDY CLAY** with some gravel (coralline), medium stiff (lagoonal deposit)
							15				
	34	104			9						
			0								grades to soft
							20				
LL=37 PI=22	43				4						
			0								
							25				
	39	84			4						
			29	0							
											grades with some cobbles (coralline)
							30				Tan **CORAL**, severely fractured, moderately to highly weathered, soft (coral formation)
	40				36						
			19	0							
											grades to medium hard
							35				

Date Started:	August 25, 2020	Water Level: ∇	4.7 ft. 08/25/2020 0948 HRS	Plate
Date Completed:	August 25, 2020			
Logged By:	D. Gremminger	Drill Rig:	CME-75DR *(Energy Transfer Ratio = 77.3%)*	A - 3.1
Total Depth:	42 feet	Drilling Method:	4" Solid-Stem Auger & PQ Coring	
Work Order:	8115-00	Driving Energy:	140 lb. wt., 30 in. drop	

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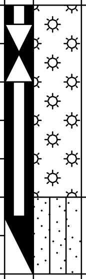
ALA MOANA BOULEVARD
ELEVATED PEDESTRIAN WALKWAY
FEDERAL AID PROJECT NO. BLD-092-1(029)
HONOLULU, OAHU, HAWAII

Log of Boring

2A

Laboratory			Field								
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)	Depth (feet)	Sample	Graphic	USCS	(Continued from previous plate) Description
	24	105			80						
			45	0							
											grades to hard locally
	34				29		40			SM	Whitish tan **SILTY SAND (CORALLINE)** with some gravel (coralline), medium dense (coralline detritus)
											Boring terminated at 42 feet
							45				
							50				
							55				
							60				
							65				
							70				

Date Started:	August 25, 2020	Water Level: ⊻	4.7 ft. 08/25/2020 0948 HRS
Date Completed:	August 25, 2020		
Logged By:	D. Gremminger	Drill Rig:	CME-75DR *(Energy Transfer Ratio = 77.3%)*
Total Depth:	42 feet	Drilling Method:	4" Solid-Stem Auger & PQ Coring
Work Order:	8115-00	Driving Energy:	140 lb. wt., 30 in. drop

Plate

A - 3.2

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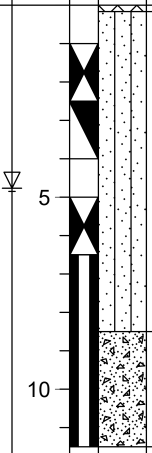
ALA MOANA BOULEVARD
ELEVATED PEDESTRIAN WALKWAY
FEDERAL AID PROJECT NO. BLD-092-1(029)
HONOLULU, OAHU, HAWAII

Log of Boring

2B

Approximate Ground Surface Elevation (feet): 4.5 *

Laboratory			Field						
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)	Depth (feet)	USCS	Description
							0	SM	2-inch **TOPSOIL WITH GRASS**
					5				Brownish tan **SILTY SAND** with some gravel, very loose, moist (fill)
					2				
					10		5		grades to tannish gray, loose
			60	80					
									CONCRETE
							10		
									Boring terminated at 11.5 feet
							15		
							20		
							25		
							30		
							35		



Date Started: August 17, 2020
Date Completed: August 17, 2020
Logged By: D. Gremminger
Total Depth: 11.5 feet
Work Order: 8115-00

Water Level: ⊻ 4.8 ft. 08/17/2020 0000 HRS
Drill Rig: CME-75DG2 *(Energy Transfer Ratio = 91.5%)*
Drilling Method: 4" Solid-Stem Auger & PQ Coring
Driving Energy: 140 lb. wt., 30 in. drop

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ALA MOANA BOULEVARD
ELEVATED PEDESTRIAN WALKWAY
FEDERAL AID PROJECT NO. BLD-092-1(029)
HONOLULU, OAHU, HAWAII

Log of Boring

3

Approximate Ground Surface Elevation (feet): 5.5 *

Laboratory			Field								
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)	Depth (feet)	Sample	Graphic	USCS	Description
										SM	2-inch **TOPSOIL**
	18	85			13						Brownish tan **SILTY SAND (CORALLINE)** with a little gravel (coralline), loose to medium dense, moist (fill)
	18				22	▼					
						∇					
	35	87			11		5			GM	Grayish tan **SILTY GRAVEL (CORALLINE)** with some sand and a little cobbles (coralline), loose (fill)
			0								
							10				
	39				10						grades to medium dense
			0								
										SM	Tannish gray **SILTY SAND (CORALLINE)** with some gravel and cobbles (coralline), very loose (lagoonal deposit)
							15				
					3						
	40	80			0						
LL=NP PI=NP	32				0		20				
			0								
							25				
					WOR/ 24"						
			0								
							30				
	30				10					SP-SM	Tan **GRAVELLY SAND (CORALLINE)** with a little silt, medium dense (coralline detritus)
			71	17							
							35				

Date Started:	July 7, 2020	Water Level: ∇	4.8 ft. 07/07/2020 1003 HRS
Date Completed:	July 8, 2020	▼	3.0 ft. 07/08/2020 0844 HRS
Logged By:	D. Gremminger	Drill Rig:	CME-75DG2 *(Energy Transfer Ratio = 91.5%)*
Total Depth:	122 feet	Drilling Method:	4" Solid-Stem Auger & PQ Coring
Work Order:	8115-00	Driving Energy:	140 lb. wt., 30 in. drop

Plate

A - 5.1

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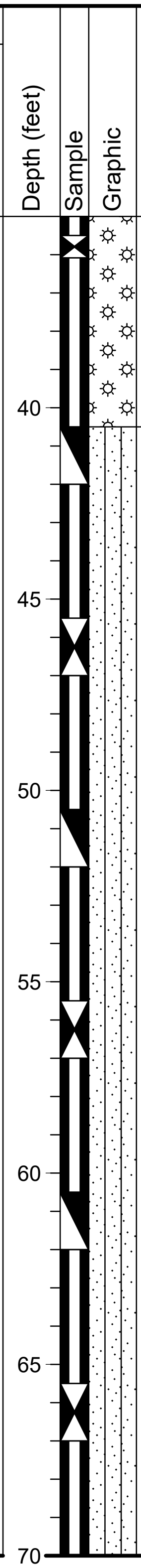
ALA MOANA BOULEVARD
ELEVATED PEDESTRIAN WALKWAY
FEDERAL AID PROJECT NO. BLD-092-1(029)
HONOLULU, OAHU, HAWAII

Log of Boring

3

(Continued from previous plate)

Laboratory			Field					
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)	USCS	Description
UC= 650 psi	25	94	75	19	25/1"			Tannish white **CORAL**, severely to closely fractured, moderately weathered, medium hard (coral formation) grades to moderately fractured, slightly weathered locally
	29		0		19		SM	Tannish white **SILTY SAND (CORALLINE)** with some gravel (coralline), medium dense (coralline detritus)
	54	74	26		20			grades with pockets of brown clayey silt
	25		0		17			grades without clayey silt pockets
	22	86	48		13			grades to moderately cemented
Sieve - #200 = 23.3%	25		55		15			
	20	95	52		21			



Date Started: July 7, 2020
Date Completed: July 8, 2020
Logged By: D. Gremminger
Total Depth: 122 feet
Work Order: 8115-00

Water Level: ∇ 4.8 ft. 07/07/2020 1003 HRS
▼ 3.0 ft. 07/08/2020 0844 HRS
Drill Rig: CME-75DG2 *(Energy Transfer Ratio = 91.5%)*
Drilling Method: 4" Solid-Stem Auger & PQ Coring
Driving Energy: 140 lb. wt., 30 in. drop

Plate

A - 5.2

GEOLABS, INC.

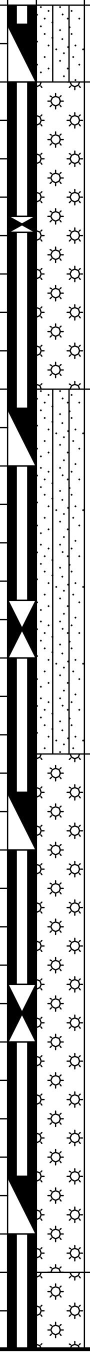
Geotechnical Engineering

ALA MOANA BOULEVARD
ELEVATED PEDESTRIAN WALKWAY
FEDERAL AID PROJECT NO. BLD-092-1(029)
HONOLULU, OAHU, HAWAII

Log of Boring

3

(Continued from previous plate)



Laboratory			Field						
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)	Depth (feet)	USCS	Description
	16				16			SM	
UC= 1100 psi			93	50					Tannish white **CORAL**, closely fractured, moderately weathered, medium hard (coral formation)
							75		
	15	92	89	65	45/5"				grades to moderately fractured
							80	SM	White **SILTY SAND (CORALLINE)** with some gravel (coralline), medium dense (coralline detritus)
	25				13				
			0						
							85		
	38	82			24				
			43						
							90		Grayish tan **CORAL**, severely to closely fractured, highly weathered, soft (coral formation)
	37				12				
			71	17					
							95		
	25	96			11				grades to extremely weathered
			74	57					
							100		
	29				9				
			62	0					
									Tan **CORAL**, severely fractured, highly weathered, soft (coral formation)
							105		

Date Started:	July 7, 2020
Date Completed:	July 8, 2020
Logged By:	D. Gremminger
Total Depth:	122 feet
Work Order:	8115-00

Water Level: ∇ 4.8 ft. 07/07/2020 1003 HRS
▼ 3.0 ft. 07/08/2020 0844 HRS

Drill Rig: CME-75DG2 *(Energy Transfer Ratio = 91.5%)*
Drilling Method: 4" Solid-Stem Auger & PQ Coring
Driving Energy: 140 lb. wt., 30 in. drop

Plate

A - 5.3

GEOLABS, INC.

Geotechnical Engineering

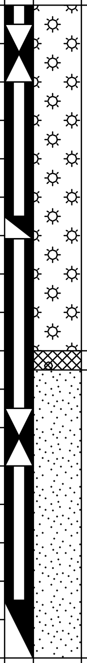
ALA MOANA BOULEVARD
ELEVATED PEDESTRIAN WALKWAY
FEDERAL AID PROJECT NO. BLD-092-1(029)
HONOLULU, OAHU, HAWAII

Log of Boring

3

(Continued from previous plate)

Laboratory			Field								
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)	Depth (feet)	Sample	Graphic	USCS	Description
	12	103			80						grades to medium hard grades to reddish tan, moderately weathered
			71	21							
							110				
	21		57	28	15/1"						
											Brown **TUFF**, severely fractured, moderately weathered, hard (volcanic tuff)
	18	98			11		115			SP-SM	Whitish tan **GRAVELLY SAND (CORALLINE)** with a little silt and cobbles (coralline), medium dense (coralline detritus)
			0								
	16				22		120				
											Boring terminated at 122 feet
							125				
							130				
							135				
							140				



Date Started:	July 7, 2020
Date Completed:	July 8, 2020
Logged By:	D. Gremminger
Total Depth:	122 feet
Work Order:	8115-00

Water Level: ∇ 4.8 ft. 07/07/2020 1003 HRS
▼ 3.0 ft. 07/08/2020 0844 HRS

Drill Rig: CME-75DG2 *(Energy Transfer Ratio = 91.5%)*

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

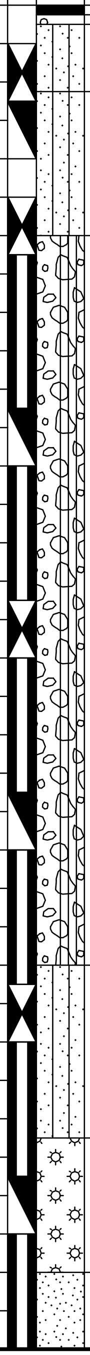
Driving Energy: 140 lb. wt., 30 in. drop

Plate

A - 5.4

GEOLABS, INC. Geotechnical Engineering	ALA MOANA BOULEVARD ELEVATED PEDESTRIAN WALKWAY FEDERAL AID PROJECT NO. BLD-092-1(029) HONOLULU, OAHU, HAWAII	**Log of Boring** 4

Laboratory			Field								
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)	Depth (feet)	Sample	Graphic	USCS	Approximate Ground Surface Elevation (feet): 5 * **Description**
											3-inch **ASPHALTIC CONCRETE**
	12	101			44					GP-GM	Gray **SANDY GRAVEL** with a little silt, dry (fill)
										SM	Brown **SILTY SAND** with a little gravel, medium dense, moist (fill)
Sieve - #200 = 12.9%	19				12					SM	Grayish tan **SILTY SAND (CORALLINE)** with some gravel, medium dense, moist (fill)
						⊻					
	20	103			53		5				
			0							GP-GM	Gray **SANDY GRAVEL (CORALLINE)** with a little silt, medium dense (lagoonal deposit)
											grades to very loose
							10				
	35				4						
			0								
							15				
	29	85			8						grades with a little cobbles (coralline), loose
			0								
							20				
Sieve - #200 = 11.0%	32				12						grades to medium dense
			0								
							25			SM	Tan **SILTY SAND (CORALLINE)** with some gravel, medium dense (coralline detritus)
	37	79			34						
			33	12							
							30				Tan **CORAL**, severely fractured, highly weathered, soft (coral formation)
	32				16						
			36	10							
										SP-SM	
							35				



Date Started:	June 12, 2020	Water Level: ⊻	4.5 ft. 06/12/2020 0917 HRS
Date Completed:	June 12, 2020		
Logged By:	D. Gremminger	Drill Rig:	CME-75DR *(Energy Transfer Ratio = 77.3%)*
Total Depth:	77 feet	Drilling Method:	4" Solid-Stem Auger & PQ Coring
Work Order:	8115-00	Driving Energy:	140 lb. wt., 30 in. drop

Plate

A - 6.1

	GEOLABS, INC. Geotechnical Engineering	ALA MOANA BOULEVARD ELEVATED PEDESTRIAN WALKWAY FEDERAL AID PROJECT NO. BLD-092-1(029) HONOLULU, OAHU, HAWAII	Log of Boring 4

(Continued from previous plate)

Laboratory			Field								
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)	Depth (feet)	Sample	Graphic	USCS	Description
	26				27					SP-SM	Tan **SAND (CORALLINE)** with a little silt and gravel (coralline), medium dense (coralline detritus)
			0								
							40				
	25				29						grades to whitish tan
			0								
							45				
	32	84			49						
			0								
							50				
	29				4					SM	Tannish white **SILTY SAND (CORALLINE)**, loose (coralline detritus)
			0								
							55				
	25	90			16						
			0								
							60				
	25				7						
			0								
							65				
Direct Shear Sieve - #200 = 15.7%	25	96			28						grades with some gravel, medium dense
			0								
							70				

Date Started:	June 12, 2020	Water Level: ⊻	4.5 ft. 06/12/2020 0917 HRS	Plate
Date Completed:	June 12, 2020			
Logged By:	D. Gremminger	Drill Rig:	CME-75DR *(Energy Transfer Ratio = 77.3%)*	
Total Depth:	77 feet	Drilling Method:	4" Solid-Stem Auger & PQ Coring	A - 6.2
Work Order:	8115-00	Driving Energy:	140 lb. wt., 30 in. drop	

GEOLABS, INC.

Geotechnical Engineering

ALA MOANA BOULEVARD
ELEVATED PEDESTRIAN WALKWAY
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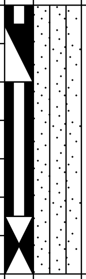
Log of Boring

4



Laboratory			Field								
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)	Depth (feet)	Sample	Graphic	USCS	(Continued from previous plate) Description
	27				9					SM	grades to tan, loose
			0								
							75				
					23						grades to medium dense
											Boring terminated at 77 feet
							80				
							85				
							90				
							95				
							100				
							105				

Date Started:	June 12, 2020	Water Level: ⊻	4.5 ft. 06/12/2020 0917 HRS
Date Completed:	June 12, 2020		
Logged By:	D. Gremminger	Drill Rig:	CME-75DR *(Energy Transfer Ratio = 77.3%)*
Total Depth:	77 feet	Drilling Method:	4" Solid-Stem Auger & PQ Coring
Work Order:	8115-00	Driving Energy:	140 lb. wt., 30 in. drop

Plate

A - 6.3

GEOLABS, INC. Geotechnical Engineering	ALA MOANA BOULEVARD ELEVATED PEDESTRIAN WALKWAY FEDERAL AID PROJECT NO. BLD-092-1(029) HONOLULU, OAHU, HAWAII	Log of Boring 5

Approximate Ground Surface Elevation (feet): 5.5 *

Laboratory			Field								
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)	Depth (feet)	Sample	Graphic	USCS	Description
										SC	3-inch **TOPSOIL**
	26	67			6						Reddish brown **CLAYEY SAND** with some gravel, very loose, moist (fill)
	31				5					SM	Grayish tan **SILTY SAND (CORALLINE)**, loose, wet (fill)
							▽				
Consol.	19	103			37		5			SM	Gray **SILTY SAND (CORALLINE)** with some gravel, medium dense (fill)
			0								
										SM	Gray **SILTY SAND (CORALLINE)** with some gravel (coralline), loose (lagoonal deposit)
							10				
	45				4						
			0								
							15				
					9						grades with a little cobbles (coralline)
			0								
							20				
	30				3						grades to very loose
			0								
							25				
	34	65			7						grades with traces of shells, loose
			0								
							30				
	37				18						
			33	0						SP-SM	Tan **GRAVELLY SAND (CORALLINE)** with a little silt, medium dense (coralline detritus)
							35				

Date Started:	June 11, 2020	Water Level: ▽	4.1 ft. 06/11/2020 0849 HRS	Plate
Date Completed:	June 11, 2020			
Logged By:	D. Gremminger	Drill Rig:	CME-75DR *(Energy Transfer Ratio = 77.3%)*	A - 7.1
Total Depth:	77 feet	Drilling Method:	4" Solid-Stem Auger & PQ Coring	
Work Order:	8115-00	Driving Energy:	140 lb. wt., 30 in. drop	

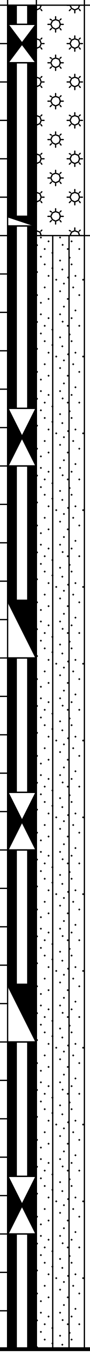
GEOLABS, INC.

Geotechnical Engineering

ALA MOANA BOULEVARD
ELEVATED PEDESTRIAN WALKWAY
FEDERAL AID PROJECT NO. BLD-092-1(029)
HONOLULU, OAHU, HAWAII

Log of Boring 5

(Continued from previous plate)



Laboratory: Other Tests	Laboratory: Moisture Content (%)	Laboratory: Dry Density (pcf)	Field: Core Recovery (%)	Field: RQD (%)	Field: Penetration Resistance (blows/foot)	Field: Pocket Pen. (tsf)	Depth (feet)	USCS	Description
	27	77			65/6"				Whitish tan cemented **CORAL**, severely fractured, highly to moderately weathered, medium hard (coral formation)
			19	0					
							40		
			0		20/3"				
								SM	Whitish tan **SILTY SAND (CORALLINE)** with some gravel (coralline), medium dense (coralline detritus)
							45		
Sieve - #200 = 21.2%	29	82			18				
			0						
							50		
	32				37				grades to dense locally
			0						
							55		
	28	82			18				
			0						
							60		
	27				2				grades to very loose
			0						
							65		
Sieve - #200 = 20.0%	25	85			4				
			0						
							70		

Date Started: June 11, 2020
Date Completed: June 11, 2020
Logged By: D. Gremminger
Total Depth: 77 feet
Work Order: 8115-00

Water Level: ⊻ 4.1 ft. 06/11/2020 0849 HRS
Drill Rig: CME-75DR *(Energy Transfer Ratio = 77.3%)*
Drilling Method: 4" Solid-Stem Auger & PQ Coring
Driving Energy: 140 lb. wt., 30 in. drop

Plate A - 7.2

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Geotechnical Engineering

ALA MOANA BOULEVARD
ELEVATED PEDESTRIAN WALKWAY
FEDERAL AID PROJECT NO. BLD-092-1(029)
HONOLULU, OAHU, HAWAII

Log of Boring

5

Laboratory			Field								
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)	Depth (feet)	Sample	Graphic	USCS	(Continued from previous plate) Description
	22				6					SM	grades to loose
			0								
							75				
	21	92			25						grades to medium dense
											Boring terminated at 77 feet
							80				
							85				
							90				
							95				
							100				
							105				

Date Started:	June 11, 2020	Water Level: ⊽	4.1 ft. 06/11/2020 0849 HRS
Date Completed:	June 11, 2020		
Logged By:	D. Gremminger	Drill Rig:	CME-75DR *(Energy Transfer Ratio = 77.3%)*
Total Depth:	77 feet	Drilling Method:	4" Solid-Stem Auger & PQ Coring
Work Order:	8115-00	Driving Energy:	140 lb. wt., 30 in. drop

Plate

A - 7.3

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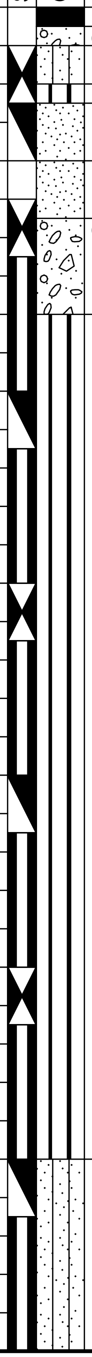
ALA MOANA BOULEVARD
ELEVATED PEDESTRIAN WALKWAY
FEDERAL AID PROJECT NO. BLD-092-1(029)
HONOLULU, OAHU, HAWAII

Log of Boring
6

Approximate Ground Surface Elevation (feet): 5 *

Laboratory			Field						
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)	Depth (feet)	USCS	Description
									6-inch **ASPHALTIC CONCRETE**
	13	93			30			GW-GM	Gray **SANDY GRAVEL (BASALTIC)** with a little silt, dry (fill)
	13				14			SM	Tan **SILTY SAND (CORALLINE)** with a little gravel (coralline), dense, moist (fill)
								ML	Brown **SANDY SILT** with a little gravel, very stiff, moist (fill)
								SP	Black **SAND**, medium dense, moist (fill)
Sieve - #200 = 9.8%	17	111			78		⊻ 5	SP	Tan **GRAVELLY SAND (CORALLINE)**, medium dense, moist (fill)
			0					GW-GM	Gray **SANDY GRAVEL (CORALLINE)** with a little silt, dense to medium dense (fill)
LL=28 PI=5	37				0		10	ML	Gray **SANDY SILT** with some gravel (coralline), very soft (lagoonal deposit)
			0						
					0		15		
			0						
	41				WOH/ 18"		20		
			0						
	25	76			2		25		
			0						
	37				15		30	SM	Tan **SILTY SAND (CORALLINE)** with some gravel (coralline), medium dense (coralline detritus)
			0						
							35		



Date Started:	June 10, 2020	Water Level: ⊻	5.1 ft. 06/10/2020 0925 HRS
Date Completed:	June 10, 2020		
Logged By:	D. Gremminger	Drill Rig:	CME-75DR *(Energy Transfer Ratio = 77.3%)*
Total Depth:	66.5 feet	Drilling Method:	4" Solid-Stem Auger & PQ Coring
Work Order:	8115-00	Driving Energy:	140 lb. wt., 30 in. drop

Plate

A - 8.1

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ALA MOANA BOULEVARD
ELEVATED PEDESTRIAN WALKWAY
FEDERAL AID PROJECT NO. BLD-092-1(029)
HONOLULU, OAHU, HAWAII

Log of Boring
6

(Continued from previous plate)

Laboratory			Field								
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)	Depth (feet)	Sample	Graphic	USCS	Description
	27	97			54					GP-GM	Tan **SANDY GRAVEL (CORALLINE)** with a little silt, medium dense to dense (coralline detritus)
			0								
	34				19		40			SM	Tan **SILTY SAND (CORALLINE)** with some gravel (coralline), medium dense (coralline detritus)
			0								
	32	87			15		45				grades to loose
			0								
Sieve - #200 = 17.7%	25				6		50				grades to whitish tan
			0								
	23	90			16		55				
			0								
	27				2		60				grades to very loose locally
			0								
	28	89			10		65				
											Boring terminated at 66.5 feet
							70				

Date Started: June 10, 2020
Date Completed: June 10, 2020
Logged By: D. Gremminger
Total Depth: 66.5 feet
Work Order: 8115-00

Water Level: ⊻ 5.1 ft. 06/10/2020 0925 HRS
Drill Rig: CME-75DR *(Energy Transfer Ratio = 77.3%)*
Drilling Method: 4" Solid-Stem Auger & PQ Coring
Driving Energy: 140 lb. wt., 30 in. drop

APPENDIX B

APPENDIX B

Seismic Shear Wave Velocity Test

Seismic shear wave velocity profiling of the subsurface materials at the elevated pedestrian walkway was performed using Seismic Cone Penetration Testing (SCPT) equipment. The purpose of the seismic shear wave velocity profiling of the subsurface materials was to analyze the seismic design considerations more closely for the project. Shear wave velocity testing using seismic cone penetration test equipment was performed at a selected boring location, designated as B-1(SW), as shown on the Site Plan, Plate 2. The seismic shear wave velocity profiling was performed at various depths extending to a depth of about 111.7 feet below the existing ground surface.

In order to conduct the seismic shear wave velocity test in the boring, the test boring was advanced utilizing rotary coring methods to the maximum depth of the boring. Log of the materials encountered in the boring are presented on the Logs of Borings in Appendices A and B. After the boring was advanced to the maximum depth of the borehole, the bored hole was backfilled with 0.25-inch diameter coated bentonite pellets. The temporary casing from the coring operations was used as a tremie pipe to place the bentonite pellets starting from the bottom and advancing upward. When the bentonite pellets are in contact with the groundwater in the borehole, the pellets start to hydrate slowly. As the bentonite pellets hydrate, they swell and soften. The probe was then pushed through the softened bentonite extending to a depth of about 111.7 feet below the existing ground surface using seismic cone penetration testing equipment (SCPT).

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

The seismic shear wave velocities measured and the weighted average seismic shear wave velocity calculated for the top 100 feet of the soil profile at the boring location are presented on Plates B-1.1 through B-2.2. The weighted average shear wave velocity was calculated based on the average shear wave velocity method described in the AASHTO 2020 LRFD Bridge Design Specifications (9th Edition).

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ALA MOANA BOULEVARD
ELEVATED PEDESTRIAN WALKWAY
FEDERAL AID PROJECT NO. BLD-092-1(029)
HONOLULU, OAHU, HAWAII

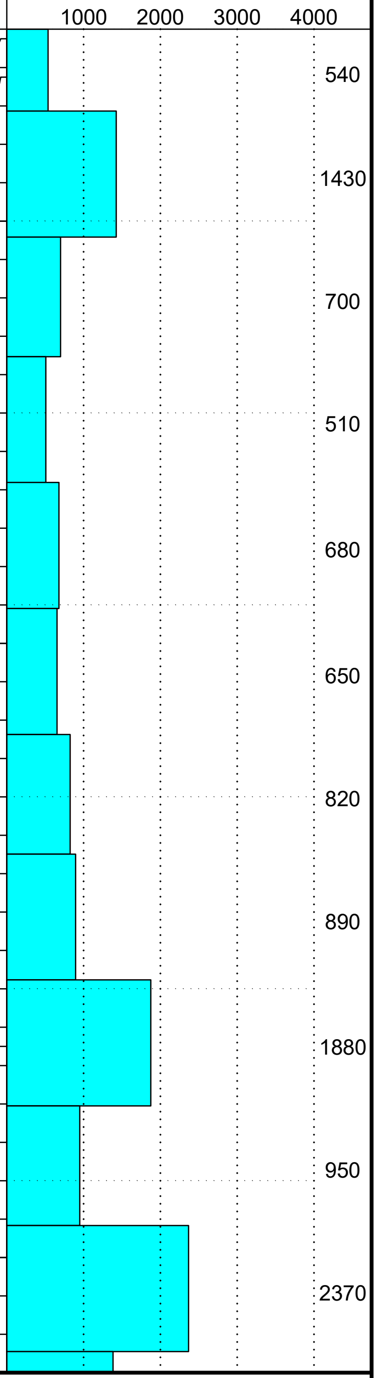
Data Plot of Boring

1

Approximate Ground Surface Elevation (feet): 4.5 *

Core Recovery (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)	Depth (feet)	USCS	Description
				MH	3-inch **TOPSOIL**
	3			SM	Brown **CLAYEY SILT** with some sand, very soft, moist (fill)
	2				Grayish brown **SILTY SAND (CORALLINE)** with some gravel, very loose, moist (fill)
		▽			
	19/6" +10/0" Ref.	▼	5	SM	Light gray **SILTY SAND (CORALLINE)** with some gravel and cobbles (coralline), medium dense, wet (lagoonal deposit)
0					
			10		
	4				grades to loose
0					
			15		
	2			ML	Light gray **SANDY SILT** with a little gravel (coralline), very soft (lagoonal deposit)
0					
			20		
	0				
0					
			25		
	98			GP	Tan **SANDY GRAVEL (CORALLINE)** with a little cobbles (coralline), very dense (coralline detritus)
0					
			30		
	52				
29				SP-SM	Tan **GRAVELLY SAND (CORALLINE)** with a little silt and cobbles (coralline), medium dense (coralline detritus)
			35		

SHEAR WAVE VELOCITY (feet/sec)



Date Started: June 19, 2020
Date Completed: June 20, 2020
Logged By: D. Gremminger
Total Depth: 122.5 feet
Work Order: 8115-00

Water Level: ▽ 4.3 ft. 06/19/2020 0912 HRS
▼ 5.5 ft. 06/20/2020 0820 HRS
Drill Rig: CME-75DR
Drilling Method: 4" Solid-Stem Auger & PQ Coring
Driving Energy: 140 lb. wt., 30 in. drop

Plate

B - 1.1

GEOLABS, INC.

Geotechnical Engineering

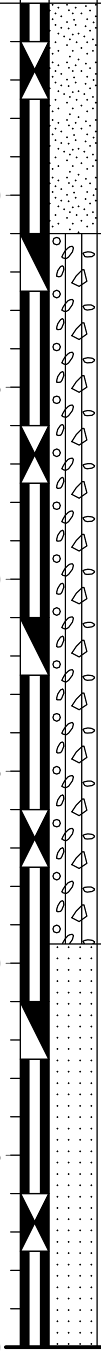
ALA MOANA BOULEVARD
ELEVATED PEDESTRIAN WALKWAY
FEDERAL AID PROJECT NO. BLD-092-1(029)
HONOLULU, OAHU, HAWAII

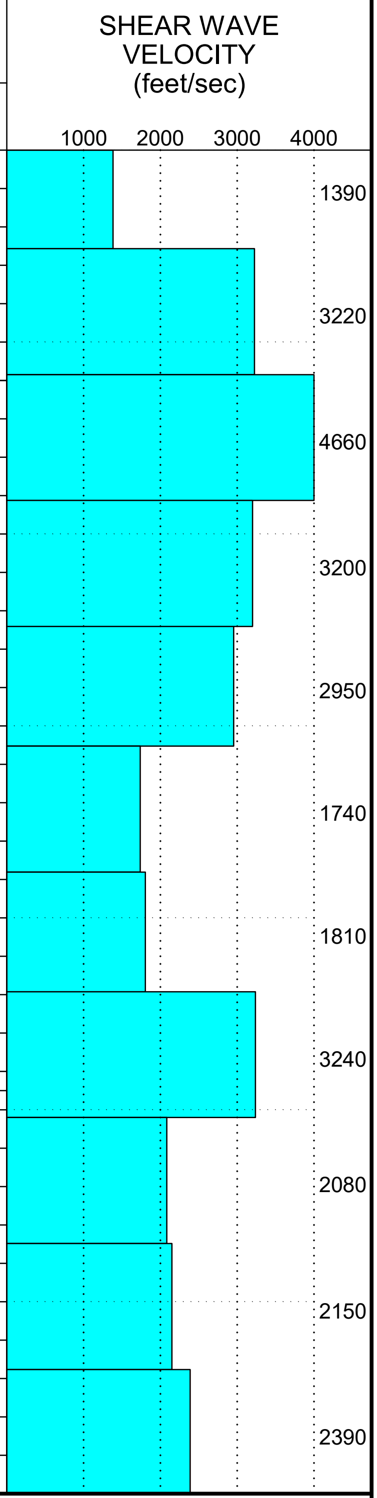
Data Plot of Boring

1

(Continued from previous plate)

Core Recovery (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)	Depth (feet)	USCS	Description
	22			SP-SM	
0					
			40		
	11			GM	Tannish white **SILTY GRAVEL (CORALLINE)** with some sand (coralline), medium dense (coralline detritus)
0					
			45		
	22				
0					
			50		
	3				grades to very loose locally
0					
			55		
	23				
24					
					Tannish white **SANDSTONE**, closely fractured, highly weathered, soft (sandstone formation)
			60		
	11				
36					
			65		
	9				
40					
			70		





Date Started:	June 19, 2020	Water Level: ∇	4.3 ft. 06/19/2020 0912 HRS
Date Completed:	June 20, 2020	▼	5.5 ft. 06/20/2020 0820 HRS
Logged By:	D. Gremminger	Drill Rig:	CME-75DR
Total Depth:	122.5 feet	Drilling Method:	4" Solid-Stem Auger & PQ Coring
Work Order:	8115-00	Driving Energy:	140 lb. wt., 30 in. drop

Plate

B - 1.2

GEOLABS, INC.

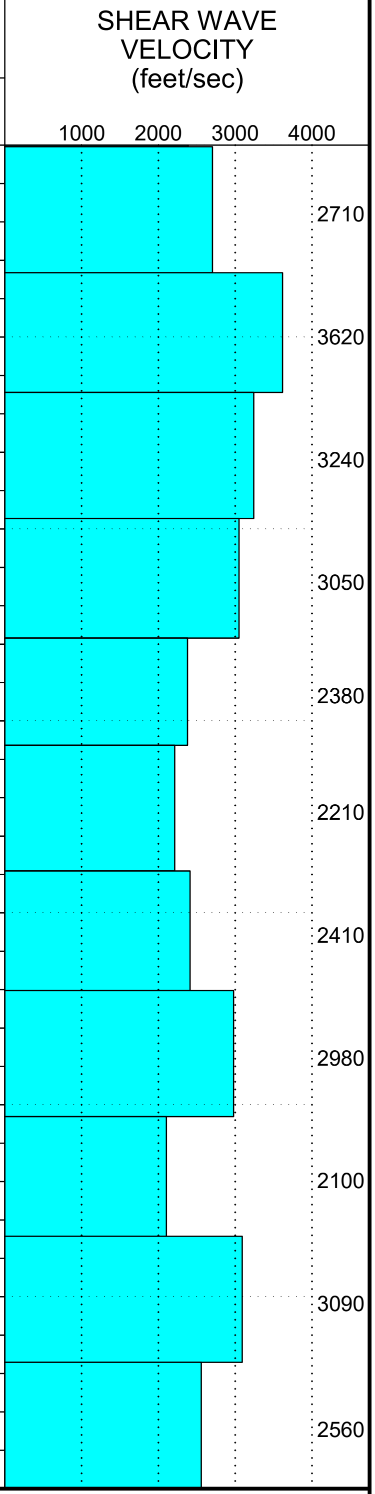
Geotechnical Engineering

ALA MOANA BOULEVARD
ELEVATED PEDESTRIAN WALKWAY
FEDERAL AID PROJECT NO. BLD-092-1(029)
HONOLULU, OAHU, HAWAII

Data Plot of Boring
1

(Continued from previous plate)

Core Recovery (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)	Depth (feet)	USCS	Description
0	7				
			75		
0	10				
			80		
0	20			GP-GM	Tannish white **SANDY GRAVEL (CORALLINE)** with a little silt, medium dense (coralline detritus)
			85		
0	30				
			90		
0	10			SP-SM	Tannish white **GRAVELLY SAND (CORALLINE)** with a little silt, medium dense (coralline detritus)
			95		
0	11				grades to loose
			100		
0	46			SP-SM	Tan **GRAVELLY SAND (CORALLINE)** with a little silt and cobbles (coralline), very dense (weathered sandstone)
			105		



Date Started: June 19, 2020
Date Completed: June 20, 2020
Logged By: D. Gremminger
Total Depth: 122.5 feet
Work Order: 8115-00

Water Level: ∇ 4.3 ft. 06/19/2020 0912 HRS
▼ 5.5 ft. 06/20/2020 0820 HRS
Drill Rig: CME-75DR
Drilling Method: 4" Solid-Stem Auger & PQ Coring
Driving Energy: 140 lb. wt., 30 in. drop

Plate
B - 1.3

GEOLABS, INC.

Geotechnical Engineering

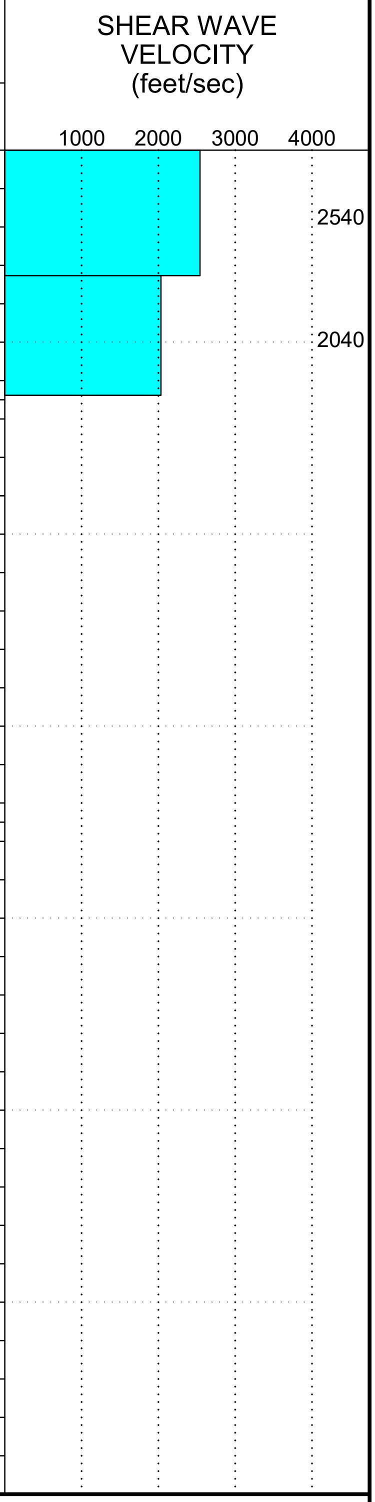
ALA MOANA BOULEVARD
ELEVATED PEDESTRIAN WALKWAY
FEDERAL AID PROJECT NO. BLD-092-1(029)
HONOLULU, OAHU, HAWAII

Data Plot of Boring

1

(Continued from previous plate)

Core Recovery (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)	Depth (feet)	Sample	Graphic	USCS	Description
	83					SP-SM	grades to dense
0							
			110				
42	40/6" +10/0" Ref.					SC	Brown **CLAYEY SAND** with some gravel, very dense (weathered volcanic tuff)
							Whitish tan **CORAL**, closely fractured, moderately weathered, medium hard (coral formation)
			115				
	22						grades to severely fractured, soft
0							
			120				
	18						
							Boring terminated at 122.5 feet * Elevations estimated from Ala Moana Boulevard Elevated Pedestrian Walkway Foundation and Ground Disturbance Survey provided by WSP USA, Inc. on January 4, 2021.
			125				
			130				
			135				
			140				



Date Started:	June 19, 2020	Water Level: ▽	4.3 ft. 06/19/2020 0912 HRS
Date Completed:	June 20, 2020	▼	5.5 ft. 06/20/2020 0820 HRS
Logged By:	D. Gremminger	Drill Rig:	CME-75DR
Total Depth:	122.5 feet	Drilling Method:	4" Solid-Stem Auger & PQ Coring
Work Order:	8115-00	Driving Energy:	140 lb. wt., 30 in. drop

Plate

B - 1.4

SHEAR WAVE VELOCITY TEST RESULTS

ALA MOANA BOULEVARD
ELEVATED PEDESTRIAN WALKWAY
FEDERAL AID PROJECT NO. BLD-092-1(029)
HONOLULU, OAHU, HAWAII

B-1

Depth (From)	Depth (To)	Layer Thickness (d_i)	Estimated Shear Wave Velocity (V_{si})	Average Travel Time (d_i/V_{si})
(feet)	(feet)	(feet)	(feet/second)	(milliseconds)
0.0	2.1	2.1	538	3.96
2.1	5.4	3.3	1,425	2.30
5.4	8.5	3.1	698	4.47
8.5	11.8	3.3	510	6.43
11.8	15.1	3.3	680	4.82
15.1	18.4	3.3	654	5.02
18.4	21.5	3.1	824	3.78
21.5	24.8	3.3	894	3.67
24.8	28.1	3.3	1,875	1.75
28.1	31.2	3.1	949	3.28
31.2	34.4	3.3	2,368	1.39
34.4	37.6	3.1	1,385	2.25
37.6	40.8	3.3	3,223	1.02
40.8	44.1	3.3	4,656	0.70
44.1	47.4	3.3	3,198	1.03
47.4	50.5	3.1	2,953	1.06
50.5	53.8	3.3	1,737	1.89
53.8	56.9	3.1	1,805	1.73
56.9	60.2	3.3	3,237	1.01
60.2	63.5	3.3	2,083	1.58
63.5	66.8	3.3	2,152	1.52
66.8	70.0	3.3	2,389	1.37
70.0	73.3	3.3	2,705	1.21
73.3	76.4	3.1	3,616	0.86
76.4	79.7	3.3	3,242	1.01
79.7	82.8	3.1	3,050	1.02
82.8	85.6	2.8	2,379	1.17
85.6	88.9	3.3	2,213	1.48
88.9	92.0	3.1	2,413	1.29
92.0	95.3	3.3	2,978	1.10
95.3	98.4	3.1	2,103	1.48
98.4	101.7	3.3	3,091	1.06

SHEAR WAVE VELOCITY TEST RESULTS

ALA MOANA BOULEVARD
ELEVATED PEDESTRIAN WALKWAY
FEDERAL AID PROJECT NO. BLD-092-1(029)
HONOLULU, OAHU, HAWAII

B-1				
Depth (From)	Depth (To)	Layer Thickness (d_i)	Estimated Shear Wave Velocity (V_{si})	Average Travel Time (d_i/V_{si})
(feet)	(feet)	(feet)	(feet/second)	(milliseconds)
101.7	105.0	3.3	2,560	1.28
105.0	108.3	3.3	2,541	1.29
108.3	111.4	3.1	2,035	1.53
TOTAL		111.4		71.84
Standard Weighted Average			2,166	feet/second
Computed $V_{s100'}$ Using AASHTO LRFD 2020 Formula			1,502	feet/second

APPENDIX C

APPENDIX C

Laboratory Tests

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

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

Fifteen Sieve Analysis tests (ASTM C117 and C136), including one hydrometer test (ASTM D7928), was performed on selected soil samples to evaluate the gradation characteristics of the soils and to aid in soil classification. Graphic presentations of the grain size distributions are provided on Plates C-2 through C-4.

Two Direct Shear tests (ASTM D3080) were performed on selected samples to evaluate the shear strength characteristics of the materials tested. The test results are presented on Plates C-5 and C-6.

Two Consolidation tests with time rates (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 C-7 and C-8.

Five Uniaxial Compression tests (ASTM D7012, Method C) were performed on selected core runs to evaluate the uniaxial compressive strength of the rock cores encountered. Test results are presented on Plate C-9.

One Unconfined Compression test (ASTM C39) was performed on a selected concrete core to evaluate the unconfined compressive strength of the buried concrete obstruction encountered. Compressive strength test results for the selected concrete core are presented on Plate C-10.

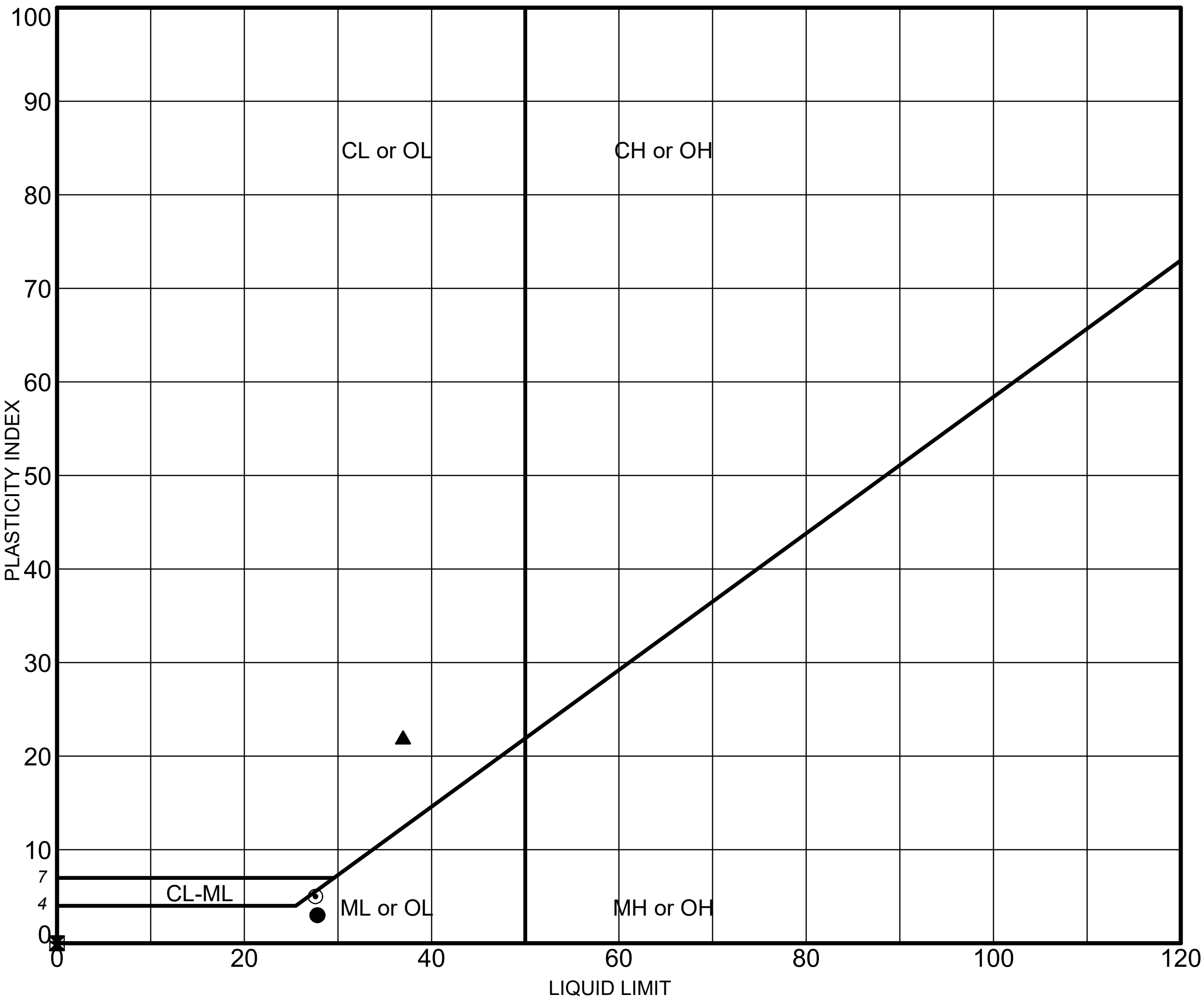
Three laboratory California Bearing Ratio tests (ASTM D1883) were performed on bulk samples of the near-surface soils near optimum moisture content to evaluate the pavement support characteristics of the soils. The test results are presented on Plates C-11 through C-13.

One laboratory Resistance (R) Value test (ASTM D2844) was performed by Ninyo & Moore on a selected bulk sample (BULK-2) of the near-surface soils to evaluate the pavement support characteristics of the soils. The test results are presented on Plate C-14.

One Modified Proctor Compaction test (ASTM D1557 Method A) was performed on a selected bulk sample of the near-surface soils to evaluate the dry density and moisture content relationship. The test results are presented on Plate C-15.

Two set of corrosivity tests, including pH (ASTM G51 or Method 9045C), minimum resistivity (ASTM G57 or SM 2510B), Chloride Content (EPA 300.0), and Sulfate Content (EPA 300.0) tests, was performed by our office and Eurofins TestAmerica on selected soil samples obtained from our field exploration. The test results are presented on Plate C-16.

Three sets of Moisture, Ash, and Organic content tests (ASTM D2974) were performed on selected samples as an aid in the classification and evaluation of organics content of the materials encountered. The test results are summarized on Plate C-17.



	Sample	Depth (ft)	LL	PL	PI	Description
●	B-1	21.0-22.5	28	25	3	Light gray sandy silt (ML) with a little gravel
⧗	B-2	65.5-67.0	NP	NP	NP	Tan gravelly sand (SW) with traces of silt
▲	B-2A	20.5-22.0	37	15	22	Light gray sandy clay (CL) with some gravel
★	B-3	19.5-21.5	NP	NP	NP	Tannish gray silty sand (NP) with some gravel
⦿	B-6	10.0-11.5	28	23	5	Gray sandy silt (ML) with some gravel

NP = NON-PLASTIC

G ATTERBERG PI-100 LL-120 8115-00.GPJ GEOLABS.GDT 3/8/21

GEOLABS, INC.

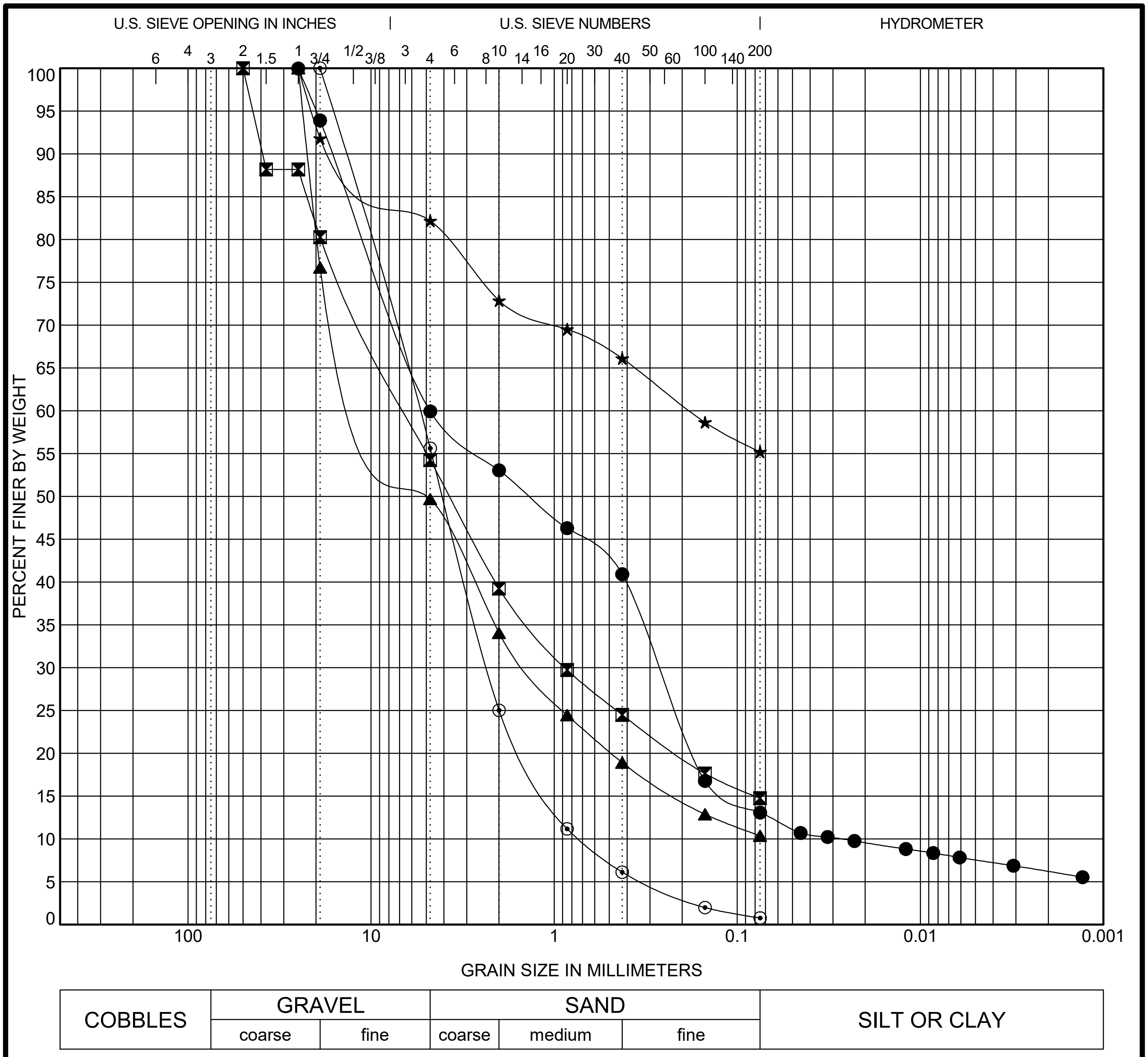
GEOTECHNICAL ENGINEERING

W.O. 8115-00

ATTERBERG LIMITS TEST RESULTS - ASTM D4318

ALA MOANA BOULEVARD
ELEVATED PEDESTRIAN WALKWAY
FEDERAL AID PROJECT NO. BLD-092-1(029)
HONOLULU, OAHU, HAWAII

Plate
C - 1



	Sample	Depth (ft)	Description	LL	PL	PI	Cc	Cu
●	B-1	11.0-12.5	Light gray silty sand (SM) with some gravel				0.5	173.9
⊠	B-1	46.0-47.5	Tannish white silty gravel (GM) with some sand					
▲	B-1	81.0-82.5	Tannish white sandy gravel (GP-GM) with a little silt				3.5	118.5
★	B-2	15.5-17.0	Light gray sandy clay (CL) with some gravel					
⊙	B-2	65.5-67.0	Tan gravelly sand (SW) with traces of silt	NP	NP	NP	1.3	7.5

	Sample	Depth (ft)	D100 (mm)	D60 (mm)	D30 (mm)	D10 (mm)	%Gravel	%Sand	%Fine
●	B-1	11.0-12.5	25	4.765	0.266	0.027	40.0	46.9	13.1
⊠	B-1	46.0-47.5	50	6.454	0.872		45.8	39.5	14.7
▲	B-1	81.0-82.5	25	8.063	1.395		50.3	39.3	10.4
★	B-2	15.5-17.0	25	0.181			17.8	27.0	55.2
⊙	B-2	65.5-67.0	19	5.447	2.303	0.724	44.4	54.9	0.8

G GRAIN SIZE MOD 8115-00.GPJ GEOLABS.GDT 3/8/21

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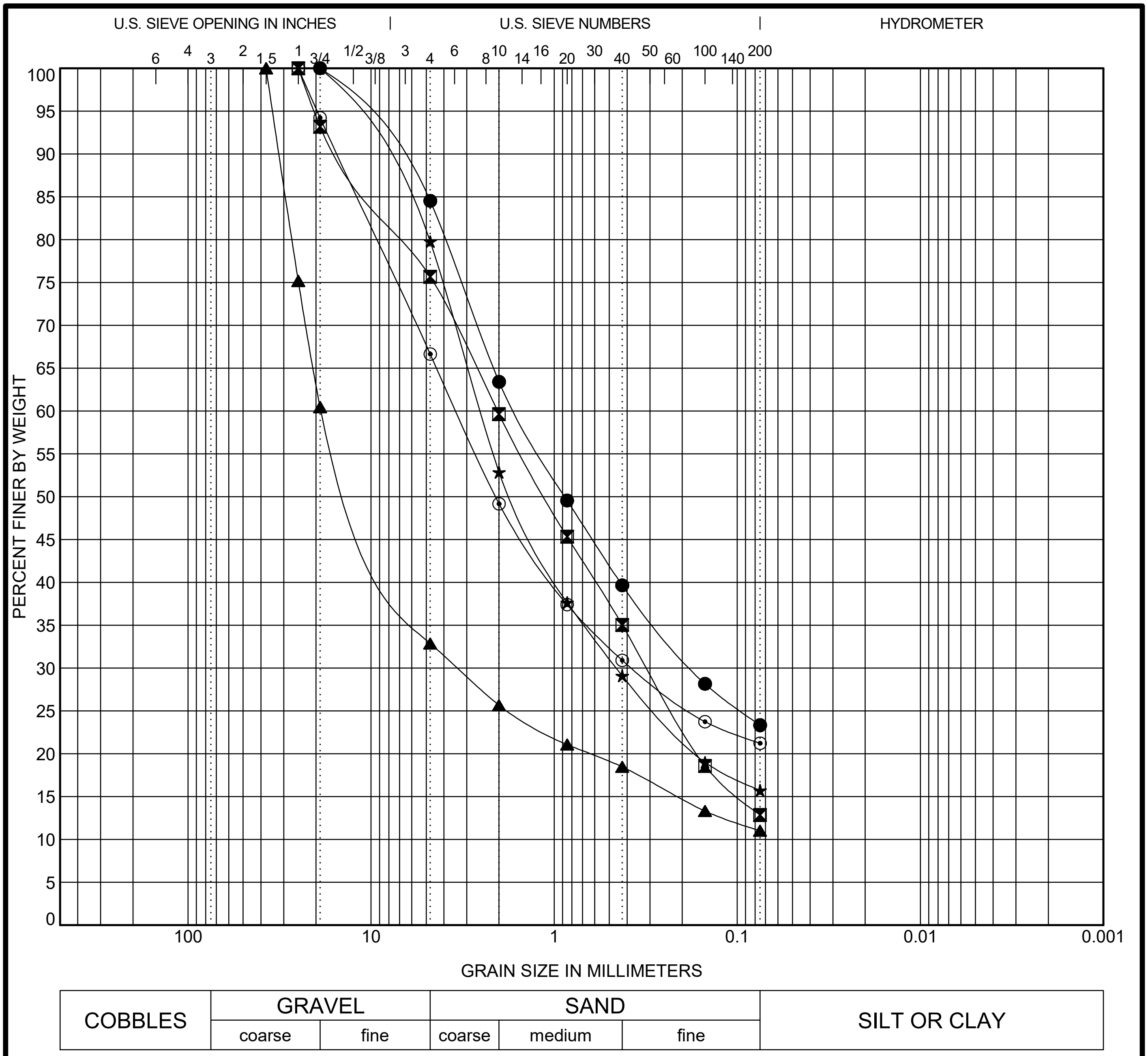
GRAIN SIZE DISTRIBUTION - ASTM C117 & C136

ALA MOANA BOULEVARD
ELEVATED PEDESTRIAN WALKWAY
FEDERAL AID PROJECT NO. BLD-092-1(029)
HONOLULU, OAHU, HAWAII

Plate
C - 2



	Sample	Depth (ft)	Description	LL	PL	PI	Cc	Cu
●	B-3	60.5-62.0	Tannish white silty sand (SM) with some gravel					
⊠	B-4	2.5-4.0	Grayish tan silty sand (SM) with some gravel					
▲	B-4	20.5-22.0	Gray sandy gravel (GP-GM) with a little silt				11.0	335.1
★	B-4	65.5-67.0	Tannish white silty sand (SM) with some gravel					
⊙	B-5	45.5-47.0	Whitish tan silty sand (SM) with some gravel					

	Sample	Depth (ft)	D100 (mm)	D60 (mm)	D30 (mm)	D10 (mm)	%Gravel	%Sand	%Fine
●	B-3	60.5-62.0	19	1.621	0.177		15.5	61.2	23.3
⊠	B-4	2.5-4.0	25	2.042	0.309		24.3	62.8	12.9
▲	B-4	20.5-22.0	37.5	18.582	3.372		67.1	21.9	11.0
★	B-4	65.5-67.0	19	2.517	0.458		20.2	64.1	15.7
⊙	B-5	45.5-47.0	25	3.42	0.373		33.4	45.4	21.2

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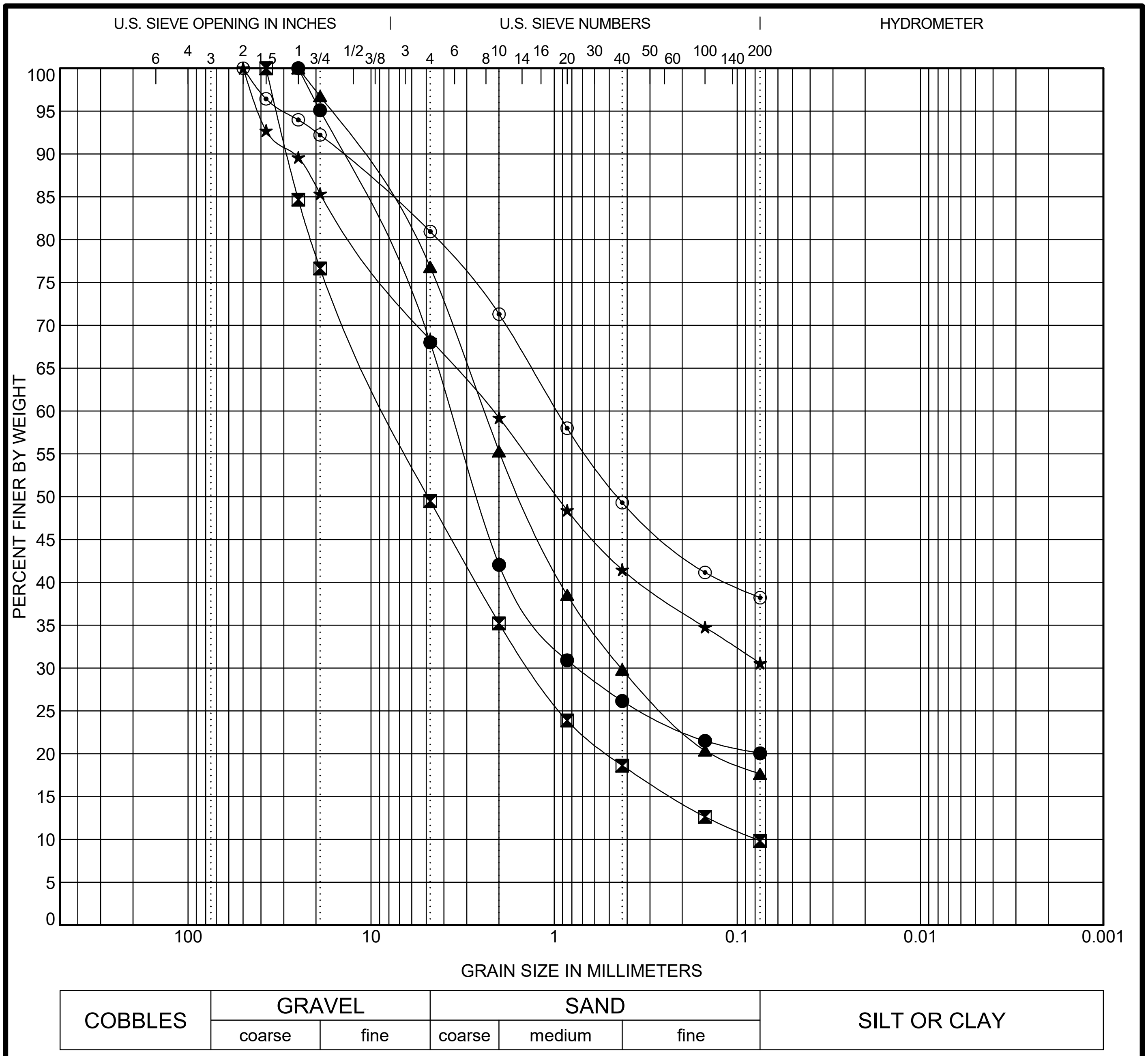
GRAIN SIZE DISTRIBUTION - ASTM C117 & C136

ALA MOANA BOULEVARD
ELEVATED PEDESTRIAN WALKWAY
FEDERAL AID PROJECT NO. BLD-092-1(029)
HONOLULU, OAHU, HAWAII

Plate
C - 3



	Sample	Depth (ft)	Description	LL	PL	PI	Cc	Cu
●	B-5	65.5-67.0	Whitish tan silty sand (SM) with some gravel					
⊠	B-6	5.0-6.5	Gray sandy gravel (GW-GM) with a little silt				2.9	103.6
▲	B-6	50.0-51.5	Whitish tan silty sand (SM) with some gravel					
★	BULK-1	1.0-3.0	Grayish brown silty sand (SM) with some gravel					
⊙	BULK-3	0.0-3.0	Reddish brown clayey sand (SC) with some gravel					

	Sample	Depth (ft)	D100 (mm)	D60 (mm)	D30 (mm)	D10 (mm)	%Gravel	%Sand	%Fine
●	B-5	65.5-67.0	25	3.639	0.746		32.0	48.0	20.0
⊠	B-6	5.0-6.5	37.5	8.131	1.35	0.078	50.5	39.7	9.8
▲	B-6	50.0-51.5	25	2.418	0.431		23.2	59.2	17.7
★	BULK-1	1.0-3.0	50	2.159			31.7	37.7	30.6
⊙	BULK-3	0.0-3.0	50	0.967			19.1	42.7	38.2

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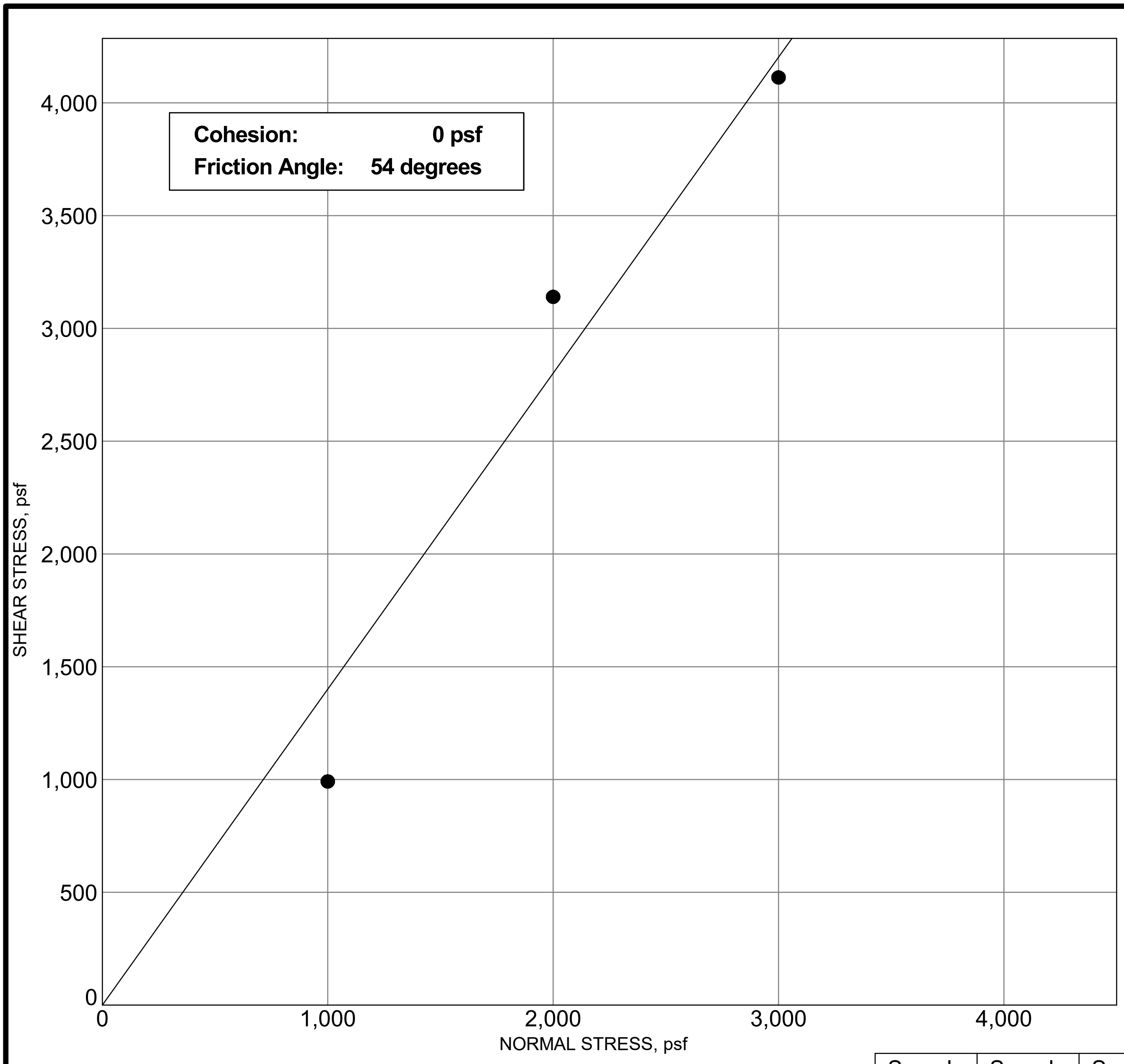
GRAIN SIZE DISTRIBUTION - ASTM C117 & C136

ALA MOANA BOULEVARD
ELEVATED PEDESTRIAN WALKWAY
FEDERAL AID PROJECT NO. BLD-092-1(029)
HONOLULU, OAHU, HAWAII

Plate
C - 4



		Sample #1	Sample #2	Sample #3
INITIAL	Moisture Content, %	16.5	14.8	16.9
	Dry Density, pcf	86.4	86.5	87.5
	Height, inches	1.00	1.00	1.00
FINAL	Moisture Content, %	24.0	22.2	22.1
	Dry Density, pcf	88.0	86.2	87.4
	Height, inches	0.981	1.004	1.001
	Diameter, inches	2.42	2.42	2.42
	Deformation Rate, inch/minute	0.0025	0.0023	0.0023
	Normal Stress, psf	**1000**	**2000**	**3000**
	Peak Shear Stress, psf	**991**	**3140**	**4112**
	Shear Displacement, inches	0.43	0.40	0.39

Sample: B-2
Depth: 50.5 - 52.0 feet
Description: Tan gravelly sand with traces of silt

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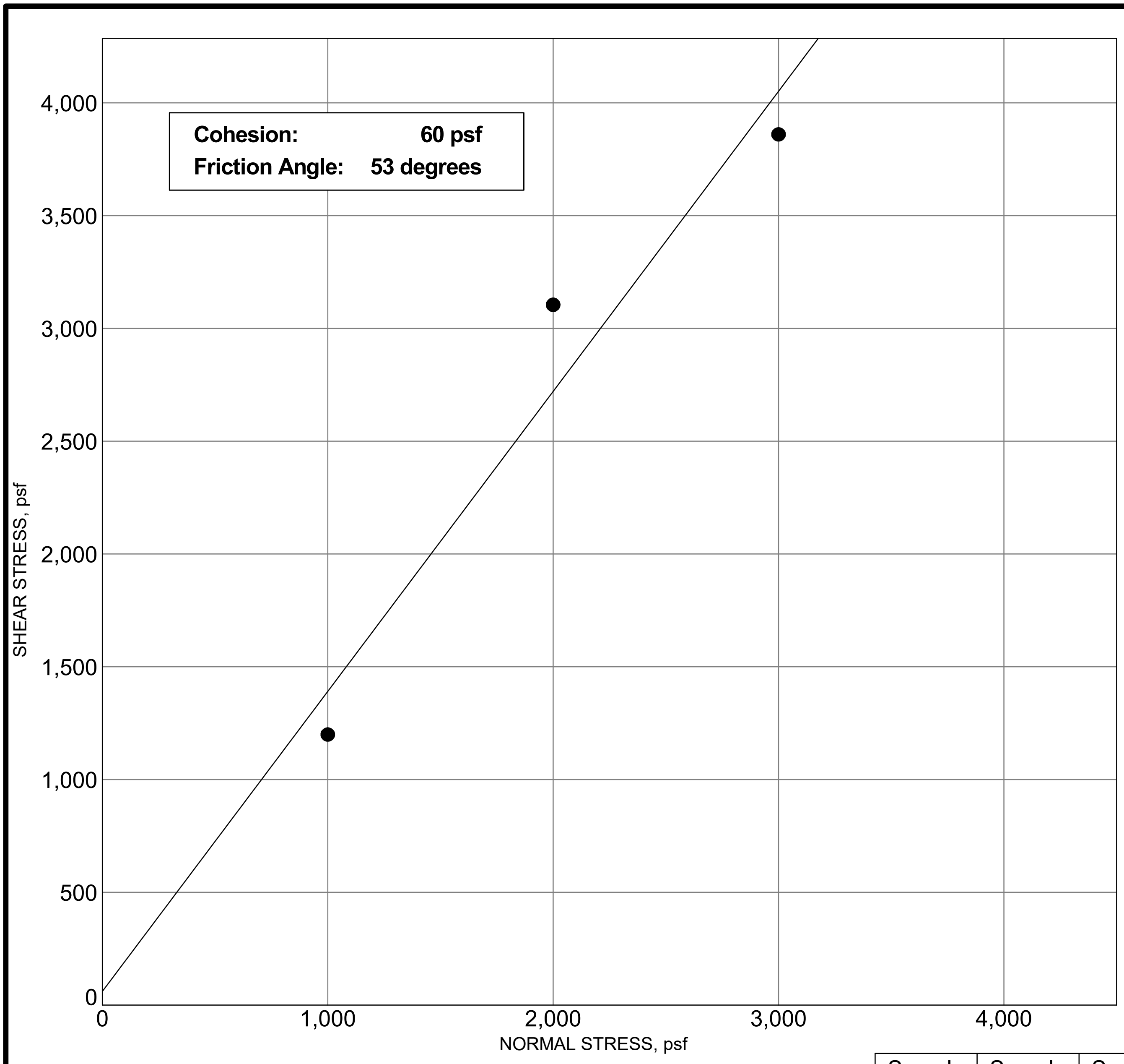
DIRECT SHEAR TEST - ASTM D3080

ALA MOANA BOULEVARD
ELEVATED PEDESTRIAN WALKWAY
FEDERAL AID PROJECT NO. BLD-092-1(029)
HONOLULU, OAHU, HAWAII

Plate
C - 5



Sample: B-4
Depth: 65.5 - 67.0 feet
Description: Tannish white silty sand (SM) with some gravel

		Sample #1	Sample #2	Sample #3
INITIAL	Moisture Content, %	19.3	17.8	24.4
	Dry Density, pcf	104.4	106.3	104.9
	Height, inches	1.00	1.00	1.00
FINAL	Moisture Content, %	21.2	18.6	19.3
	Dry Density, pcf	102.0	103.3	103.3
	Height, inches	1.023	1.029	1.016
	Diameter, inches	2.42	2.42	2.42
	Deformation Rate, inch/minute	0.0024	0.0022	0.0022
	Normal Stress, psf	**1000**	**2000**	**3000**
	Peak Shear Stress, psf	**1199**	**3104**	**3860**
	Shear Displacement, inches	0.43	0.40	0.39

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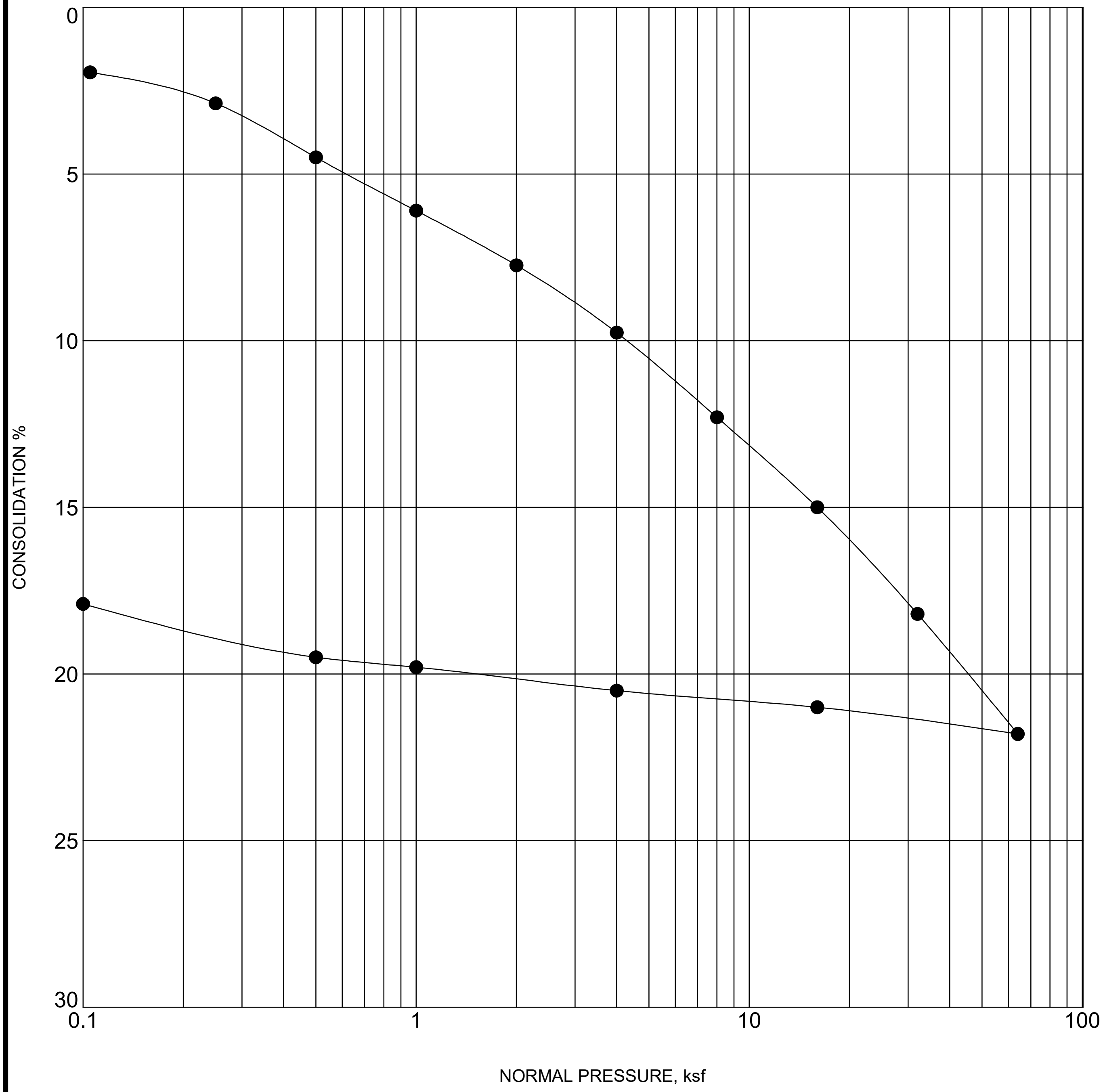
DIRECT SHEAR TEST - ASTM D3080

ALA MOANA BOULEVARD
ELEVATED PEDESTRIAN WALKWAY
FEDERAL AID PROJECT NO. BLD-092-1(029)
HONOLULU, OAHU, HAWAII

Plate
C - 6



Sample: B-1
Depth: 5.0 - 6.0 feet
Description: Light gray silty sand with some gravel

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

	Initial	Final
Water Content, %	25.8	20.8
Dry Density, pcf:	95.9	116.8
Void Ratio	0.995	0.638
Degree of Saturation, %	79.5	100.0
Sample Height, inches	1.0000	0.8200

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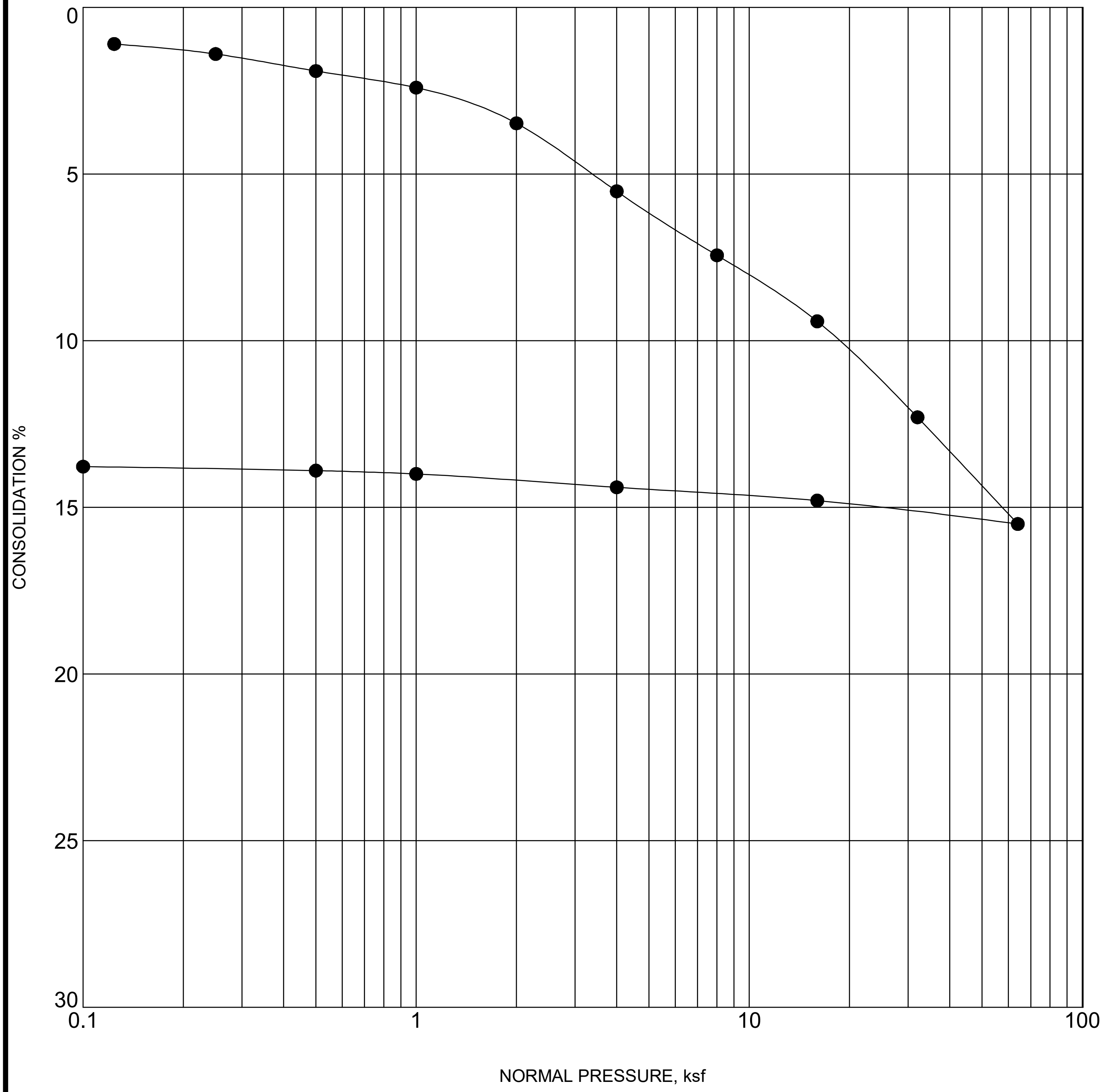
CONSOLIDATION TEST - ASTM D2435

ALA MOANA BOULEVARD
ELEVATED PEDESTRIAN WALKWAY
FEDERAL AID PROJECT NO. BLD-092-1(029)
HONOLULU, OAHU, HAWAII

Plate
C - 7

G_CONSOL 8115-00.GPJ GEOLABS.GDT 3/8/21



Sample: B-5
Depth: 5.0 - 6.5 feet
Description: Gray silty sand with some gravel

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

	Initial	Final
Water Content, %	20.2	16.7
Dry Density, pcf:	110.3	128.1
Void Ratio	0.767	0.522
Degree of Saturation, %	82.4	100.0
Sample Height, inches	1.0000	0.8600

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CONSOLIDATION TEST - ASTM D2435

ALA MOANA BOULEVARD
ELEVATED PEDESTRIAN WALKWAY
FEDERAL AID PROJECT NO. BLD-092-1(029)
HONOLULU, OAHU, HAWAII

Plate
C - 8

Location	Depth	Length	Diameter	Length/ Diameter Ratio	Density	Load	Compressive Strength
	(feet)	(inches)	(inches)		(pcf)	(lbs)	(psi)
B-1	62.5 - 66	6.542	3.195	2.05	95.4	2,000	250
B-1	112 - 116	6.770	3.290	2.06	126.7	14,770	1,740
B-2	32 - 35.5	6.600	3.300	2.00	98.7	4,720	550
B-3	36.08 - 40.5	6.692	3.231	2.07	83.6	5,320	650
B-3	72 - 75.5	6.608	3.248	2.03	113.4	9,150	1,100

ASTM D7012 (METHOD C)

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

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UNIAXIAL COMPRESSIVE STRENGTH TEST

ALA MOANA BOULEVARD
ELEVATED PEDESTRIAN WALKWAY
FEDERAL AID PROJECT NO. BLD-092-1(029)
HONOLULU, OAHU, HAWAII

Plate
C - 9

Location	Depth	Weight of Concrete	Length	Diameter	Density	Load	Length/ Diameter Ratio	Compressive Strength	Compressive Strength
	(feet)	(g)	(inches)	(inches)	(pcf)	(lb)		(psi)	(ksf)
B-2	6.5 - 11	1899.0	6.573	3.328	127	25190	1.98	2,900	420

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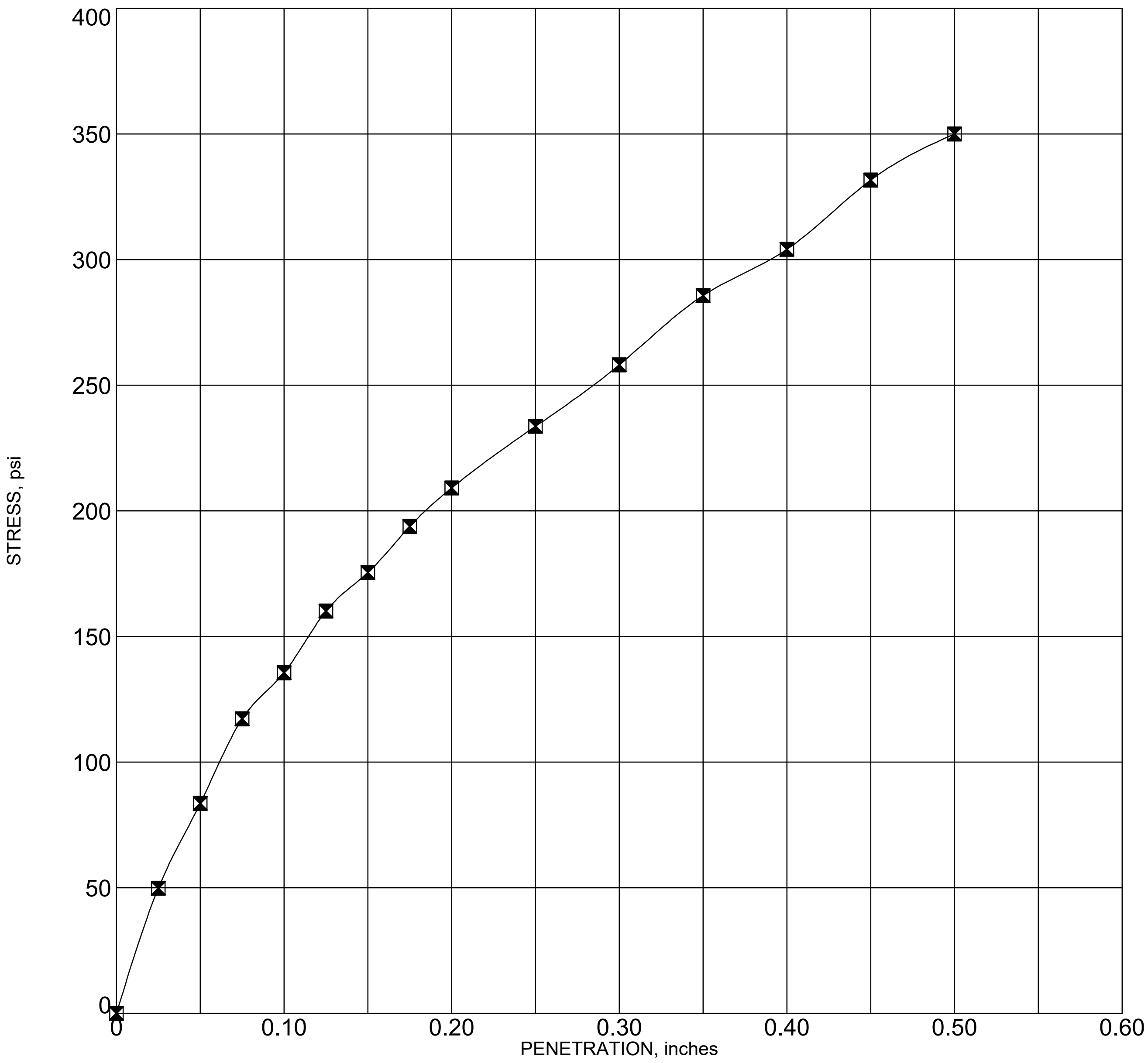
UNCONFINED COMPRESSION TEST - ASTM C39

ALA MOANA BOULEVARD
ELEVATED PEDESTRIAN WALKWAY
FEDERAL AID PROJECT NO. BLD-092-1(029)
HONOLULU, OAHU, HAWAII

Plate
C - 10



Corr. CBR @ 0.1"	13.6
Corr. CBR @ 0.2"	13.9
Swell (%)	0.20

Sample: BULK-1
Depth: 1.0 - 3.0 feet
Description: Grayish brown silty sand (SM) with some gravel

Molding Dry Density (pcf)	98.5	Hammer Wt. (lbs)	10
Molding Moisture (%)	24.6	Hammer Drop (inches)	18
Days Soaked	5	No. of Blows	56
Aggregate	3/4 inch minus	No. of Layers	5

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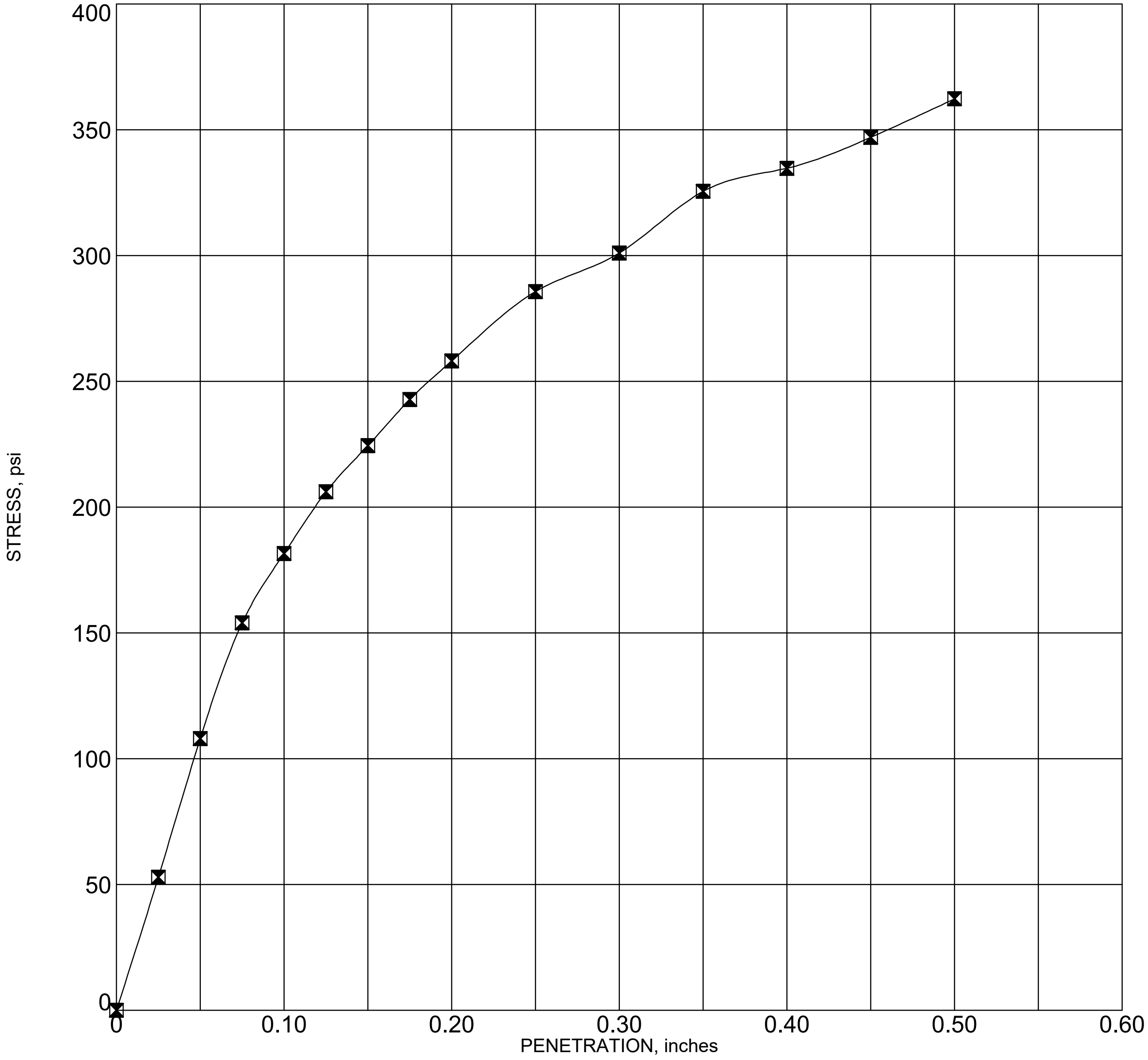
CALIFORNIA BEARING RATIO - ASTM D1883

ALA MOANA BOULEVARD
ELEVATED PEDESTRIAN WALKWAY
FEDERAL AID PROJECT NO. BLD-092-1(029)
HONOLULU, OAHU, HAWAII

Plate
C - 11

G CBR 8115-00.GPJ GEOLABS.GDT 3/8/21



Corr. CBR @ 0.1"	18.2
Corr. CBR @ 0.2"	17.2
Swell (%)	0.61

Sample: BULK-2
Depth: 0.0 - 3.0 feet
Description: Tannish brown silty sand with a little gravel

Molding Dry Density (pcf)	97.1	Hammer Wt. (lbs)	10
Molding Moisture (%)	26.4	Hammer Drop (inches)	18
Days Soaked	5	No. of Blows	56
Aggregate	3/4 inch minus	No. of Layers	5

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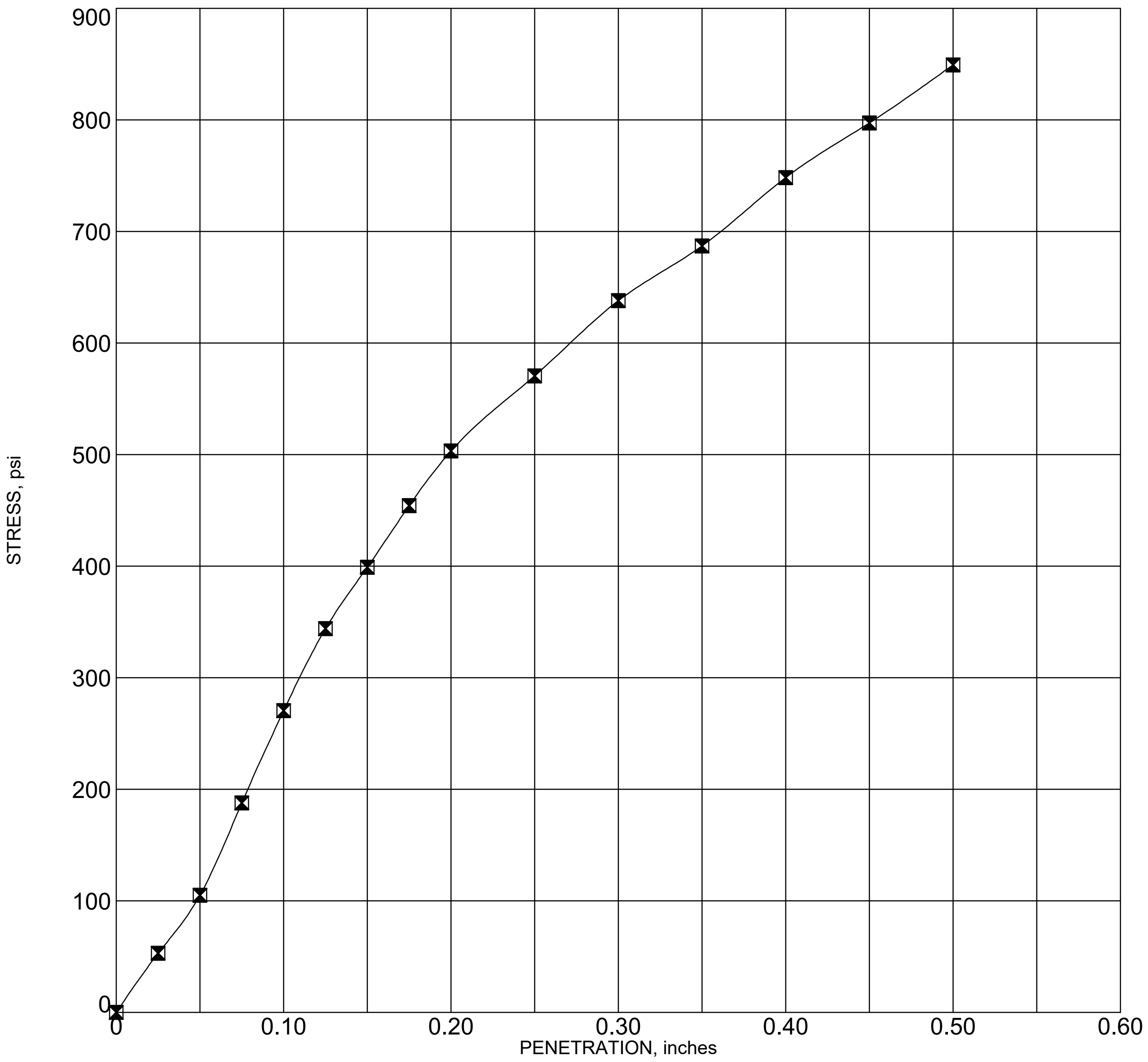
CALIFORNIA BEARING RATIO - ASTM D1883

ALA MOANA BOULEVARD
ELEVATED PEDESTRIAN WALKWAY
FEDERAL AID PROJECT NO. BLD-092-1(029)
HONOLULU, OAHU, HAWAII

Plate
C - 12



Corr. CBR @ 0.1"	32.4
Corr. CBR @ 0.2"	35.2
Swell (%)	1.99

Sample: BULK-3
Depth: 0.0 - 3.0 feet
Description: Reddish brown clayey sand (SC) with some gravel

Molding Dry Density (pcf)	105.4	Hammer Wt. (lbs)	10
Molding Moisture (%)	14.9	Hammer Drop (inches)	18
Days Soaked	5	No. of Blows	56
Aggregate	3/4 inch minus	No. of Layers	5

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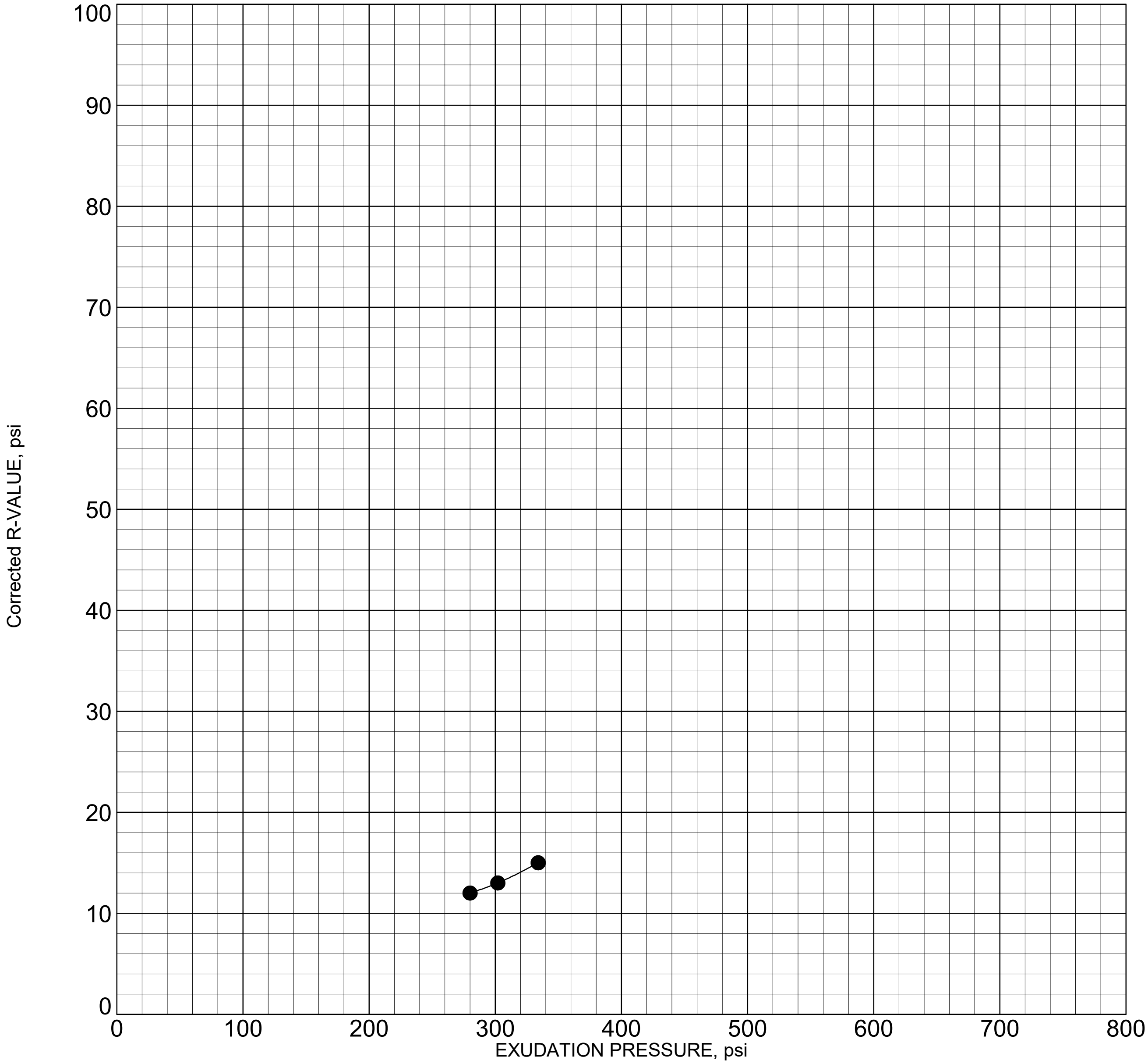
CALIFORNIA BEARING RATIO - ASTM D1883

ALA MOANA BOULEVARD
ELEVATED PEDESTRIAN WALKWAY
FEDERAL AID PROJECT NO. BLD-092-1(029)
HONOLULU, OAHU, HAWAII

Plate
C - 13

G_CBR 8115-00.GPJ GEOLABS.GDT 3/8/21



Sample: BULK-2
Depth: 0.0 - 3.0 feet
Description: Tannish brown silty sand with a little gravel

R-Value at 300 psi Exudation Pressure:	**13**

R-Value Test Performed by Ninyo & Moore

No.	Compaction Pressure	Density	Moisture Content	Horizontal Pressure @160 psi	Sample Height	Exudation Pressure	R-Value	Corrected R-Value
	(psi)	(pcf)	(%)	(psi)	(in)	(psi)		
1	50	92.2	31.8	N/A	2.55	334	15	15
2	50	91.6	32.3	N/A	2.56	302	13	13
3	50	90.9	32.8	N/A	2.54	280	12	12

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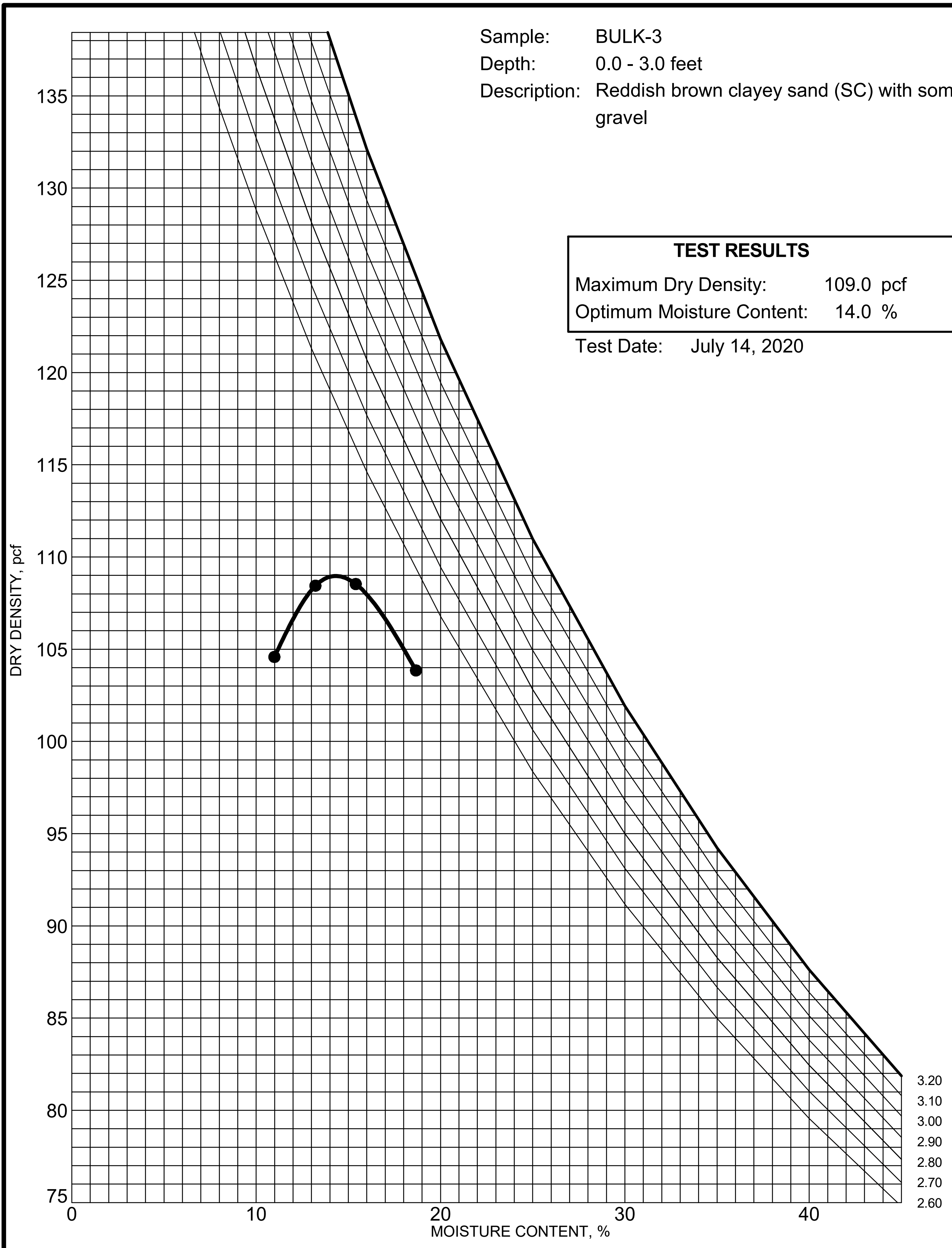
R-VALUE AND EXPANSION PRESSURE - ASTM D2844

ALA MOANA BOULEVARD
ELEVATED PEDESTRIAN WALKWAY
FEDERAL AID PROJECT NO. BLD-092-1(029)
HONOLULU, OAHU, HAWAII

Plate
C - 14



Sample: BULK-3
Depth: 0.0 - 3.0 feet
Description: Reddish brown clayey sand (SC) with some gravel

TEST RESULTS

Maximum Dry Density:	109.0 pcf
Optimum Moisture Content:	14.0 %

Test Date: July 14, 2020

G_COMPACTION 8115-00.GPJ GEOLABS.GDT 3/8/21

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MOISTURE-DENSITY RELATIONSHIP - ASTM D1557 B

ALA MOANA BOULEVARD
ELEVATED PEDESTRIAN WALKWAY
FEDERAL AID PROJECT NO. BLD-092-1(029)
HONOLULU, OAHU, HAWAII

Plate
C - 15

Location	Depth (feet)	pH Value	Minimum Resistivity (ohm-cm)	Chloride Content (mg/kg)	Sulfate Content (mg/kg)
B-4	1.0 - 2.5	8.72*	5300*	15	14
B-5	1.0 - 2.5	7.98*	2000*	130	48

TEST METHODS (by Eurofins TestAmerica Laboratories, Inc.)

pH Value	Method 9045C
Minimum Resistivity	SM 2510B
Chloride Content	EPA 300.0
Sulfate Content	EPA 300.0

TEST METHODS (by Geolabs, Inc.)*

pH Value	ASTM G51
Minimum Resistivity	ASTM G57
Chloride Content	N/A
Sulfate Content	N/A

ND: Not Detected Within Reporting Limits

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SUMMARY OF CORROSIVITY TESTS

ALA MOANA BOULEVARD
ELEVATED PEDESTRIAN WALKWAY
FEDERAL AID PROJECT NO. BLD-092-1(029)
HONOLULU, OAHU, HAWAII

Plate
C - 16

Boring Number	Depth	Moisture Content*	Ash Content**	Organic Matter
	(feet)	(%)	(%)	(%)
B-4	5 - 6.5	17.4	98.4	1.6
B-4	26 - 27.5	34.8	98.7	1.3
B-6	5 - 6.5	15.9	98.4	1.6

* DRY AT 105°C
** DRY AT 440°C

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ASTM D2974 - MOISTURE, ASH & ORGANIC MATTER

ALA MOANA BOULEVARD
ELEVATED PEDESTRIAN WALKWAY
FEDERAL AID PROJECT NO. BLD-092-1(029)
HONOLULU, OAHU, HAWAII

APPENDIX D

ALA MOANA BOULEVARD
ELEVATED PEDESTRIAN WALKWAY
FEDERAL AID PROJECT NO. BLD-092-1(029)
HONOULU, OAHU, HAWAII

B-1 32.5' TO 116.0'

ALA MOANA BOULEVARD
ELEVATED PEDESTRIAN WALKWAY
FEDERAL AID PROJECT NO. BLD-092-1(029)
HONOULU, OAHU, HAWAII

B-2 7.0' TO 120.5'

ALA MOANA BOULEVARD
ELEVATED PEDESTRIAN WALKWAY
FEDERAL AID PROJECT NO. BLD-092-1(029)
HONOULU, OAHU, HAWAII

B-2A 9.0' TO 40.5'

9.0' 27.0'

15.5' 40.5'

B-2B 6.5' TO 11.5'

6.5'

11.5'

ALA MOANA BOULEVARD
ELEVATED PEDESTRIAN WALKWAY
FEDERAL AID PROJECT NO. BLD-092-1(029)
HONOULU, OAHU, HAWAII

B-3 32.0' TO 75.5'

32.0' 36.08' 47.0' 67.0'

35.5' 40.5' 65.5' 75.5'

ALA MOANA BOULEVARD
ELEVATED PEDESTRIAN WALKWAY
FEDERAL AID PROJECT NO. BLD-092-1(029)
HONOULU, OAHU, HAWAII

B-3 75.92' TO 111.08'

75.92' 87.0' 97.0' 107.0'

80.5' 95.5' 105.5' 111.08'

ALA MOANA BOULEVARD
ELEVATED PEDESTRIAN WALKWAY
FEDERAL AID PROJECT NO. BLD-092-1(029)
HONOULU, OAHU, HAWAII

B-4 27.0’ TO 35.5’

27.0’

35.5’

B-5 32.0’ TO 40.5’

32.0’

40.5’