GEOTECHNICAL ENGINEERING EXPLORATION

FREEWAY MANAGEMENT SYSTEM

PHASE 3, UNITS 1 AND 2, IM-0300(152)

INTERSTATE ROUTES H-1 AND H-2 FREEWAYS

DISTRICTS OF HONOLULU AND EWA, OAHU, HAWAII

W.O. 6891-30(A) FEBRUARY 5, 2021

Prepared for

AUSTIN, TSUTSUMI & ASSOCIATES, INC.

GEOLABS, INC.

Geotechnical Engineering and Drilling Services

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Prepared for

AUSTIN, TSUTSUMI & ASSOCIATES, INC.

GERALD Y. SEKI
LICENSED PROFESSIONAL ENGINEER
No. 5635-C
HAWAII, U.S.A.

THIS WORK WAS PREPARED BY ME OR UNDER MY SUPERVISION.

SIGNATURE — 4-30-22
EXPIRATION DATE OF THE LICENSE

GEOLABS, INC.
Geotechnical Engineering and Drilling Services
94-0429 Koaki Street, Suite 200 • Waipahu, HI 96797

Hawaii • California

GEOLABS, INC.
Geotechnical Engineering and Drilling Services

February 5, 2021
W.O. 6891-30(A)

Mr. Neal Kasamoto
Austin, Tsutsumi & Associates, Inc.
501 Sumner Street, Suite 521
Honolulu, HI 96817

Dear **Mr. Kasamoto:**

Geolabs, Inc. is pleased to submit our report entitled "Geotechnical Engineering Exploration, Freeway Management System, Phase 3, Units 1 and 2, IM-0300(152), Interstate Routes H-1 and H-2 Freeways" prepared for the design of the proposed project.

Our work for the project was performed in general accordance with the scope of services outlined in our fee proposal dated May 17, 2018.

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

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

Very truly yours,

GEOLABS, INC.

Gerald Y. Seki, P.E.
Vice President

GS:JS:AT:cj

94-429 Koaki Street, Suite 200 • Waipahu, Hawaii 96797
Telephone: (808) 841-5064 • E-mail: hawaii@geolabs.net

Hawaii • California

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GEOTECHNICAL ENGINEERING EXPLORATION
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PHASE 3, UNITS 1 AND 2, IM-0300(152)
INTERSTATE ROUTES H-1 AND H-2 FREEWAYS
DISTRICTS OF HONOLULU AND EWA, OAHU, HAWAII
W.O. 6891-30(A) FEBRUARY 5, 2021

SUMMARY OF FINDINGS AND RECOMMENDATIONS

Based on our field exploration and research of available geologic information, the geologic units that comprise the sites include lava flows of the Koolau Volcanic series at the H-2 North, H-2 South and Waikele CCTV sites. The H-1/Kamehameha CCTV site encountered about a 14-foot thick layer of fill material overlying lava flows of the Koolau Volcanic series. The Ko Olina CCTV site encountered colluvium. We did not encounter groundwater in the borings at the time of our field exploration.

In order to develop the required bearing and lateral load resistances for the new CCTV structures, we recommend supporting them on 42-inch diameter drilled shaft foundations. Based on the anticipated structural loads and the subsoil conditions encountered at each site, the length of the drilled shafts supporting the CCTV structures would range from 6 to 13 feet.

The performance of the drilled shafts depends significantly upon the contractor's method of construction, construction procedures, and workmanship. Therefore, the contractor should follow the recommendations and general guidelines presented in this report during the drilled shaft foundation construction.

The drilled shaft subcontractor will need to have the appropriate equipment and tools to drill through the cobbles, boulders, and basalt rock encountered. Basalt rock was encountered at the Waikele CCTV and H-2 North CCTV sites. Residual and saprolitic soils were encountered at the H-1/Kamehameha CCTV and H-2 South CCTV sites. It should be noted that there is a high potential for encountering basalt rock in the form of unweathered cobbles and boulders in the residual and saprolitic soils. In addition, cobbles and boulders may be encountered in the fill. The text of this report should be referred to for detailed discussion and recommendations.

END OF SUMMARY OF FINDINGS AND RECOMMENDATIONS

SECTION 1. GENERAL

1.1 Introduction

This report presents the results of our geotechnical engineering exploration performed for the "Freeway Management System, Phase 3, Units 1 and 2, IM-0300(152), Interstate Routes H-1 and H-2 Freeways project on the Island of Oahu, Hawaii. The project locations and general vicinities are shown on the Project Location Map, Plate 1.

This report summarizes the findings and presents our geotechnical engineering recommendations derived from our field exploration, laboratory testing, and engineering analyses. These recommendations are only intended for the design of closed circuit television (CCTV) foundations only. The findings and recommendations presented herein are subject to the limitations noted at the end of this report.

1.2 Project Considerations

The project consists of the installation of closed circuit television (CCTV) camera structures generally along Interstate Route H-1 and H-2 Freeways and Farrington Highway in the Districts of Honolulu and Ewa on the Island of Oahu, Hawaii. Five CCTV cameras will be mounted on independent poles 50 feet high. The CCTV locations are presented in the following table:

CCTV Identification	Location	Interstate Route
H-2 North CCTV	South of Waipio Interchange	H-2 Freeway
H-2 South CCTV	South of Waipio Interchange	H-2 Freeway
H-1/Kamehameha CCTV	Waiawa Interchange	H-1 Freeway
Waikele CCTV	Near Waikele Shopping Center	H-1 Freeway
Ko Olina CCTV	Ko Olina Interchange	Farrington Highway

The following structural load information acting at the base of the poles were provided by the project structural engineer for our analysis of the new CCTV foundation design.

CCTV STRUCTURAL LOAD INFORMATION					
Structure	**Loading Condition**	**Vertical Load** (kips)	**Lateral Loads** (kips)	**Overturning Moments** (kip-feet)	**Torsion** (kip-feet)
CCTV	Extreme Event	$F_y = 5$ (uplift)	$F_x = 5$ $F_z = 5$	$M_x = 125$ $M_z = 50$	$M_y = 2$

1.3 Purpose and Scope

The purpose of our exploration was to obtain an overview of the subsurface conditions to develop an idealized soil/rock data set to formulate geotechnical engineering recommendations for the design of the CCTV structure foundations. The scope of work for this exploration included the following tasks and work efforts:

1. Research and review of available in-house geologic and soils information at the CCTV sites under this project.
2. Application of necessary excavation permits from the State of Hawaii – Department of Transportation, Highways Division.
3. Coordination of the boring stakeout and utility toning and clearance with various utility companies.
4. Provisions of police officers and safety devices for traffic control and lane closures at the boring locations during our field exploration program.
5. Mobilization and demobilization of a truck-mounted drill rig and two operators to the project sites and back.
6. Drilling and sampling of five borings extending to depths of about 15.5 to 21.5 feet below the existing ground surface.
7. Coordination of the field exploration and logging of the borings by our geologists.
8. Laboratory testing of selected soil and rock samples obtained during the field exploration as an aid in classifying the materials and evaluating their engineering properties.
9. Engineering analyses of the field and laboratory data to formulate geotechnical engineering recommendations for the CCTV foundations.
10. Preparation of this report summarizing our work on the project and presenting our findings and geotechnical engineering recommendations.

11. Coordination of our overall work on the project by our senior engineer.

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

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

Detailed descriptions of our field exploration and the Logs of Borings are provided in Appendix A. Results of the laboratory tests performed on selected soil and rock samples are presented in Appendix B. Photographs of core samples retrieved from our field exploration are presented in Appendix C.

END OF GENERAL

SECTION 2. SITE CHARACTERIZATION

2.1 Regional Geology

The Island of Oahu was built by the extrusion of basaltic lavas from the Waianae and Koolau shield volcanoes. The older Waianae Volcano is estimated to be middle to late Pliocene in age and forms the bulk of the western third of the island. The younger Koolau Volcano is estimated to be late Pliocene to early Pleistocene (Ice Age) in age and forms the majority of the eastern two-thirds of the island. The Waianae Volcano became extinct while the Koolau Volcano was still active, and its eastern flank was partially buried below Koolau lavas banking against its eastern flank. These banked or ponded lavas formed a broad plateau referred to as the Schofield Plateau.

The Schofield Plateau was formed when lavas from the Koolau Volcano ponded against the already eroded slopes of the Waianae Volcano in the late Pleistocene Epoch. The dips of the lava beds are generally near horizontal (between 3 to 5 degrees from horizontal). The lava flows on the plateau have undergone in-situ weathering extending to depths of 50 to 100 feet and are characterized by the red colors of the soil. In general, the H-2 North, H-2 South, H-1/Kamehameha, and Waikele CCTV sites are located on the southerly side of the Schofield Plateau.

Physical and chemical weathering, followed by erosion of this plateau, generated sediments which were transported to the coast. The Ko Olina CCTV site is generally located in areas of colluvial deposits overlying basalt rock formation at greater depths.

2.2 Existing Site Conditions

The project sites are located along Interstate Route H-1 Freeway from the Paiwa Interchange to the Waiawa Interchange, on the Route H-2 Freeway between the Waipio Interchange and the Waiawa Interchange, and Farrington Highway near the Ko Olina exit in the Districts of Honolulu and Ewa on the Island of Oahu, Hawaii. The project locations and general vicinities are shown on the Project Location Map, Plate 1. The approximate locations of each site are shown on the Site Plans, Plates 2.1 through 2.5. The following provides a brief description of the existing conditions at each CCTV site.

2.2.1 Waikele CCTV Site

The CCTV site is located next to the far-right shoulder lane before Exit No. 7 from the H-1 Freeway Westbound. The site slopes gently towards the Freeway with an existing ground surface elevation of about +168 feet Mean Sea Level (MSL). The site is covered with grass.

2.2.2 H-1/Kamehameha CCTV Site

The site is located in the median between the H-1 Freeway eastbound Exit 8A ramp and the H-99 Kamehameha Highway. The site is relatively flat; however, it is adjacent to the embankment slope of the H-1 eastbound Exit 8A ramp. The site is covered with grass.

2.2.3 H-2 North CCTV Site

The site is located in the grassy median between the H-2 Freeway Inbound and Outbound. The site is relatively flat, with an existing ground surface elevation of about +296 feet MSL.

2.2.4 H-2 South CCTV Site

The site is located on the far-right shoulder lane of the H-2 Inbound before Exit 1A. The site is relatively flat and covered with asphalt. The site has a ground surface elevation of about +195 feet MSL.

2.2.5 Ko Olina CCTV Site

The site is located between Farrington Highway Inbound and Farrington Highway merge to Ali'inui Drive. The site is relatively flat, with an existing ground surface elevation of about +62 feet MSL. The site is covered with grass.

2.3 Subsurface Conditions

We explored the subsurface conditions at the CCTV sites by drilling and sampling one boring at each location. The borings extended to depths of about 15.5 to 21.5 feet below the existing ground surface. The approximate boring locations are shown on the Site Plans, Plates 2.1 through 2.5.

The subsurface conditions at each CCTV location are presented in the following subsections.

2.3.1 Waikele CCTV Site

Boring No. 1, drilled near the Waikele CCTV site, generally encountered a 4-foot thick layer of residual soil consisting of very stiff silty clay soils underlain by boulders and saprolite extending to a depth of 10 feet below the existing ground surface. The saprolite consisted of loose silty sand and was underlain by hard to very hard basalt formation extending to the maximum depth explored about 20 feet below the existing ground surface.

2.3.2 H-1/Kamehameha CCTV Site

Boring No. 2, drilled near the H-1/Kamehameha CCTV site, generally encountered fill consisting of very stiff sandy silt and loose clayey sand extending to 14 feet below the existing ground surface. The fill layer was underlain by residual soil consisting of soft to medium stiff clayey silt extending to the maximum depth explored about 21.5 feet below the existing ground surface.

2.3.3 H-2 North CCTV Site

Boring No. 3, drilled near the H-2 North CCTV site, generally encountered fill consisting of very stiff silty clay extending to approximately 2.5 feet below the existing ground surface. The fill was underlain by hard to very hard basalt formation extending to the maximum depth explored about 15.5 feet below the existing ground surface.

2.3.4 H-2 South CCTV Site

Boring No. 4, drilled near the H-2 South CCTV site, generally encountered 3 inches of asphaltic concrete underlain by 7 feet of fill. The fill consisted of sandy gravel and very stiff clayey silt. The fill was underlain by residual soils consisting of medium dense silty gravel and very dense cobbly boulders extending to the maximum depth explored about 20.5 feet below the existing ground surface.

2.3.5 Ko Olina CCTV Site

Boring No. 5, drilled near the Ko Olina CCTV site, generally encountered fill extending to a depth of 12 feet below ground surface. The fill was underlain by colluvium extending to a maximum depth explored of 21.5 feet below the existing ground surface. The fill generally consisted of medium dense silty sand. The colluvium generally consisted of dense sandy gravel and silty sand.

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

Detailed descriptions of the materials encountered from our field exploration are presented on the Logs of Borings in Appendix A. Results of the laboratory tests performed on selected soil and rock samples are presented in Appendix B. Photographs of core samples retrieved from our field exploration are presented in Appendix C.

END OF SITE CHARACTERIZATION

SECTION 3. DISCUSSION AND RECOMMENDATIONS

In order to develop the required bearing and lateral load resistances for the new CCTV structures, we recommend supporting the CCTV structures on drilled shaft foundations. The CCTV structures should be supported on 42-inch diameter drilled shaft foundations. Based on the anticipated structural loads and the subsoil conditions encountered at each site, the length of the 42-inch diameter drilled shafts supporting the CCTV structures would range from about 6 to 13 feet.

The drilled shaft subcontractor will need to have the appropriate equipment and tools to drill through the cobbles, boulders, and basalt rock where encountered. Basalt rock was encountered at the Waikele CCTV and H-2 North CCTV sites. Residual and saprolitic soils were encountered at the H-1/Kamehameha CCTV and H-2 South CCTV site. It should be noted that there is a high potential for encountering basalt rock in the form of unweathered boulders and cobbles in the residual and saprolitic soils.

The performance of the drilled shafts depends significantly upon the contractor's method of construction, construction procedures, and workmanship. Therefore, the contractor should follow the recommendations and general guidelines presented in the “Drilled Shaft Construction Considerations” subsection of this report. Detailed discussion of these items and our geotechnical engineering design recommendations are presented in the following sections.

3.1 Drilled Shaft Foundations

In order to develop the required bearing and lateral load resistances, the proposed new CCTV camera structures may be supported by a foundation system consisting of cast-in-place concrete drilled shafts. Our recommendations for the drilled shaft foundations at each CCTV site are presented in the following table:

CCTV DRILLED SHAFT FOUNDATION				
Site	**Unfactored Compressive Load Shaft Capacity** (kips)	**Compressive Load Service Limit State** (kips)	**Shaft Diameter** (inches)	**Shaft Length** (feet)
Waikele CCTV	170	55	42	12
H-1/Kamehameha CCTV	125	40	42	13
H-2 North CCTV	130	43	42	6
H-2 South CCTV	80	25	42	10
Ko Olina CCTV	115	38	42	12

The drilled shaft foundation lengths provided in the above tables are measured from the design finished grades (ground level) to the tip of the drilled shaft. The cast-in-place concrete drilled shafts would derive vertical support principally from skin friction between the shafts and the surrounding soils.

3.1.1 Uplift Load Resistance

In general, uplift loads may be resisted by a combination of the dead weight of the drilled shaft and by shear along the shaft surface and the adjacent soils. Considering that the drilled shafts are designed based on adhesion between the shaft and the surrounding soils, we recommend using the following uplift load capacities for the extreme event and strength limit states. For the strength limit state, a resistance factor of 0.35 has been applied to the extreme event limit state uplift load capacities. In addition, the resistance factor was reduced by 20 percent for the single non-redundant drilled shaft. The project structural engineer should check the structural capacity of the shaft member in tension.

UPLIFT LOAD CAPACITIES FOR CCTV DRILLED SHAFT FOUNDATION			
		Uplift Load Capacity (kips)	
Site	Shaft Diameter (feet)	Extreme Event Limit State	Strength Limit State
Waikele CCTV	42	180	50
H-1/Kamehameha CCTV	42	136	38
H-2 North CCTV	42	140	39
H-2 South CCTV	42	84	24
Ko Olina CCTV	42	120	35

3.1.2 Lateral Load Resistance

The lateral load resistance of drilled shafts is a function of the stiffness of the surrounding soil, 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. Based on the loading conditions and the subsurface soil conditions, the results of our analyses are summarized in the following tables:

FOUNDATION ANALYSES RESULTS FOR CCTV 42-INCH DIAMETER DRILLED SHAFTS					
Site	**Loading Condition**	**Maximum Horizontal Deflection** (inch)	**Maximum Induced Shear** (kips)	**Maximum Induced Moment** (kip-feet)	**Depth to Maximum Moment** (feet)
Waikele CCTV	Extreme Event	0.1	91	152	5.5
H-1 / Kamehameha CCTV	Extreme Event	0.2	26	153	3.8
H-2 North CCTV	Extreme Event	0.2	194	141	1.7
H-2 South CCTV	Extreme Event	0.1	22	141	1.8
Ko Olina CCTV	Extreme Event	0.2	28	151	3.5

3.1.3 Torsional Resistance

In general, the CCTV structures may be subjected to torsional moments resulting in torsion on the drilled shaft foundations. Torsion may be resisted by the side shear along the drilled shaft surface and adjacent soils. Based on our analyses, we believe that the recommended diameter of drilled shafts extending to the depths recommended should be capable of resisting the torsional moment without significant movement of the foundations. We wish to emphasize that the sides of the drilled holes should be relatively free of loose materials and that the concrete for the drilled shaft should have close contact with the in-situ soils after the drilled shaft has been constructed to provide the torsional resistance required.

3.1.4 Foundation Settlement

Settlement of the drilled shaft foundation will result from elastic compression of the shaft and subgrade response of the foundation embedded in the soils encountered at the site. We anticipate that the total settlements of the drilled shafts are 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.

3.1.5 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 surrounding soils. 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.

Basalt rock was encountered at the Waikele CCTV and H-2 North CCTV sites. Residual and saprolitic soils were encountered at the H-1/Kamehameha CCTV and H-2 South CCTV sites. There is a high potential for encountering basalt rock in the form of unweathered boulders and cobbles in the residual and saprolitic soils. In addition, cobbles and boulders may be encountered in the fill. The drilled shaft subcontractor will need to have the appropriate equipment and tools to drill through the basalt rock, boulders, and cobbles where encountered.

A Geolabs representative should be present to observe the drilling and installation of drilled shafts during construction. Although the drilled shafts are designed based primarily 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 as a “Special Inspection” item.

3.2 Design Review

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

3.3 Construction Observation

Geolabs should be retained to provide geotechnical engineering services during construction of the proposed project. The critical item of construction monitoring that requires "Special Inspection" includes a review of the drilled shaft construction submittals and observation of the drilled shaft construction.

A Geolabs representative should monitor other aspects of earthwork construction 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 exposed 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. Variations of conditions between and beyond the field borings may occur, and the nature and extent of these variations may not become evident until construction is underway. If variations then appear evident, it will be necessary to re-evaluate the recommendations presented herein.

The field boring locations are approximate, having been estimated by taping from reference points and visible features shown on the Site Plans provided by Austin, Tsutsumi & Associates, Inc. on May 29, 2020. Elevations of the borings were estimated from Google Earth Pro. The physical locations and elevations of the borings should be considered accurate only to the degree implied by the methods used.

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

This report has been prepared for the exclusive use of Austin, Tsutsumi & Associates, Inc. for specific application to the “Freeway Management System, Phase 3, Units 1 and 2, IM-0300(152), Interstate Routes H-1 and H-2 Freeways” project in accordance with generally accepted geotechnical engineering principles and practices. No warranty is expressed or implied.

This report has been prepared solely for the purpose of assisting the engineer in the design of the proposed project. Therefore, this report may not contain sufficient data, or the proper information, to serve as the basis for the preparation of construction cost estimates. A contractor wishing to bid on this project is urged to retain a competent geotechnical engineer to assist in the interpretation of this report and/or in the performance of additional site-specific exploration for bid estimating purposes.

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

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

END OF LIMITATIONS

CLOSURE

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

-ΩΩΩΩΩΩΩΩΩ-

Respectfully submitted,

GEOLABS, INC.

By ______________________

Gerald Y. Seki, P.E.
Vice President

GS:JS:AT:cj

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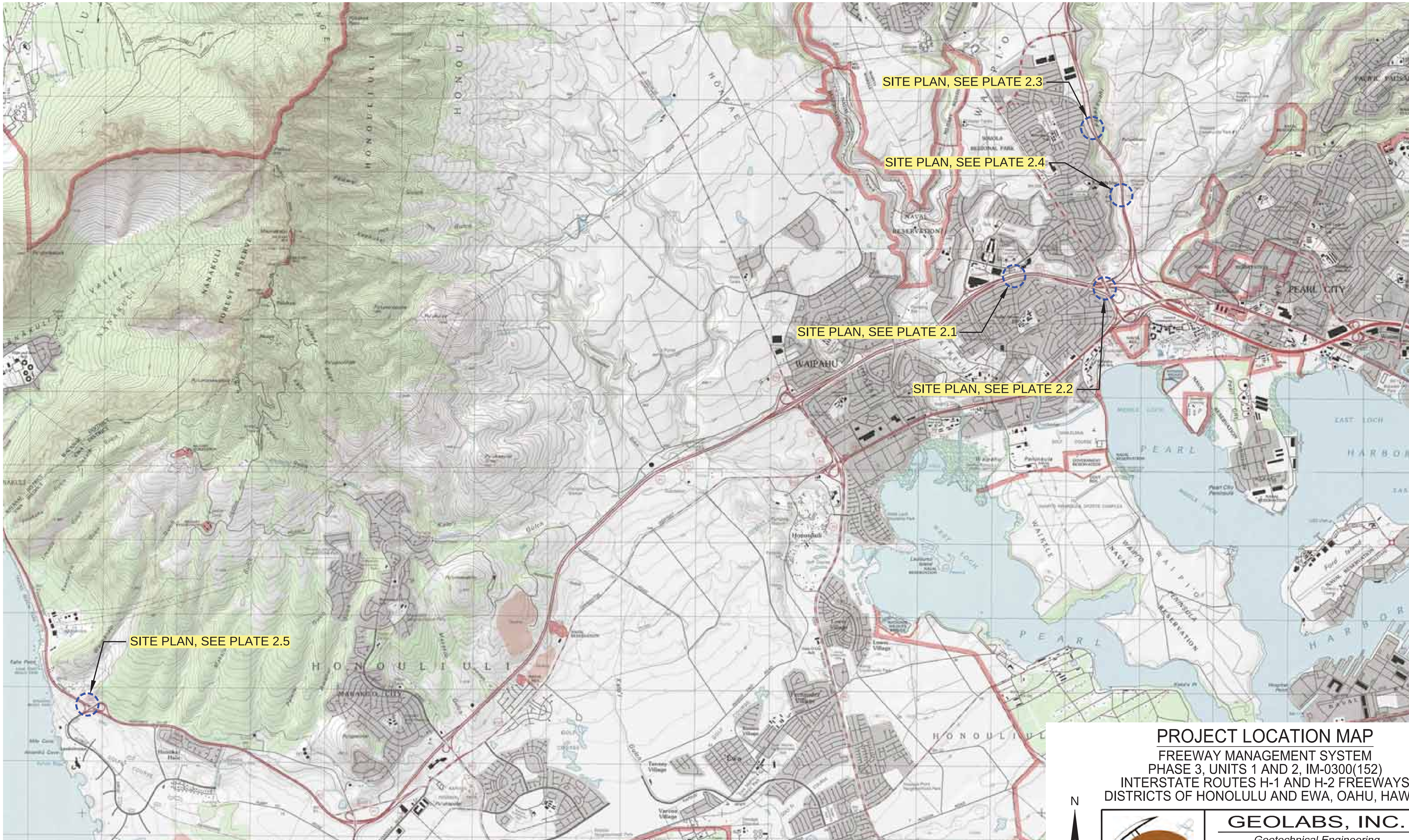
PLATES



0 2500 5000 10000 15000 FT.

GRAPHIC SCALE

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

PROJECT LOCATION MAP

FREEWAY MANAGEMENT SYSTEM
PHASE 3, UNITS 1 AND 2, IM-0300(152)
INTERSTATE ROUTES H-1 AND H-2 FREEWAYS
DISTRICTS OF HONOLULU AND EWA, OAHU, HAWAII

GEOLABS, INC.
Geotechnical Engineering

DATE	DRAWN BY	PLATE
SEPTEMBER 2020	ASP	
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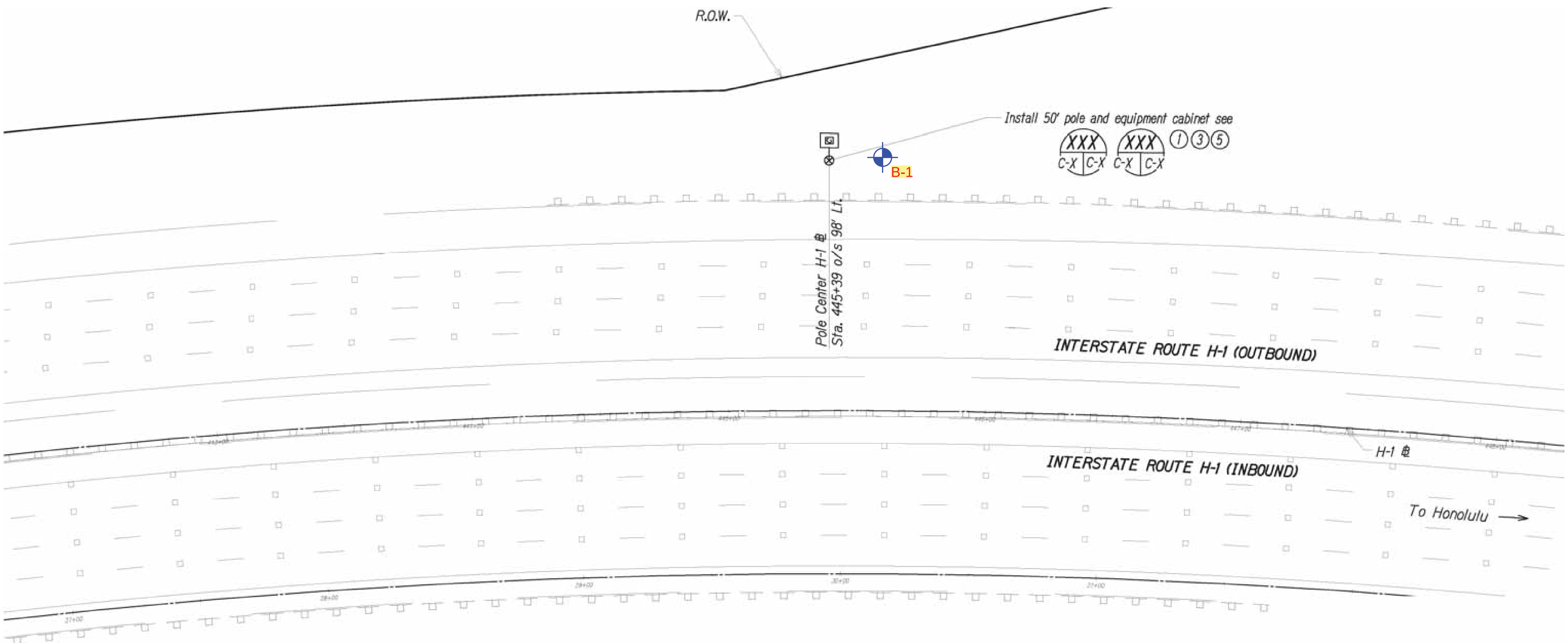
1" = 5000'	6891-30(A)	1



LEGEND:

APPROXIMATE BORING LOCATION

0 20 40 80 120 FT.

GRAPHIC SCALE

N

SITE PLAN

FREEWAY MANAGEMENT SYSTEM
PHASE 3, UNITS 1 AND 2, IM-0300(152)
INTERSTATE ROUTES H-1 AND H-2 FREEWAYS
DISTRICTS OF HONOLULU AND EWA, OAHU, HAWAII

GEOLABS, INC.
Geotechnical Engineering

DATE	DRAWN BY	PLATE
SEPTEMBER 2020	ASP	
SCALE	W.O.	
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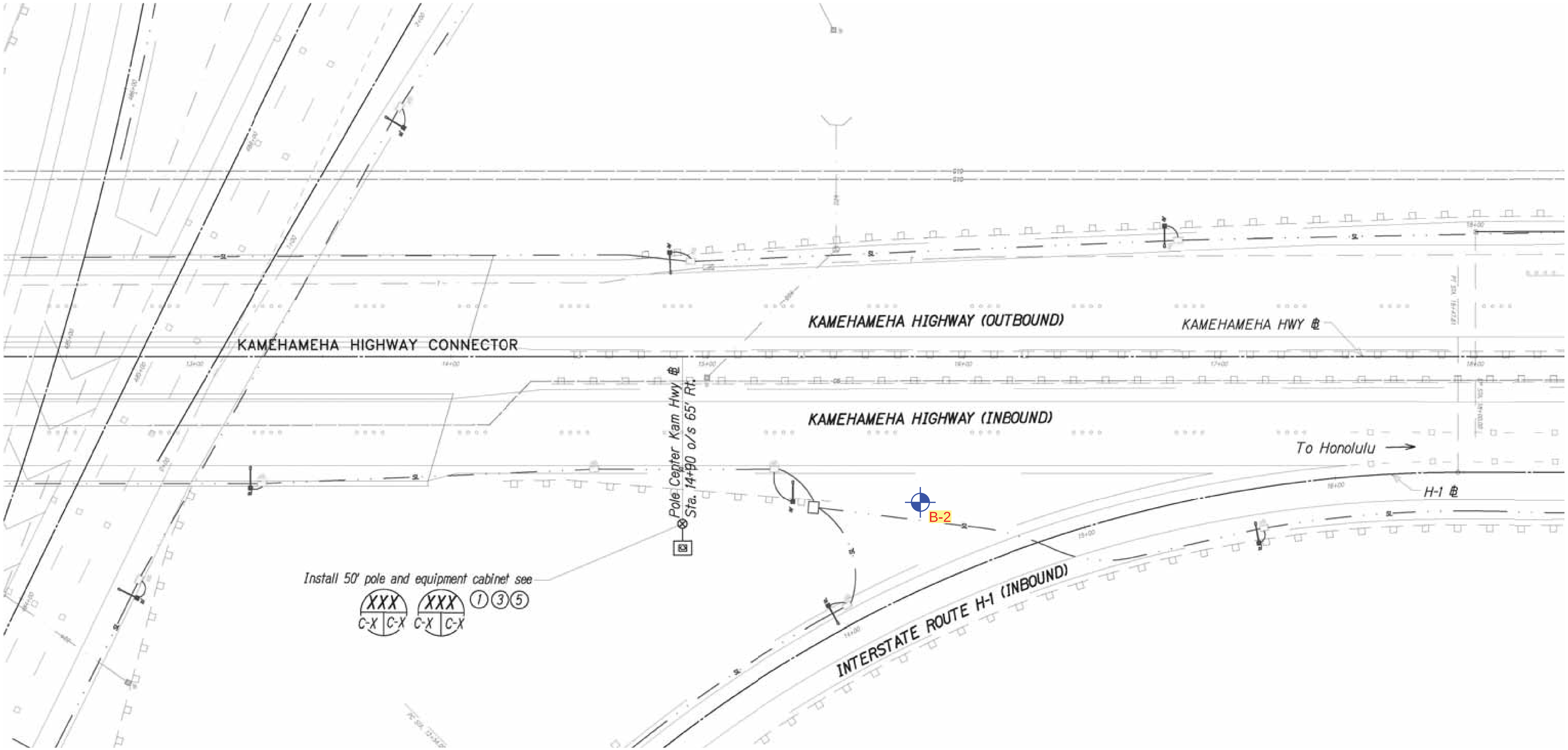
REFERENCE: WAIKELE CCTV SITE PLAN TRANSMITTED BY AUSTIN, TSUTSUMI & ASSOCIATES, INC. ON MAY 29, 2020.



SITE PLAN

FREEWAY MANAGEMENT SYSTEM
PHASE 3, UNITS 1 AND 2, IM-0300(152)
INTERSTATE ROUTES H-1 AND H-2 FREEWAYS
DISTRICTS OF HONOLULU AND EWA, OAHU, HAWAII

LEGEND:

APPROXIMATE BORING LOCATION

0 20 40 80 120 FT.

GRAPHIC SCALE

N

GEOLABS, INC.

Geotechnical Engineering

DATE	DRAWN BY	PLATE
SEPTEMBER 2020	ASP	
SCALE	W.O.	
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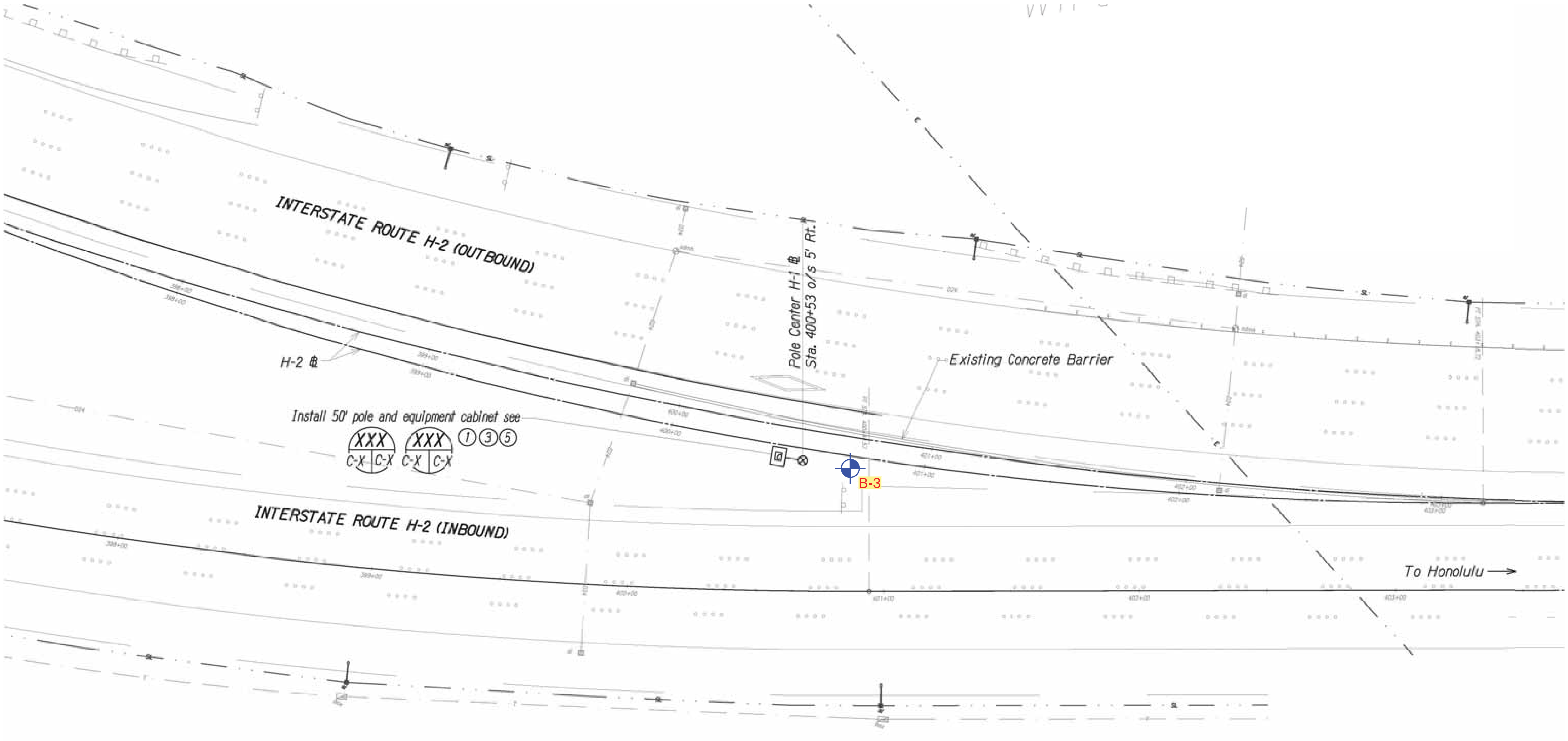
REFERENCE: WAIAWA IC CCTV SITE PLAN TRANSMITTED BY AUSTIN, TSUTSUMI & ASSOCIATES, INC. ON MAY 29, 2020.



LEGEND:

APPROXIMATE BORING LOCATION

0 20 40 80 120 FT.
GRAPHIC SCALE

REFERENCE: H-2 NORTH CCTV SITE PLAN TRANSMITTED BY AUSTIN, TSUTSUMI & ASSOCIATES, INC. ON MAY 29, 2020.

SITE PLAN

FREEWAY MANAGEMENT SYSTEM
PHASE 3, UNITS 1 AND 2, IM-0300(152)
INTERSTATE ROUTES H-1 AND H-2 FREEWAYS
DISTRICTS OF HONOLULU AND EWA, OAHU, HAWAII

GEOLABS, INC.
Geotechnical Engineering

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SEPTEMBER 2020	ASP	
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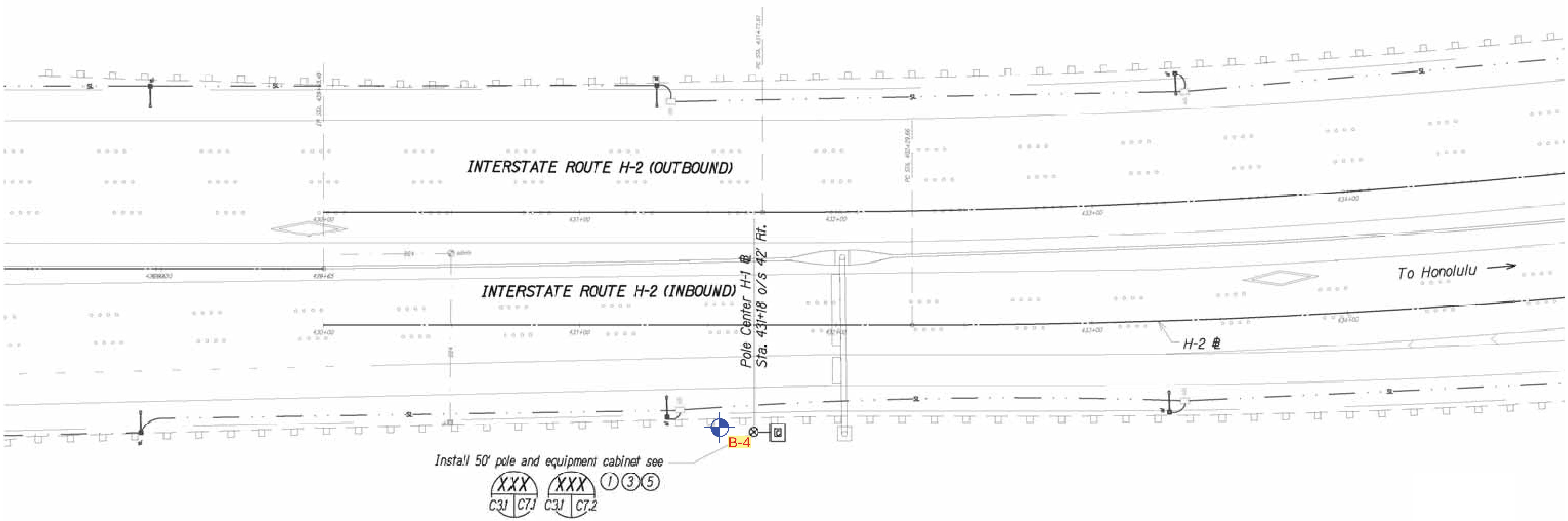


LEGEND:

APPROXIMATE BORING LOCATION

0 20 40 80 120 FT.

GRAPHIC SCALE

N

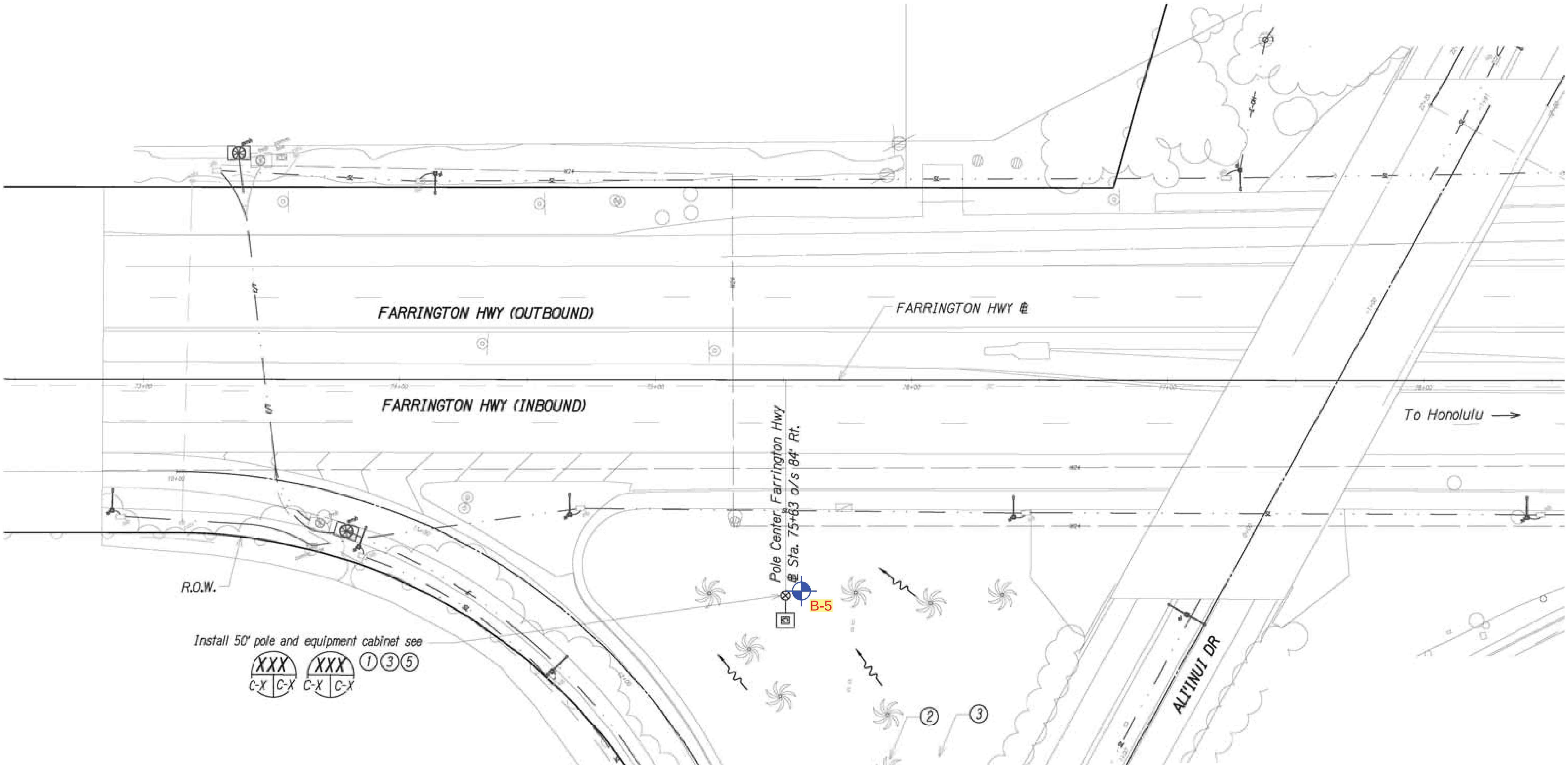
SITE PLAN

FREEWAY MANAGEMENT SYSTEM
PHASE 3, UNITS 1 AND 2, IM-0300(152)
INTERSTATE ROUTES H-1 AND H-2 FREEWAYS
DISTRICTS OF HONOLULU AND EWA, OAHU, HAWAII

GEOLABS, INC.
Geotechnical Engineering

DATE	DRAWN BY	PLATE
SEPTEMBER 2020	ASP	
SCALE	W.O.	
1" = 40'	6891-30(A)	2.4

REFERENCE: H-2 SOUTH CCTV SITE PLAN TRANSMITTED BY AUSTIN, TSUTSUMI & ASSOCIATES, INC. ON MAY 29, 2020.



SITE PLAN

FREEWAY MANAGEMENT SYSTEM
PHASE 3, UNITS 1 AND 2, IM-0300(152)
INTERSTATE ROUTES H-1 AND H-2 FREEWAYS
DISTRICTS OF HONOLULU AND EWA, OAHU, HAWAII

LEGEND:

⊕ APPROXIMATE BORING LOCATION

0 20 40 80 120 FT.
GRAPHIC SCALE

N

GEOLABS, INC. *Geotechnical Engineering*		
DATE: SEPTEMBER 2020	DRAWN BY: ASP	PLATE
SCALE: 1" = 40'	W.O.: 6891-30(A)	2.5

REFERENCE: FARRINGTON HWY CCTV SITE PLAN TRANSMITTED BY AUSTIN, TSUTSUMI & ASSOCIATES, INC. ON MAY 29, 2020.

APPENDIX A

A P P E N D I X   A

Field Exploration

We explored the subsurface conditions by drilling and sampling five borings, designated as Boring Nos. 1 through 5, to depths of about 15.5 to 21.5 feet below the existing ground surface at the approximate locations shown on the Site Plans, Plates 2.1 through 2.5. We used a truck-mounted drill rig equipped with solid-stem augers and rotary coring tools.

Our geologists classified the materials encountered in the borings by visual and textural examination in the field in general accordance with ASTM D2488, Standard Practice for Description and Identification of Soils, and monitored the drilling operations on a near-continuous (full-time) basis. These classifications were further reviewed visually and by testing in the laboratory. Soils were classified in general accordance with ASTM D2487, Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System), as shown on the Soil Log Legend, Plate A-0.1. Deviations made to the soil classification in accordance with ASTM D2487 are described on the Soil Classification Log Key, Plate A-0.2. Graphic representations of the materials encountered are presented on the Logs of Borings, Plates A-1 through A-5.

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

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

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

"Suggested Methods for the Quantitative Description of Discontinuities in Rock Masses" by the International Society for Rock Mechanics (March 1977).

Recovery (REC) is used as a subjective guide to the interpretation of the relative quality of rock masses. Recovery is defined as the actual length of material recovered from a coring attempt versus the length of the core attempt. For example, if 4 feet of material is recovered from a 5-foot core run, the recovery would be 80 percent and would be shown on the Logs of Borings as REC = 80%.

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-induced fractures or breaks. If 2.5 feet of sound material is recovered from a 5-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."

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 rip and would probably require more arduous methods of excavation.

Soil Log Legend

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)

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

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

LEGEND

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

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

GEOLABS, INC.

Geotechnical Engineering

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

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)

ABBREVIATIONS

WOH: Weight of Hammer

WOR: Weight of Drill Rods

SPT: Standard Penetration Test Split-Spoon Sampler

MCS: Modified California Sampler

PP: Pocket Penetrometer

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

Plate

A-0.2

GEOLABS, INC.

Geotechnical Engineering

Rock Log Legend

ROCK DESCRIPTIONS

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

ROCK DESCRIPTION SYSTEM

ROCK FRACTURE CHARACTERISTICS

The following terms describe general fracture spacing of a rock:

Massive: Greater than 24 inches apart

Slightly Fractured: 12 to 24 inches apart

Moderately Fractured: 6 to 12 inches apart

Closely Fractured: 3 to 6 inches apart

Severely Fractured: Less than 3 inches apart

DEGREE OF WEATHERING

The following terms describe the chemical weathering of a rock:

Unweathered: Rock shows no sign of discoloration or loss of strength.

Slightly Weathered: Slight discoloration inwards from open fractures.

Moderately Weathered: Discoloration throughout and noticeably weakened though not able to break by hand.

Highly Weathered: Most minerals decomposed with some corestones present in residual soil mass. Can be broken by hand.

Extremely Weathered: Saprolite. Mineral residue completely decomposed to soil but fabric and structure preserved.

HARDNESS

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

Very Hard: Specimen breaks with difficulty after several "pinging" hammer blows.
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 6891-30A&B.GPJ GEOLABS.GDT 9/17/20

GEOLABS, INC. Geotechnical Engineering	FREEWAY MANAGEMENT SYSTEM PHASE 3, UNITS 1 AND 2, IM-0300(152) INTERSTATE ROUTES H-1 AND H-2 FREEWAYS DISTRICTS OF HONOLULU AND EWA, OAHU, HAWAII	Log of Boring 1

Approximate Ground Surface Elevation (feet): 168 *

Laboratory			Field								
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)	Depth (feet)	Sample	Graphic	USCS	Description
							0			CH	Reddish brown **SILTY CLAY**, very stiff, moist (residual soil)
Direct Shear	23	69			36	4.5					
LL=71 PI=42	20				18						
											Brownish gray **BOULDERS (BASALTIC)**
Sieve - #200 = 31.6%	35	80			10		5			SM	Reddish brown with gray mottling **SILTY SAND (BASALTIC)** with a little gravel (basaltic), loose, moist (saprolite)
UC= 3710 psi	3		100	100	20/2"		10				Gray **BASALT**, closely to slightly fractured, moderately to slightly weathered, hard to very hard (basalt formation)
UC= 2910 psi			100	80			15				grades to moderately to severely fractured
							20				Boring terminated at 20 feet
							25				
							30				
							35				

Date Started:	September 1, 2020	Water Level: ▼	Not Encountered	Plate
Date Completed:	September 1, 2020			
Logged By:	D. Gremminger	Drill Rig:	CME-45C TRUCK	A - 1
Total Depth:	20 feet	Drilling Method:	4" Solid-Stem Auger & PQ Coring	
Work Order:	6891-30(A)	Driving Energy:	140 lb. wt., 30 in. drop	

GEOLABS, INC. Geotechnical Engineering	FREEWAY MANAGEMENT SYSTEM PHASE 3, UNITS 1 AND 2, IM-0300(152) INTERSTATE ROUTES H-1 AND H-2 FREEWAYS DISTRICTS OF HONOLULU AND EWA, OAHU, HAWAII	Log of Boring 2

Approximate Ground Surface Elevation (feet): 117 *

Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)	Depth (feet)	USCS	Description
							0	ML	Reddish brown **SANDY SILT** with a little gravel (basaltic), very stiff, moist (fill)
Direct Shear	19	67			38	4.5			
	22				26				
Direct Shear	18	66			12		5		grades to medium stiff
							6	SC	Reddish brown **CLAYEY SAND** with some gravel, loose, moist (fill)
LL=57 PI=29	30				10		10		
							14	MH	Reddish brown with gray mottling **CLAYEY SILT** with some sand, medium stiff, moist (residual soil)
	35	79			13	2.0	15		
	39				4		20	MH	Brown **CLAYEY SILT** with some sand, soft, wet (residual soil)
							21.5		Boring terminated at 21.5 feet

Date Started:	September 1, 2020	Water Level:	Not Encountered	Plate A - 2
Date Completed:	September 1, 2020			
Logged By:	D. Gremminger	Drill Rig:	CME-45C TRUCK	
Total Depth:	21.5 feet	Drilling Method:	4" Solid-Stem Auger	
Work Order:	6891-30(A)	Driving Energy:	140 lb. wt., 30 in. drop	

BORING_LOG 6891-30A&B.GPJ GEOLABS.GDT 2/9/21

	GEOLABS, INC. Geotechnical Engineering	FREEWAY MANAGEMENT SYSTEM PHASE 3, UNITS 1 AND 2, IM-0300(152) INTERSTATE ROUTES H-1 AND H-2 FREEWAYS DISTRICTS OF HONOLULU AND EWA, OAHU, HAWAII	Log of Boring 3

Approximate Ground Surface Elevation (feet): 296 *

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)	Sample	Graphic	USCS	Description
LL=63 PI=41	18	85			50	4.5				CH	Reddish brown **SILTY CLAY** with some gravel (basaltic), very stiff, dry (fill)
	8		100	16	50/4"						Gray **BASALT**, severely to moderately fractured, moderately to slightly weathered, hard to very hard (basalt formation)
UC= 13160 psi			100	23			5				
UC= 29720 psi			83	0			10				
							15				Boring terminated at 15.5 feet
							20				
							25				
							30				
							35				

Date Started:	September 2, 2020	Water Level:	Not Encountered	Plate
Date Completed:	September 2, 2020			
Logged By:	D. Gremminger	Drill Rig:	CME-45C TRUCK	
Total Depth:	15.5 feet	Drilling Method:	4" Solid-Stem Auger & PQ Coring	A - 3
Work Order:	6891-30(A)	Driving Energy:	140 lb. wt., 30 in. drop	

GEOLABS, INC. Geotechnical Engineering	FREEWAY MANAGEMENT SYSTEM PHASE 3, UNITS 1 AND 2, IM-0300(152) INTERSTATE ROUTES H-1 AND H-2 FREEWAYS DISTRICTS OF HONOLULU AND EWA, OAHU, HAWAII	Log of Boring 4

Approximate Ground Surface Elevation (feet): 195 *

Other Tests (Laboratory)	Moisture Content (%) (Laboratory)	Dry Density (pcf) (Laboratory)	Core Recovery (%) (Field)	RQD (%) (Field)	Penetration Resistance (blows/foot) (Field)	Pocket Pen. (tsf) (Field)	Depth (feet)	USCS	Description
							0		3-inch **ASPHALTIC CONCRETE**
TXUU	27	96			41	4.5		GP-GM	Reddish gray **SANDY GRAVEL (BASALTIC)** with a little silt, dry (fill)
LL=53 PI=24	26				25			MH	Reddish brown **CLAYEY SILT** with some sand and gravel, very stiff, dry (fill)
	25	76			65	4.5	5		grades to hard
								GM	Grayish brown **SILTY GRAVEL (BASALTIC)** with some sand and a little clay, medium dense, dry (residual soil)
	17				22		10		
UC= 9790 psi			43						Brownish gray **COBBLY BOULDERS (BASALTIC)** with some clay seams, very dense, dry (residual soil)
			70				15		
					20/0" Ref.		20		Boring terminated at 20.5 feet
							25		
							30		
							35		

Date Started:	September 2, 2020	Water Level: ▼	Not Encountered	Plate
Date Completed:	September 2, 2020			
Logged By:	D. Gremminger	Drill Rig:	CME-45C TRUCK	A - 4
Total Depth:	20.5 feet	Drilling Method:	4" Solid-Stem Auger & PQ Coring	
Work Order:	6891-30(A)	Driving Energy:	140 lb. wt., 30 in. drop	

GEOLABS, INC. Geotechnical Engineering	FREEWAY MANAGEMENT SYSTEM PHASE 3, UNITS 1 AND 2, IM-0300(152) INTERSTATE ROUTES H-1 AND H-2 FREEWAYS DISTRICTS OF HONOLULU AND EWA, OAHU, HAWAII	Log of Boring 5

Approximate Ground Surface Elevation (feet): 62 *

Other Tests (Laboratory)	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%) (Field)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)	Depth (feet)	USCS	Description
							0	SM	3-inch **GRASSED SURFACE**
TXUU	15	106			22				Brownish tan **SILTY SAND (CORALLINE)** with some gravel (coralline), medium dense, moist (fill)
Sieve - #200 = 15.3%	9				25				grades with some silty clay
Direct Shear	22	74			36		5		grades with a little cobbles (coralline)
	18				14		10		grades to wet
								GP-GM	Brownish gray **SANDY GRAVEL (BASALTIC)** with a little silt, dense, moist (colluvium)
	16	90			64		15		
	11				34		20	SM	Brownish gray **SILTY SAND (BASALTIC)** with some gravel (basaltic), medium dense, moist (colluvium)
									Boring terminated at 21.5 feet
							25		
							30		
							35		

Date Started:	August 31, 2020	Water Level: ▼	Not Encountered	Plate
Date Completed:	August 31, 2020			
Logged By:	D. Gremminger	Drill Rig:	CME-45C TRUCK	
Total Depth:	21.5 feet	Drilling Method:	4" Solid-Stem Auger	A - 5
Work Order:	6891-30(A)	Driving Energy:	140 lb. wt., 30 in. drop	

APPENDIX B

APPENDIX B

Laboratory Tests

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

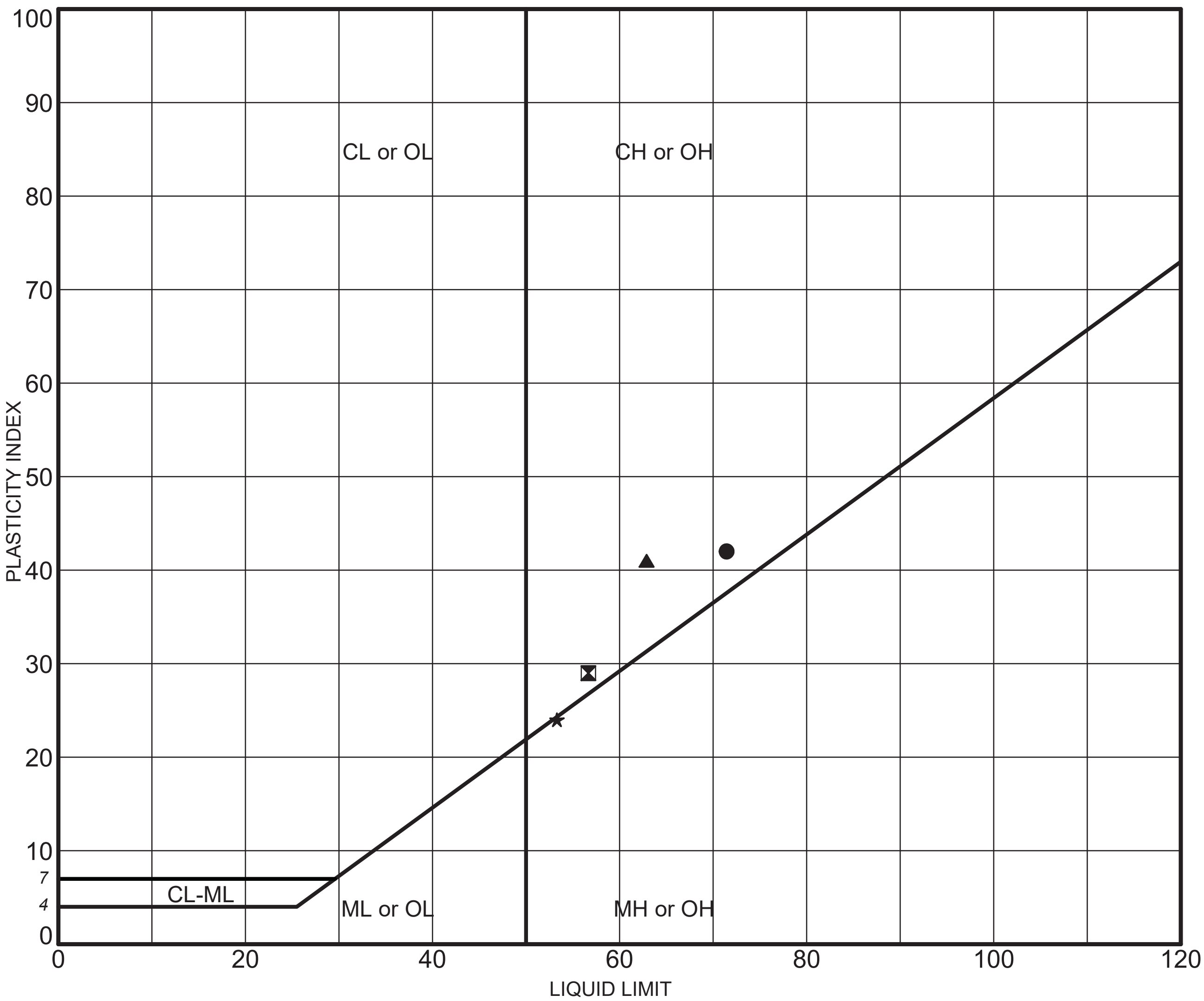
Four Atterberg Limits tests (ASTM D4318) were performed on selected soil samples to evaluate the liquid and plastic limits and to aid in soil classification. The test results are summarized on the Logs of Borings at the appropriate sample depths. The test results are provided on Plate B-1.

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

Two Unconsolidated Undrained Triaxial Compression tests (ASTM D2850) were performed on selected soil samples to evaluate the undrained shear strength of the soils. The undrained shear strength test results along with the shear stress-strain curves are presented on Plates B-3 and B-4.

Four 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 B-5 through B-8.

Five Uniaxial Compression tests (ASTM D7012, Method C) were performed on selected core samples to evaluate the unconfined compressive strength of the basalt rock encountered. The test results are presented on Plate B-9.



	Sample	Depth (ft)	LL	PL	PI	Description
●	B-1	2.5-4.0	71	29	42	Reddish brown silty clay (CH)
⊠	B-2	10.0-11.5	57	28	29	Reddish brown clayey sand (SC) with some gravel
▲	B-3	1.0-2.5	63	22	41	Reddish brown silty clay (CH) with some gravel
★	B-4	2.5-4.0	53	29	24	Reddish brown clayey silt (MH) with some sand and gravel

NP = NON-PLASTIC

GEOLABS, INC.

GEOTECHNICAL ENGINEERING

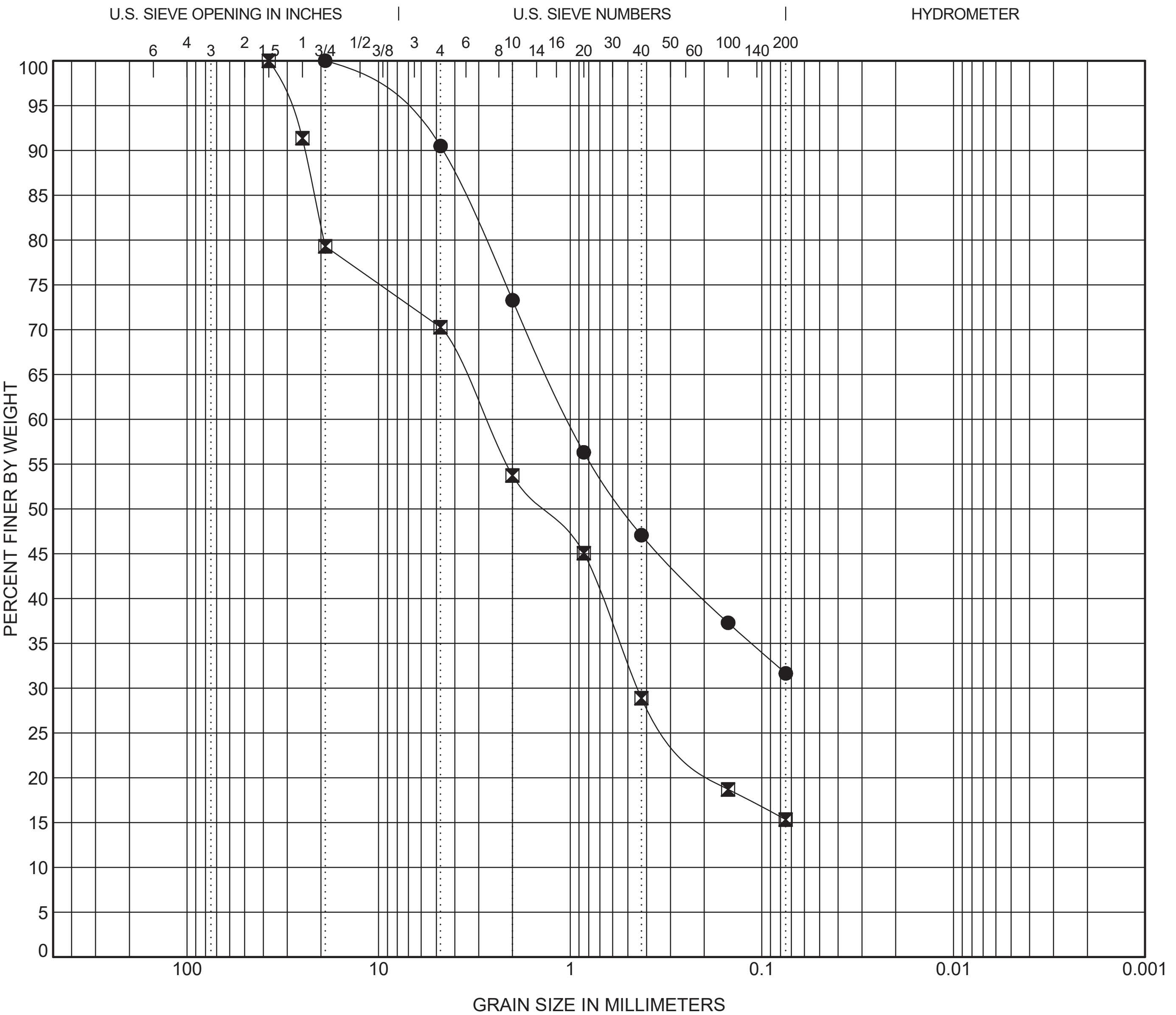
W.O. 6891-30(A)

ATTERBERG LIMITS TEST RESULTS - ASTM D4318

FREEWAY MANAGEMENT SYSTEM
PHASE 3, UNITS 1 AND 2, IM-0300(152)
INTERSTATE ROUTES H-1 AND H-2 FREEWAYS
DISTRICTS OF HONOLULU AND EWA, OAHU, HAWAII

Plate B - 1

G_ATTERBERG PI-100 LL-120 6891-30A&B.GPJ GEOLABS.GDT 2/9/21



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Sample	Depth (ft)	Description	LL	PL	PI	Cc	Cu
●	B-1	5.0-6.5	Reddish brown with gray mottling silty sand (SM) with a little gravel					
⧗	B-5	2.5-4.0	Brownish tan silty sand (SM) with some gravel					

	Sample	Depth (ft)	D100 (mm)	D60 (mm)	D30 (mm)	D10 (mm)	%Gravel	%Sand	%Fine
●	B-1	5.0-6.5	19	1.024			9.5	58.9	31.6
⧗	B-5	2.5-4.0	37.5	2.778	0.446		29.7	54.9	15.3

G_GRAIN_SIZE_MOD 6891-30A&B.GPJ GEOLABS.GDT 2/9/21

GEOLABS, INC.

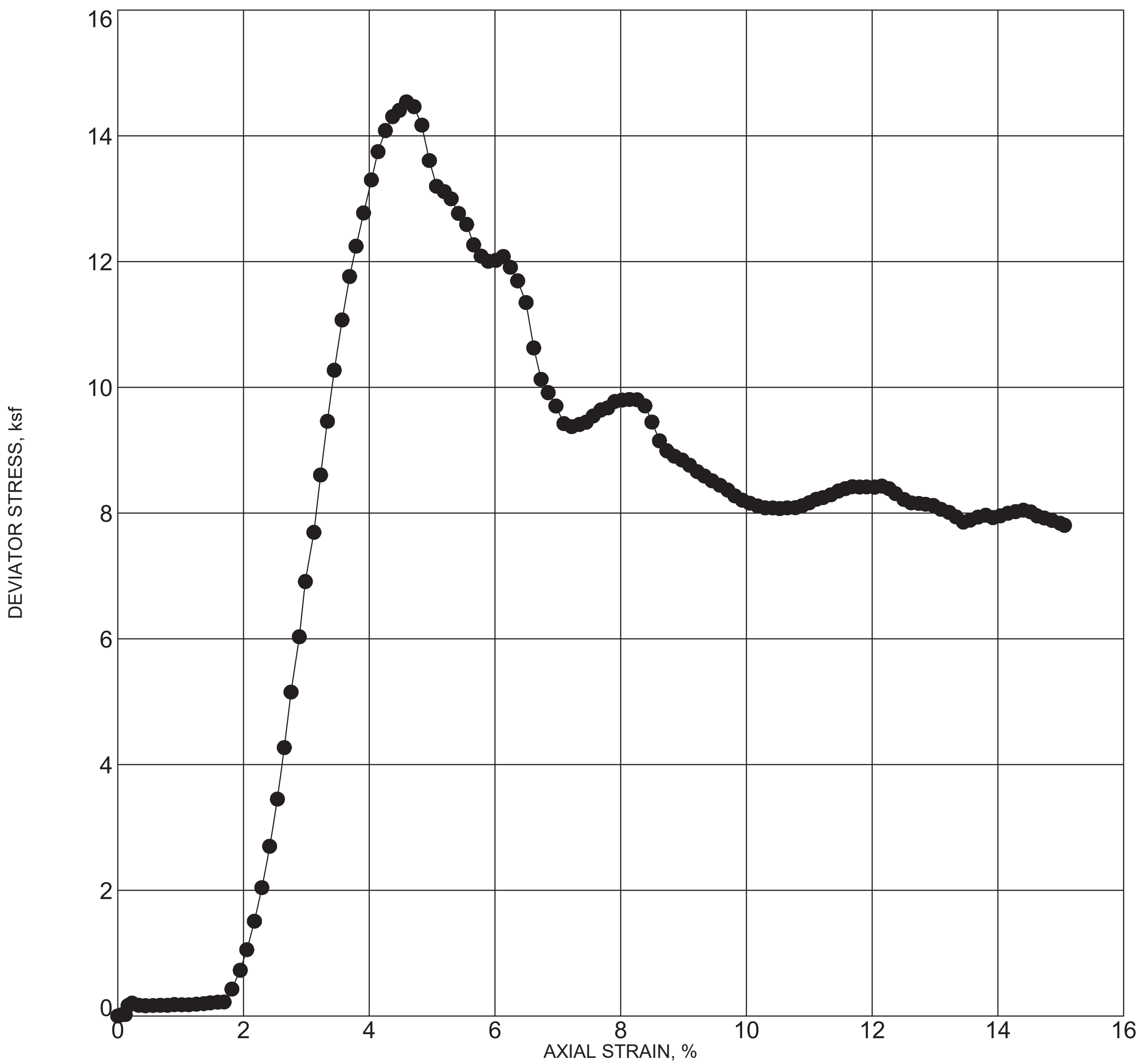
GEOTECHNICAL ENGINEERING

W.O. 6891-30(A)

GRAIN SIZE DISTRIBUTION - ASTM D6913

FREEWAY MANAGEMENT SYSTEM
PHASE 3, UNITS 1 AND 2, IM-0300(152)
INTERSTATE ROUTES H-1 AND H-2 FREEWAYS
DISTRICTS OF HONOLULU AND EWA, OAHU, HAWAII

Plate
B - 2



Max. Deviator Stress (ksf):	14.5
Confining Stress (ksf):	0.3

Location: B-4
Depth: 1.0 - 2.5 feet
Description: Reddish brown clayey silt (MH) with some sand and gravel
Test Date: 9/4/2020

Dry Density (pcf)	96.1	Sample Diameter (inches)	2.413
Moisture (%)	27.0	Sample Height (inches)	5.167
Axial Strain at Failure (%)	4.6	Strain Rate (% / minute)	0.69

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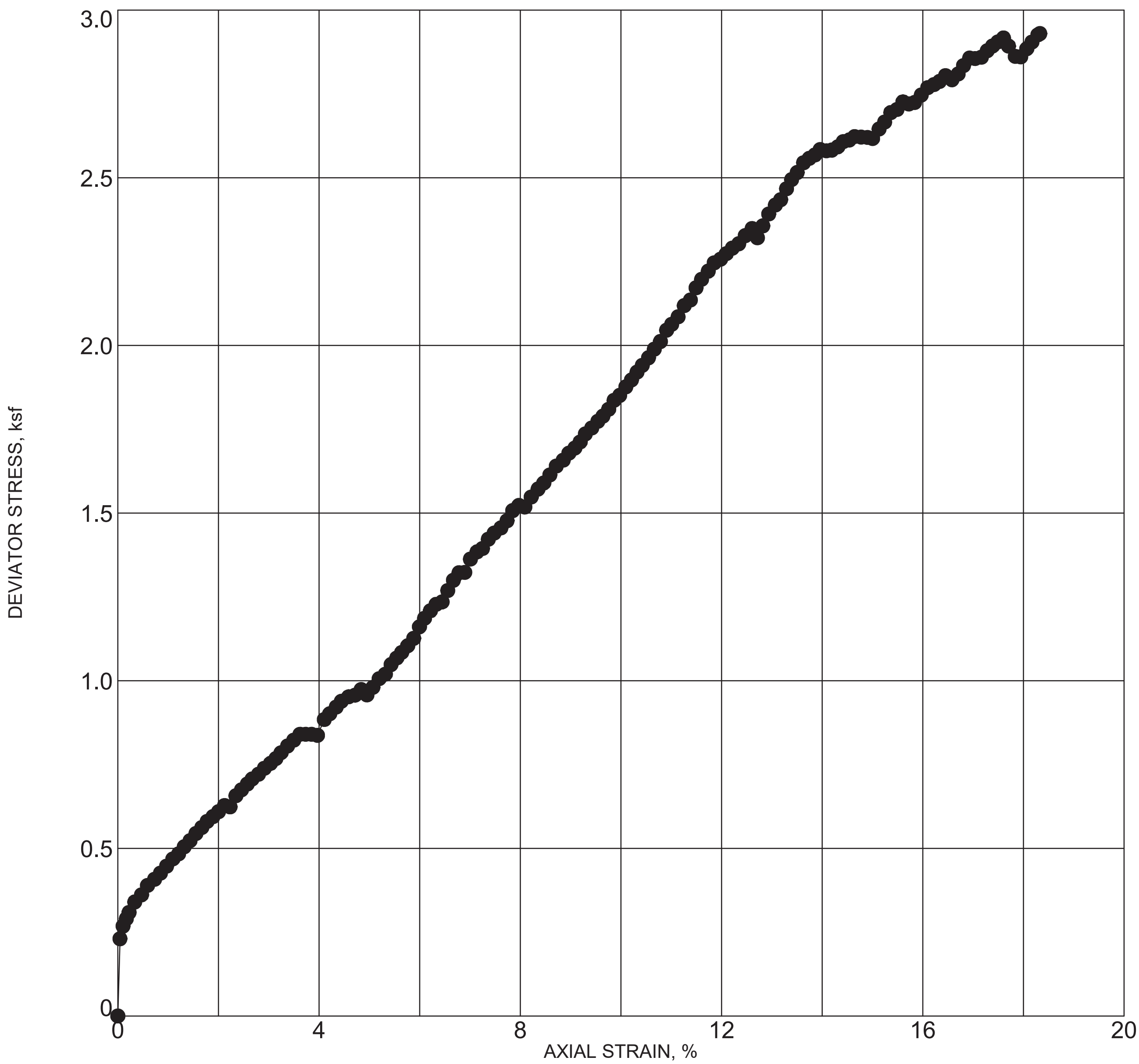
TRIAXIAL UU COMPRESSION TEST - ASTM D2850

FREEWAY MANAGEMENT SYSTEM
PHASE 3, UNITS 1 AND 2, IM-0300(152)
INTERSTATE ROUTES H-1 AND H-2 FREEWAYS
DISTRICTS OF HONOLULU AND EWA, OAHU, HAWAII

Plate
B - 3

G_TXUU 6891-30A&B.GPJ GEOLABS.GDT 2/9/21



Max. Deviator Stress (ksf):	2.6
Confining Stress (ksf):	0.3

Location: B-5
Depth: 1.0 - 2.5 feet
Description: Brownish tan silty sand (SM) with some gravel
Test Date: 9/4/2020

Dry Density (pcf)	106.3	Sample Diameter (inches)	2.413
Moisture (%)	14.8	Sample Height (inches)	5.133
Axial Strain at Failure (%)	15.0	Strain Rate (% / minute)	0.70

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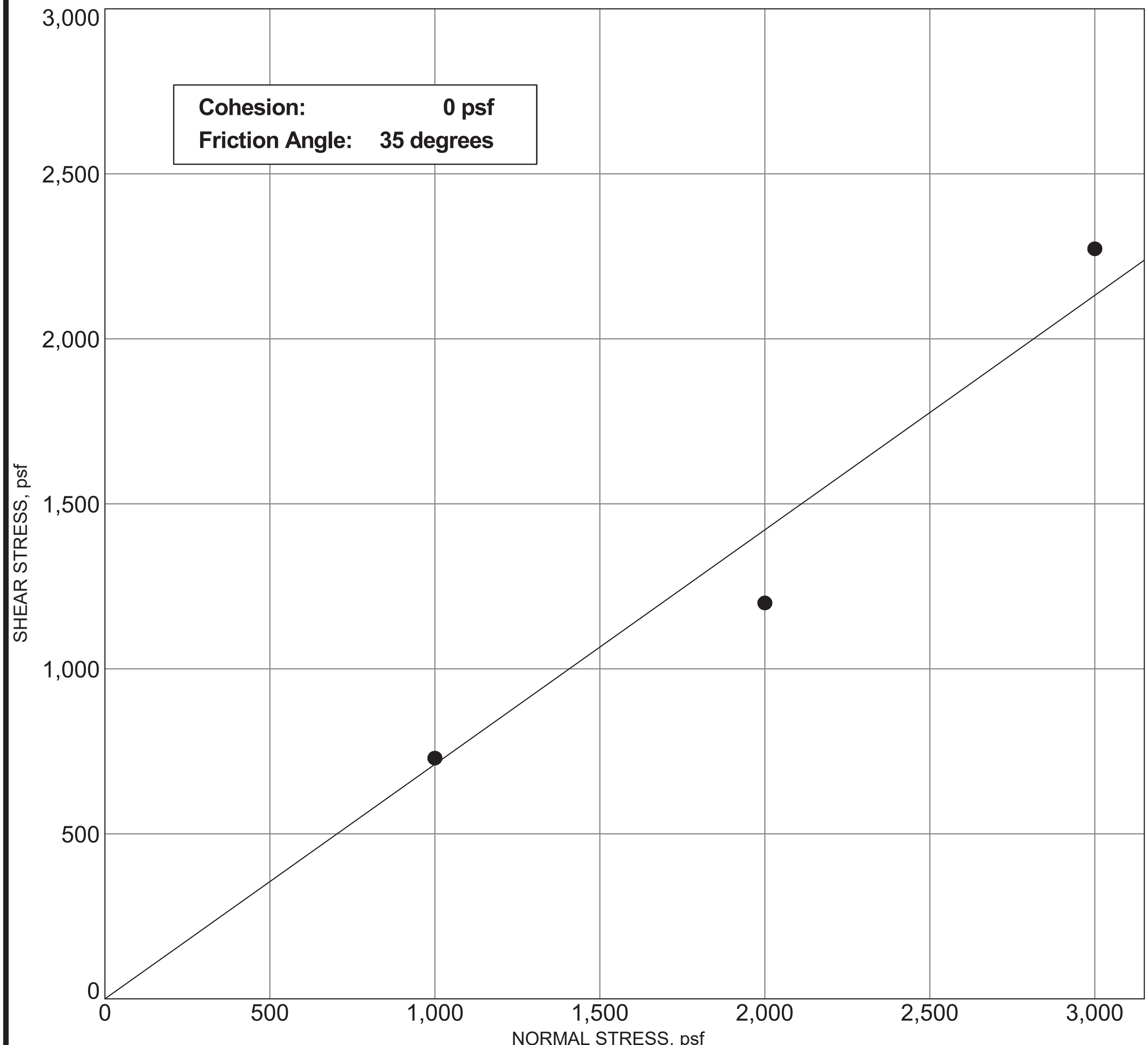
TRIAXIAL UU COMPRESSION TEST - ASTM D2850

FREEWAY MANAGEMENT SYSTEM
PHASE 3, UNITS 1 AND 2, IM-0300(152)
INTERSTATE ROUTES H-1 AND H-2 FREEWAYS
DISTRICTS OF HONOLULU AND EWA, OAHU, HAWAII

Plate
B - 4

G_TXUU 6891-30A&B.GPJ GEOLABS.GDT 2/9/21



		Sample #1	Sample #2	Sample #3
INITIAL	Moisture Content, %	22.8	19.1	20.6
	Dry Density, pcf	69.0	72.8	72.8
	Height, inches	1.00	1.00	1.00
FINAL	Moisture Content, %	40.9	37.7	35.9
	Dry Density, pcf	66.3	75.8	76.6
	Height, inches	1.041	0.961	0.951
	Diameter, inches	2.42	2.42	2.42
	Deformation Rate, inch/minute	0.0024	0.0022	0.0023
	Normal Stress, psf	**1000**	**2000**	**3000**
	Peak Shear Stress, psf	**729**	**1199**	**2273**
	Shear Displacement, inches	0.43	0.42	0.41

Sample: B-1
Depth: 1.0 - 2.5 feet
Description: Reddish brown silty clay

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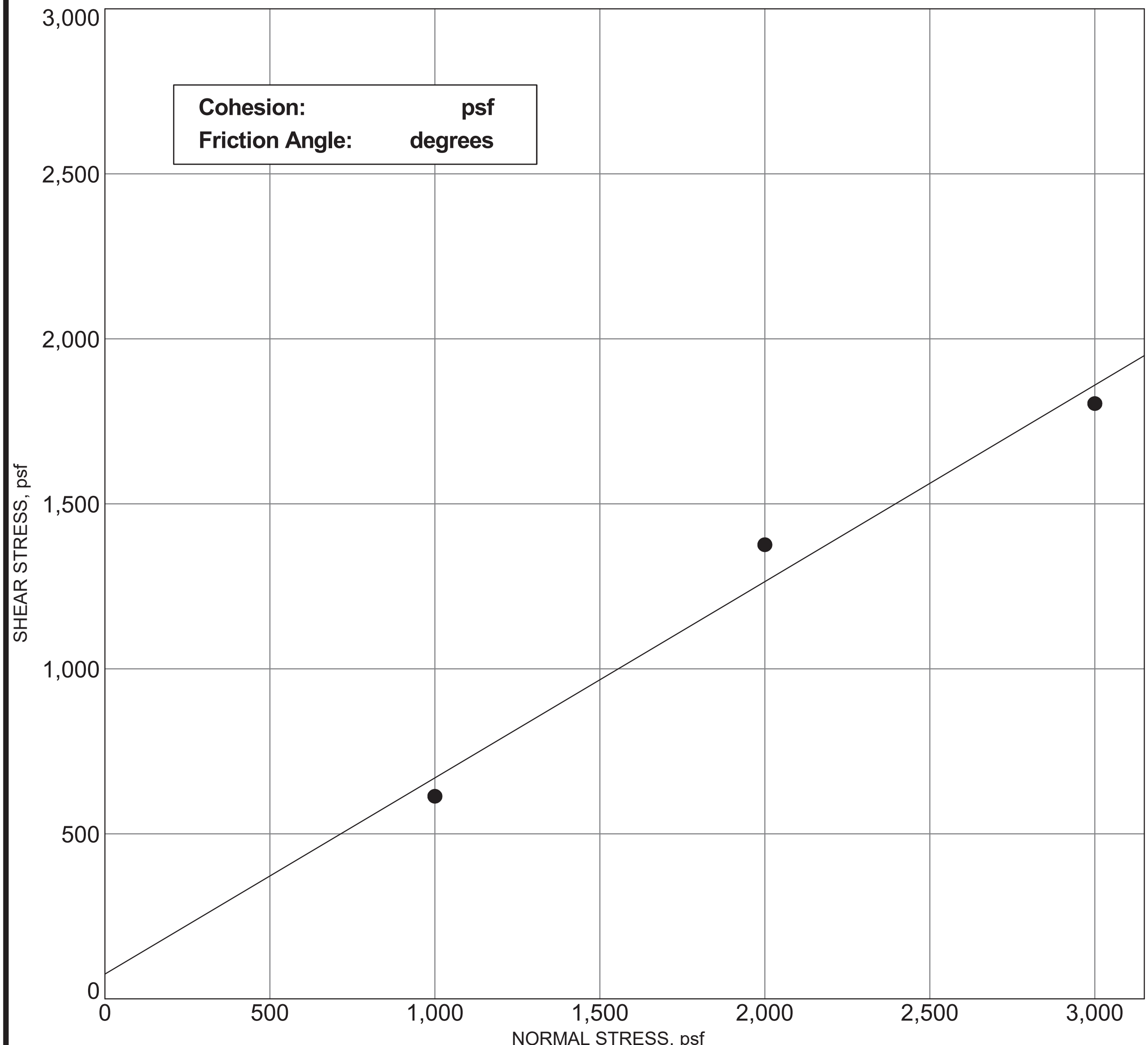
DIRECT SHEAR TEST - ASTM D3080

FREEWAY MANAGEMENT SYSTEM
PHASE 3, UNITS 1 AND 2, IM-0300(152)
INTERSTATE ROUTES H-1 AND H-2 FREEWAYS
DISTRICTS OF HONOLULU AND EWA, OAHU, HAWAII

Plate
B - 5



		Sample #1	Sample #2	Sample #3
INITIAL	Moisture Content, %	18.6	21.2	21.6
	Dry Density, pcf	66.7	67.8	67.9
	Height, inches	1.00	1.00	1.00
FINAL	Moisture Content, %	41.7	36.1	37.0
	Dry Density, pcf	64.3	71.2	70.4
	Height, inches	1.038	0.953	0.965
	Diameter, inches	2.42	2.42	2.42
	Deformation Rate, inch/minute	0.0024	0.0022	0.0024
	Normal Stress, psf	**1000**	**2000**	**3000**
	Peak Shear Stress, psf	**614**	**1376**	**1804**
	Shear Displacement, inches	0.43	0.41	0.42

Sample: B-2
Depth: 1.0 - 2.5 feet
Description: Reddish brown sandy silt with a little gravel

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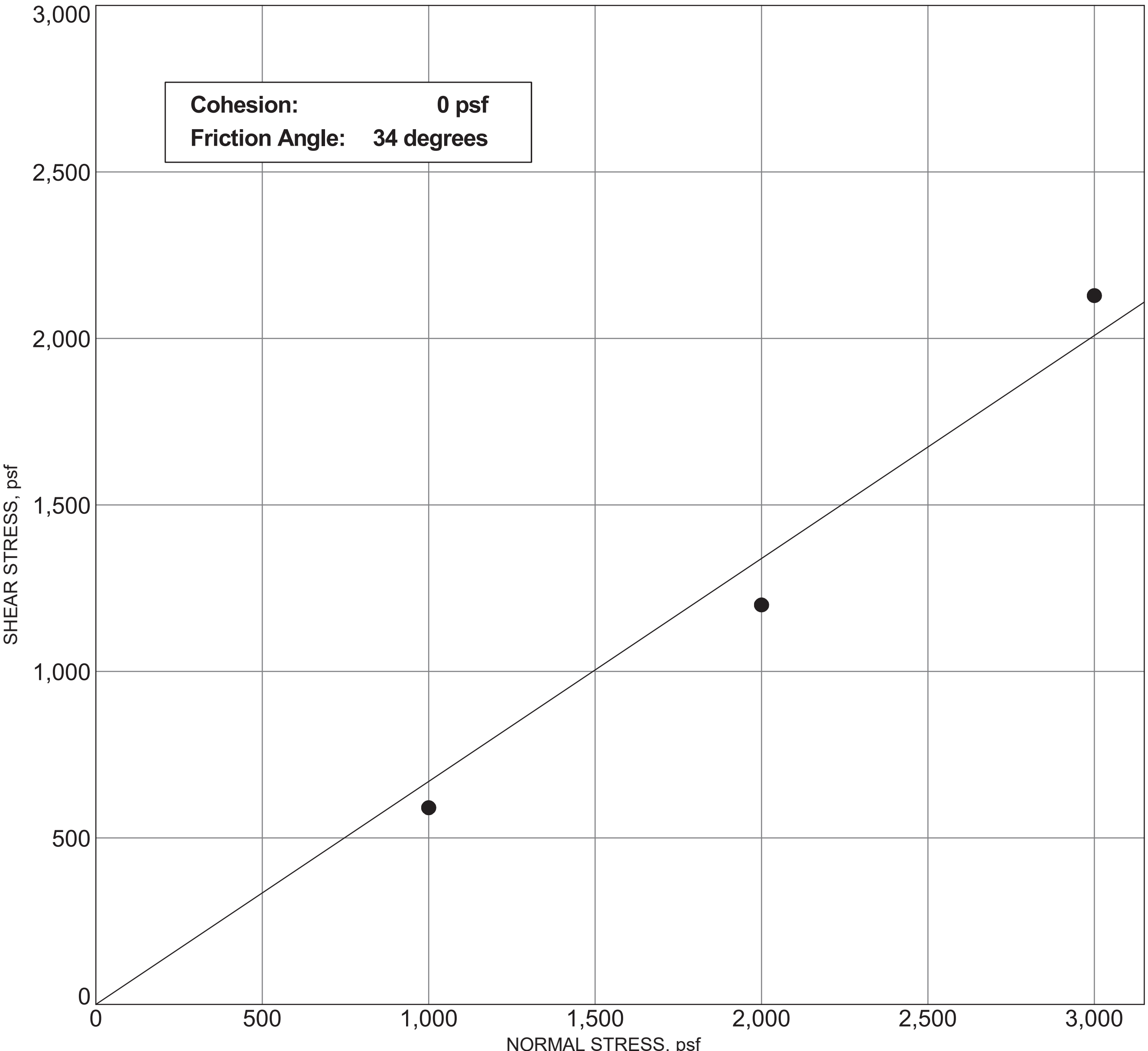
DIRECT SHEAR TEST - ASTM D3080

FREEWAY MANAGEMENT SYSTEM
PHASE 3, UNITS 1 AND 2, IM-0300(152)
INTERSTATE ROUTES H-1 AND H-2 FREEWAYS
DISTRICTS OF HONOLULU AND EWA, OAHU, HAWAII

Plate
B - 6



		Sample #1	Sample #2	Sample #3
INITIAL	Moisture Content, %	18.3	29.2	21.0
	Dry Density, pcf	66.0	63.3	66.8
	Height, inches	1.00	1.00	1.00
FINAL	Moisture Content, %	37.8	34.4	31.4
	Dry Density, pcf	63.8	66.6	67.5
	Height, inches	1.035	0.950	0.990
	Diameter, inches	2.42	2.42	2.42
	Deformation Rate, inch/minute	0.0025	0.0022	0.0024
	Normal Stress, psf	**1000**	**2000**	**3000**
	Peak Shear Stress, psf	**591**	**1199**	**2128**
	Shear Displacement, inches	0.43	0.41	0.42

Sample: B-2
Depth: 5.0 - 6.5 feet
Description: Reddish brown clayey sand with some gravel

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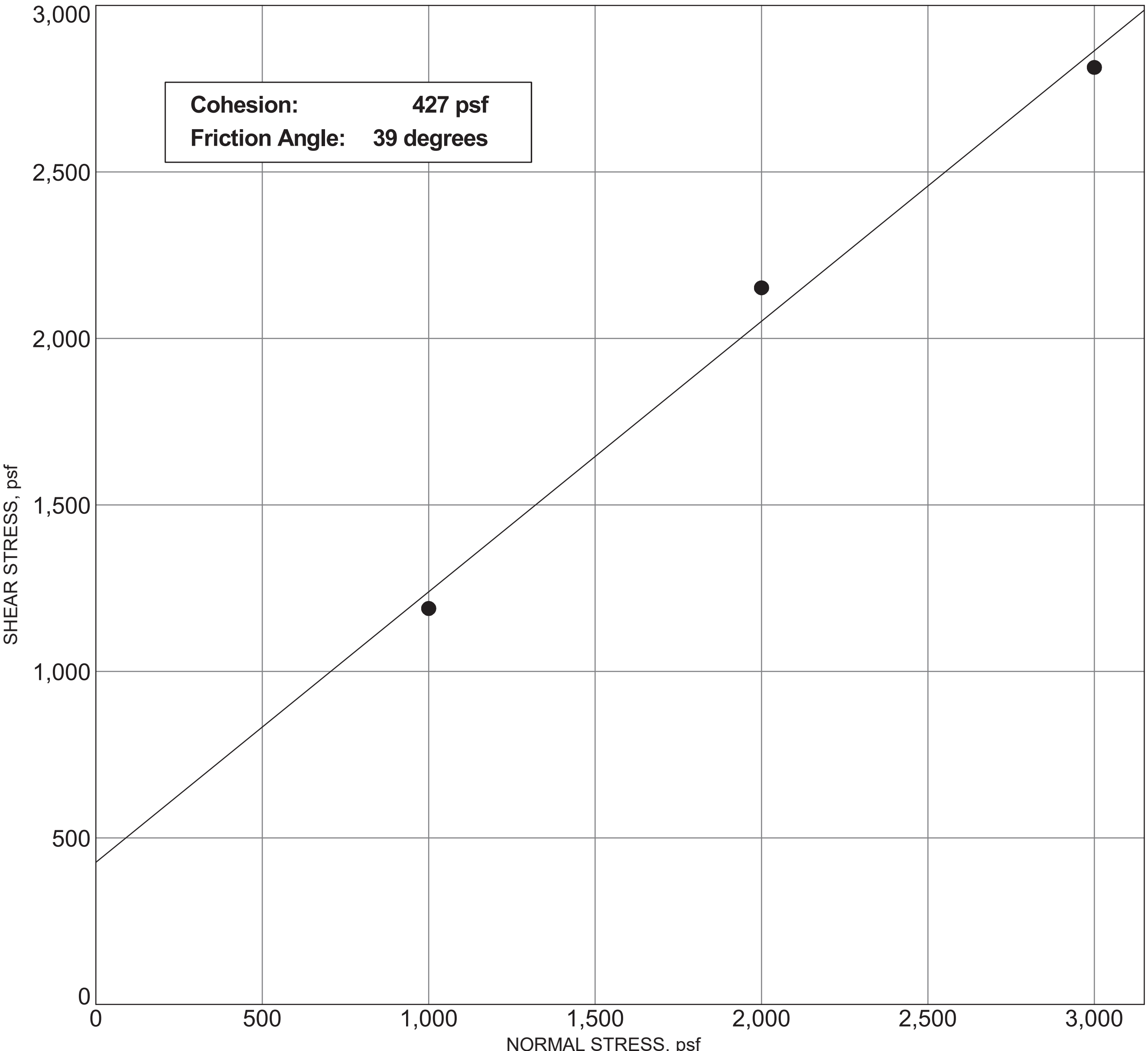
DIRECT SHEAR TEST - ASTM D3080

FREEWAY MANAGEMENT SYSTEM
PHASE 3, UNITS 1 AND 2, IM-0300(152)
INTERSTATE ROUTES H-1 AND H-2 FREEWAYS
DISTRICTS OF HONOLULU AND EWA, OAHU, HAWAII

Plate
B - 7



Sample: B-5
Depth: 5.0 - 6.5 feet
Description: Brownish tan silty sand (SM) with some gravel

		Sample #1	Sample #2	Sample #3
INITIAL	Moisture Content, %	22.0	22.3	17.9
	Dry Density, pcf	73.7	74.3	84.1
	Height, inches	1.00	1.00	1.00
FINAL	Moisture Content, %	31.8	30.8	27.8
	Dry Density, pcf	73.2	76.1	86.7
	Height, inches	1.006	0.976	0.971
	Diameter, inches	2.42	2.42	2.42
	Deformation Rate, inch/minute	0.0024	0.0020	0.0024
	Normal Stress, psf	**1000**	**2000**	**3000**
	Peak Shear Stress, psf	**1189**	**2152**	**2814**
	Shear Displacement, inches	0.43	0.40	0.42

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DIRECT SHEAR TEST - ASTM D3080

FREEWAY MANAGEMENT SYSTEM
PHASE 3, UNITS 1 AND 2, IM-0300(152)
INTERSTATE ROUTES H-1 AND H-2 FREEWAYS
DISTRICTS OF HONOLULU AND EWA, OAHU, HAWAII

Plate
B - 8

Location	Depth	Length	Diameter	Length/ Diameter Ratio	Density	Load	Compressive Strength
	(feet)	(inches)	(inches)		(pcf)	(lbs)	(psi)
B-1	10.5 - 15	6.659	3.257	2.04	117.9	30,940	3,710
B-1	15 - 20	6.624	3.305	2.00	120.1	25,000	2,910
B-3	5.5 - 10.5	6.603	3.219	2.05	172.4	107,070	13,160
B-3	10.5 - 15.5	6.628	3.258	2.03	170.1	247,660	29,720
B-4	12 - 15.5	6.668	3.250	2.05	144.7	81,220	9,790

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

FREEWAY MANAGEMENT SYSTEM
PHASE 3, UNITS 1 AND 2, IM-0300(152)
INTERSTATE ROUTES H-1 AND H-2 FREEWAYS
DISTRICTS OF HONOLULU AND EWA, OAHU, HAWAII

APPENDIX C

FREEWAY MANAGEMENT SYSTEM
PHASE 3, UNITS 1 AND 2, IM-0300(152)
INTERSTATE ROUTES H-1 AND H-2 FREEWAYS
DISTRICTS OF HONOLULU AND EWA, OAHU, HAWAII

B-1 10.5' TO 20.0'

10.5' 15.0'

15.0' 20.0'

FREEWAY MANAGEMENT SYSTEM
PHASE 3, UNITS 1 AND 2, IM-0300(152)
INTERSTATE ROUTES H-1 AND H-2 FREEWAYS
DISTRICTS OF HONOLULU AND EWA, OAHU, HAWAII

B-3 2.5' TO 15.5'

2.5' **5.5'** **10.5'**

5.5' **10.5'** **15.5'**

B-4 12.0' TO 20.0'

12.0'

20.0'