GEOTECHNICAL ENGINEERING EXPLORATION

TRAFFIC SIGNAL MODERNIZATION PROJECT

KALANIANAOLE HIGHWAY & KALANIIKI STREET INTERSECTION

HONOLULU, OAHU, HAWAII

W.O. 7328-20(B)   JUNE 13, 2024

Prepared for

ENGINEERING CONCEPTS INC.

GEOLABS, INC.

Geotechnical Engineering and Drilling Services

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Prepared for

ENGINEERING CONCEPTS 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

4-30-26
EXPIRATION DATE OF THE LICENSE

GEOLABS, INC.
Geotechnical Engineering and Drilling Services
94-429 Koaki Street, Suite 200 • Waipahu, Hawaii 96797

Hawaii • California

June 13, 2024
W.O. 7328-20(B)

Mr. Conrad Higashionna
Engineering Concepts Inc.
1150 South King Street, Suite 700
Honolulu, HI 96814

Dear **Mr. Higashionna:**

Geolabs, Inc. is pleased to submit our report entitled "Geotechnical Engineering Exploration, Traffic Signal Modernization Project, Kalanianaole Highway and Kalaniiki Street Intersection, Honolulu, Oahu, Hawaii," prepared for the design of the project.

Our work was performed in general accordance with the scope of services outlined in our revised fee proposal dated September 8, 2022.

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 unclear, please contact our office.

Very truly yours,

GEOLABS, INC.

Gerald Y. Seki, P.E.
Vice President

GS:AT:ZS:lf

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SUMMARY OF FINDINGS AND RECOMMENDATIONS

Our previous field exploration generally encountered a pavement structure consisting of approximately 5 inches of asphaltic concrete overlay followed by about 6 inches of Portland cement concrete. Below the pavement, fill material consisting of stiff to very stiff silty clay was encountered to a depth of approximately 6 feet. The fill layer was underlain by medium-hard to hard basalt rock formation extending to the maximum depth explored of about 26.7 feet below the existing ground surface. We did not encounter groundwater in the boring at the time of our field exploration. However, it should be noted that groundwater levels are subject to change due to rainfall, time of year, seasonal precipitation, surface water runoff, and other factors.

We recommend supporting the new traffic signal poles on single cast-in-place drilled shaft foundations. Based on the loading demands provided and anticipated subsurface soil conditions encountered, we recommend the following:

- For Traffic Signal Type I, we recommend using drilled shafts with a diameter of 24 inches and a minimum embedment length of 8 feet.

- For Traffic Signal Type II with mast arm lengths of 27 to 38 feet, we recommend using drilled shafts with a diameter of 36 or 42 inches and a minimum embedment length of 8 feet.

For both Traffic Signal Type I and Traffic Signal Type II, the drilled shaft should be embedded a minimum of 2 feet into the basalt formation to ensure adequate stability and load-bearing capacity.

It is imperative that a Geolabs representative is present at the project site to observe the drilling and installation of the drilled shafts during construction and confirm the assumed subsurface conditions.

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 conducted for the *Traffic Signal Modernization Project* at the Kalanianaole Highway and Kalaniiki Street Intersection in 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 geotechnical recommendations resulting from our field exploration, laboratory testing, and engineering analyses for the project. These findings and geotechnical recommendations are intended for the design of traffic signal pole foundations and utilities only. The findings and recommendations presented herein are subject to the limitations noted at the end of this report.

1.1 Project Considerations

The project involves the installation of eight Type I and five Type II traffic signal poles at the Kalanianaole Highway and Kalaniiki Street intersection in the Waialae Iki area of Honolulu on the Island of Oahu, Hawaii. The existing intersection is signalized in all four directions with both metal single pole and mast arm traffic signal poles. The new traffic signal poles are shown on the Site Plan, Plate 2. Based on the information provided, the mast arm lengths of the traffic signal poles range from 27 to 38 feet in length.

1.2 Purpose and Scope

The purpose of our geotechnical engineering exploration was to obtain an overview of the surface and subsurface conditions to develop an idealized soil/rock data set to formulate geotechnical engineering recommendations for the project. The work was performed in general accordance with the scope of services outlined in our revised fee proposal dated September 8, 2022. A previously performed boring near the intersection of Kalanianaole Highway and Kalaniiki Street was used in our analysis. The scope of work for this exploration included the following tasks and work efforts:

1. Preparation of this report summarizing our work on the project and presenting our findings and recommendations.

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

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

Detailed descriptions of our field exploration methodology and the Log of Boring are presented in Appendix A. Results of the laboratory tests performed on selected soil samples are presented in Appendix B. Photographs of core samples recovered from our field exploration are provided in Appendix C.

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END OF GENERAL

SECTION 2. SITE CHARACTERIZATION

2.1 Regional Geology

The Island of Oahu was built by the extrusion of basaltic lava from the Waianae and Koolau shield volcanoes. The older Waianae Volcano is estimated to be middle to late Pliocene in age, and the younger Koolau Volcano is estimated to be late Pliocene to early Pleistocene in age. After a long period of volcanic inactivity, during which time erosion incised deep valleys into the Koolau shield, volcanic activity returned with a series of lava flows followed by cinder and tuff cone formations. These series are referred to as the Honolulu Volcanic Series. The project site is at the southwestern flank of the Koolau Mountain Range.

During the Pleistocene Epoch (Ice Age), sea levels fluctuated in response to the cycles of continental glaciation. As the glaciers grew and advanced, less water was available to fill the oceanic basins such that sea levels fell below the present stands of the sea. When the glaciers melted and receded, an excess of water became available such that the sea levels rose to elevations above the present sea level.

The processes of erosion and deposition were affected by these glacio-eustatic sea level fluctuations. When the sea level was low, the erosional base level was correspondingly lower, and valleys were carved to depths below the present sea level. When the sea level was high, the erosional base level was raised such that sediments accumulated at higher elevations.

In the mountainous regions of Hawaii and in the heads of valleys, erosional processes are dominated by detachment of soil and rock masses from the valley walls and are transported downslope toward the axis of a valley primarily by gravity as colluvium. Once these materials reach the stream in the central portion of a valley, alluvial processes become dominant, and the sediments are transported and deposited as alluvium.

The project site is near the mouth of Kapakahi Valley, which trends roughly north to south from the Koolau Mountain Range toward the Pacific Ocean. Kapakahi Valley is essentially a deep erosional valley carved into the Koolau Shield Volcano by stream processes and mass wasting of the adjacent slopes. As a result, the project site is generally underlain by colluvial and alluvial deposits, followed by Koolau basalt formation. In addition, some fills were placed at portions of the site as a result of the original roadway construction. The fill materials are believed to resemble the native colluvial and alluvial deposits in character.

2.2 Site Description

The project site is located at the intersection of Kalanianaole Highway and Kalaniiki Street in the Waialae Iki area of Honolulu on the Island of Oahu, Hawaii. The intersection is generally bounded by Kalani High School to the northeast and residential homes to the south and northwest.

Based on our field observations, the project site was observed to be relatively flat with a gentle slope in the eastbound direction of Kalanianaole Highway. Based on the provided project drawings, the existing ground surface elevations of the intersection range from about +16 to +19 feet Mean Sea Level (MSL) with a slope gradient of about 1 percent. At the intersection of Kalanianaole Highway and Kalaniiki Street, Kalanianaole Highway generally consists of three lanes of traffic in each direction, with additional left turn-only lanes onto Kalaniiki Street and Waieli Street in either direction. Kalaniiki and Waieli Streets generally consist of three traffic and turn lanes at the intersection.

Based on the information provided, we understand that eight Traffic Signal Pole Type I and five Traffic Signal Pole Type II are planned to be installed at the project site. The layout of the intersection and proposed traffic signal replacement location are presented on the Site Plan, Plate 2.

Based on additional information provided by the State of Hawaii Department of Transportation, we understand that a layer of rockfill was placed under the pavement near the intersection at various times between the years 1934 and 1952.

2.3 Subsurface Conditions

The subsurface conditions described herein are based on our previous report, entitled "Geotechnical Engineering Exploration, Traffic Signal Modernization Project, Kalanianaole Highway and Kalaniiki Street Intersection, Honolulu, Oahu, Hawaii," dated August 12, 2019.

We explored the subsurface conditions at the project site by drilling and sampling one boring, designated as Boring No. 2, to a depth of about 26.7 feet below the existing ground surface. The approximate boring location is shown on the Site Plan, Plate 2.

Our boring generally encountered a pavement structure consisting of approximately 5 inches of asphaltic concrete overlay followed by about 6 inches of Portland cement concrete. Below the pavement, fill material consisting of stiff to very stiff silty clay was encountered at a depth of approximately 6 feet, underlain by medium hard to hard basalt rock formation extending to the maximum depth explored of about 26.7 feet below the existing ground surface.

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

Detailed descriptions of the field exploration methodology are presented in Appendix A. Descriptions and graphic representations of the materials encountered in the boring are presented on the Log of Boring in Appendix A. Results of the laboratory tests performed on selected soil samples are presented in Appendix B. Photographs of core samples recovered from our field exploration are provided in Appendix C.

END OF SITE CHARACTERIZATION

SECTION 3. DISCUSSION AND RECOMMENDATIONS

Our boring generally encountered a pavement structure consisting of approximately 5 inches of asphaltic concrete overlay followed by about 6 inches of Portland cement concrete. Below the pavement, fill material consisting of stiff to very stiff silty clay was encountered at a depth of approximately 6 feet, underlain by medium-hard to hard basalt rock formation extending to the maximum depth explored of about 26.7 feet below the existing ground surface. We did not encounter groundwater in the boring drilled at the time of our field exploration.

We recommend supporting the new traffic signal poles on single cast-in-place drilled shaft foundations. Based on the loading demands provided and anticipated subsurface soil conditions encountered, we recommend the following:

- For Traffic Signal Type I, we recommend using drilled shafts with a diameter of 24 inches and a minimum embedment length of 8 feet.
- For Traffic Signal Type II with mast arm lengths of 27 to 38 feet, we recommend using drilled shafts with a diameter of 36 or 42 inches and a minimum embedment length of 8 feet.

For both Traffic Signal Types I and II poles, the drilled shaft should be embedded a minimum of 2 feet into the basalt formation to ensure adequate stability and load-bearing capacity.

Detailed discussions and recommendations for the design of pole foundations, utility trenches, and other geotechnical aspects of the project are presented in the following sections.

3.1 Traffic Signal Pole Foundations

Based on the information provided, we understand that new traffic signal poles with mast arm lengths of up to 38 feet are planned to replace the existing traffic signal poles at the Kalanianaole Highway and Kalaniiki Street intersection. Based on the typical loading demands and anticipated subsurface soil conditions, we recommend supporting the new traffic signal poles on single cast-in-place drilled shaft foundations.

We understand that the new traffic signal poles will be supported on a single-drilled shaft foundation. The following structural loads for the new Traffic Signal Types I and II poles with 27-foot and 38-foot mast arms were provided by Engineering Concepts, Inc. The provided structural loads are summarized in the table below.

TRAFFIC SIGNAL POLES STRUCTURAL INFORMATION SUMMARY				
Structure	**Axial Load** (kips)	**Lateral Load** (kips)	**Moment Induced** (kip-feet)	**Torsion** (kip-feet)
Type I Pole	0.49	1.87	16.3	N/A
Type II Pole with 27-foot mast arm	21.37	3.30	59.29	30.13
Type II Pole with 38-foot mast arm	22.29	4.83	70.76	46.41

The cast-in-place concrete drilled shafts would derive vertical support primarily from friction between the concrete shaft and the surrounding soils. In general, the end-bearing component of the drilled shafts has been discounted in our analysis due to difficulties associated with obtaining a clean bottom during construction in the relatively deep drilled shaft.

It is anticipated that the poles would be supported on a single-drilled shaft designed to support the above structural loads. Based on the structural loads provided and the results of our axial and lateral analyses, we recommend the following:

- For Traffic Signal Type I, we recommend using drilled shafts with a diameter of 24 inches and a minimum embedment length of 8 feet.
- For Traffic Signal Type II with mast arm lengths of 27 to 38 feet, we recommend using drilled shafts with a diameter of 36 or 42 inches and a minimum embedment length of 8 feet.

For both Traffic Signal Types I and II, the drilled shaft should be embedded a minimum of 2 feet into the basalt formation to ensure adequate stability and load-bearing capacity.

Based on our evaluation of the subsurface conditions and the foundation design parameters, we anticipate that 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

3.1.1 Lateral Load Resistance

In general, the 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 the induced moment in the shaft. The lateral load analyses were performed using the program LPILE v2022 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 cast-in-place concrete drilled shaft was modeled using a 28-day concrete strength of 4,000 psi. Vertical reinforcement was assumed to be 1 percent of the total cross-sectional area. The lateral deflection at the top of the shaft, the maximum induced moment, and the maximum induced shear of the drilled shaft are presented in the table below.

TRAFFIC SIGNAL POLES
DRILLED SHAFT ANALYSIS SUMMARY

Structure	Shaft Diameter (inches)	Drilled Shaft Length (feet)	Lateral Deflection (inches)	Maximum Induced Moment (kip-feet)	Depth to Maximum Moment (feet)
Type I Pole	24	8	0.018	25.27	6.0
Type II Pole with 27-foot mast arm	36	8	0.024	75.65	6.0
Type II Pole with 38-foot mast arm	36	8	0.030	96.08	6.0
Type II Pole with 27-foot mast arm	42	8	0.020	75.32	6.0

TRAFFIC SIGNAL POLES DRILLED SHAFT ANALYSIS SUMMARY					
Structure	**Shaft Diameter** (inches)	**Drilled Shaft Length** (feet)	**Lateral Deflection** (inches)	**Maximum Induced Moment** (kip-feet)	**Depth to Maximum Moment** (feet)
Type II Pole with 38-foot mast arm	42	8	0.026	95.73	6.0
NOTE: Analyses based on concrete compressive strength of 4,000 psi and a minimum of 1% longitudinal steel reinforcement. The top 3 feet of soil are excluded from analyses.					

3.1.2 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 underlying soil. Total settlements of the drilled shafts are estimated to be on the order of about 0.5 inches. We believe a significant portion of the settlement is elastic and should occur as the loads are applied.

3.1.3 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 surrounding soils and/or formation. Therefore, proper construction techniques, especially during drilling operations, are important. The contractor should exercise care in drilling the shaft holes and placing concrete into the drilled holes.

Based on the anticipated subsurface conditions described above, some of the geotechnical considerations associated with drilled shaft foundations are discussed below.

3.1.3.a Installation in Granular Material

Drilled shaft foundations are highly effective in soil and/or rock formations that will stay open after drilling until concrete placement. Unfortunately, materials such as the granular rock fill layer expected at some areas within the project site may collapse following the drilling if it remains unsupported. Therefore, a partial-depth temporary steel casing will likely be necessary to maintain the integrity of the drilled hole during the drilled shaft installation. This condition would increase the construction difficulty and costs of the foundations. However, this condition is common in Hawaii and has been addressed by local drilled shaft subcontractors for structures such as high-rise buildings and bridges.

Care should be exercised during the removal of the temporary casing to reduce the potential for "necking" of the drilled shaft. Therefore, a minimum 5-foot head of concrete above the bottom of the casing or adequate concrete head should be maintained during the removal of the casing.

3.1.3.b Obstructions, Boulders, and Basalt Formation

Where obstructions, boulders, and/or hard basalt formation are anticipated, 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 or man-made obstructions 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 shaft to the recommended depth and dimension.

3.1.3.c Concrete Placement

A low-shrink concrete mix with a high slump (6 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 substantial completion 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 three 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 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 the placement of concrete. Therefore, Geolabs' observation of the drilled shaft installation operations is necessary to confirm the assumed subsurface conditions.

3.2 Traffic Signal Pull Box Foundations

We understand that the traffic signal pull boxes will be embedded below the existing ground surface. Based on the encountered subsurface conditions, an allowable bearing pressure of up to 1,500 pounds per square foot (psf) may be utilized for the design of the traffic signal pull box structures bearing on the near-surface fill at the project site. To provide uniform bearing support for the new pull box structure, we recommend providing a minimum 6-inch-thick layer of No. 3 Fine gravel (ASTM C33, No. 67 gradation) below the bottom of the pull box structures. We anticipate the traffic signal pull box will be a pre-cast concrete structure.

The subgrades should be scarified to a depth of at least 8 inches, if possible, moisture-conditioned to at least 2 percent above the optimum water content, and recompacted to at least 90 percent relative compaction to provide a relatively firm and smooth bearing surface prior to the placement of reinforcing steel and/or concrete. Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same soil established in accordance with ASTM D1557. Optimum moisture is the water content (percentage by dry weight) corresponding to the maximum dry density.

Soft and/or loose materials encountered at the bottom of the footing excavations should be over-excavated to expose the underlying firm materials. The over-excavation may be backfilled with general fill materials compacted to a minimum of 90 percent relative compaction, or the bottom of the footing may be extended down to bear directly on the underlying competent materials.

3.2.1 Lateral Earth Pressures

The lateral earth pressures acting on the proposed pull box structure will depend on the type of backfill used, the extent of backfill, and the compactive effort on the backfill material around the structure. We recommend designing the new pull box structure to resist the following lateral earth pressures (at-rest condition) from the adjacent soils.

LATERAL EARTH PRESSURES FOR PULL BOX STRUCTURE		
Subsoil Conditions	**At-Rest** (pcf)	**Passive** (pcf)
Level Backfill	50	340

The values provided above assume that the excavated on-site soils will be used to backfill behind and/or around the structure. The backfill behind and/or around the utility structures should be compacted to between 90 and 95 percent relative compaction per ASTM D1557. Over-compaction of the structure backfill should be avoided.

Surcharge stresses due to areal surcharges, traffic loads, line loads, and point loads within a horizontal distance equal to the depth of the structure should be considered in the design. For uniform surcharge stresses imposed on the loaded side of the pull box structure, a rectangular distribution with a uniform pressure equal to 50 percent of the vertical surcharge pressure acting over the entire depth of the structure may be used in the design. Additional analyses during the design may be needed to evaluate the surcharge effects of point loads and line loads.

Lateral loads acting on the structures may be resisted by friction developed between the bottom of the structures and the supporting subgrade soils and passive earth pressure developed against the embedded near-vertical faces of the structures system. A coefficient of friction of 0.4 may be used between the base of the structure and the granular bedding material to resist lateral loads. Based on our field exploration data and laboratory test results, the recommended passive earth pressure shown in the above table may be used in the design.

3.2.2 Fills and Backfills

The traffic signal pull box structure excavation will need to be properly backfilled to reduce the potential for subsidence at the ground surface. The excavated on-site soils or imported select granular fill materials that are free of vegetation, deleterious materials, and clay lumps and rock fragments greater than 3 inches in maximum dimension may be used as backfill up to the finished subgrades.

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

3.2.3 Fill Placement and Compaction Requirements

Pull box structure 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. If the pull box structure is located below pavement areas, the upper 3 feet of the structure backfill below the pavement grade should be 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 as determined by ASTM D1557. Optimum moisture is the water content (percentage by dry weight) corresponding to the maximum dry density.

3.3 Utility Trench

We anticipate that underground utilities, such as new electrical lines, may be installed for the project. In general, good construction practices should be utilized for the installation and backfilling of the trenches for the new utilities. The contractor should determine the method and equipment to be used for trench excavation, subject to practical limits and safety considerations. In addition, the excavations should comply with the applicable federal, state, and local safety requirements. The contractor should be responsible for trench shoring design and installation.

In general, we recommend providing granular bedding consisting of 6 inches of open-graded gravel (ASTM C33, No. 67 gradation) under the pipes for uniform support. Free-draining granular materials, such as open-graded gravel (ASTM C33, No. 67 gradation), should also be used for the initial trench backfill up to about 12 inches above the pipes to provide adequate support around the pipes. It is critical to use this free-draining material to reduce the potential for the formation of voids below the haunches of pipes and to provide adequate support for the sides of the pipes. Improper trench backfill could result in backfill settlement and pipe damage.

The upper portion of the trench backfill from the level 12 inches above the pipes to the top of the subgrade or finished grade may consist of select granular fill material. The backfill material should be moisture-conditioned to above the optimum moisture content, placed in maximum 8-inch level loose lifts, and mechanically compacted to at least 90 percent relative compaction. In areas where trenches will be in paved areas, the upper 3 feet of the trench backfill below the pavement finished grade should be compacted to no less than 95 percent relative compaction. Mechanical compaction equipment should be used to compact the backfill materials. Compaction efforts by water tamping, jetting, or ponding should not be allowed.

3.4 Design Review

Preliminary and final drawings and specifications for the project should be forwarded to Geolabs for review and written comments prior to bid solicitation for construction. This review is necessary to evaluate the conformance of the plans and specifications with the intent of the foundation and utility trench recommendations provided herein. If this review is not made, Geolabs cannot be responsible for misinterpretation of our recommendations.

3.5 Post-Design Services/Services During Construction

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

1. Observation of the drilled shaft foundation installation
2. Observation of utility trench excavation and compaction

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

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

END OF DISCUSSION AND RECOMMENDATIONS

SECTION 4. LIMITATIONS

The analyses and recommendations submitted herein are based, in part, upon information obtained from our boring. Variations of the subsurface conditions beyond the boring 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 boring location indicated herein is approximate, having been taped from visible features shown on the Signal Plan transmitted by Engineering Concepts, Inc. on January 31, 2019. The elevation of the boring was interpolated from the contour lines and spot elevations shown on the same plan. The field boring location and elevation should be considered accurate only to the degree implied by the methods used.

The stratification breaks represented on the Log of Boring show the approximate boundaries between soil types and, as such, may denote a gradual transition. Water level data from the boring were measured at the times shown on the graphic representations and/or presented in the text of this report. The data has been reviewed and interpretations made in the formulation of this report. However, it must be noted that fluctuation may occur due to variations in seasonal rainfall and other factors.

This report has been prepared for the exclusive use of Engineering Concepts, Inc., and their consultants for specific application to the Kalanianaole Highway and Kalaniiki Street Intersection for the Traffic Signal Modernization 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 client/owner in the design of the traffic signal pole foundations for the project. Therefore, this report may not contain sufficient data or the proper information to serve as the basis for construction cost estimates or for bidding purposes. A contractor wishing to bid on this project should 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 subsurface conditions, such as perched groundwater, soft deposits, or hard layers may occur in localized areas and may require additional corrections in the field (which may result in construction delays) to attain a properly constructed project. Therefore, a sufficient contingency fund is recommended to accommodate these possible extra costs.

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

END OF LIMITATIONS

CLOSURE

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

-ΩΩΩΩΩΩΩΩΩ-

Respectfully submitted,

GEOLABS, INC.

By ______________________

Gerald Y. Seki, P.E.
Vice President

GS:AT:ZS:lf

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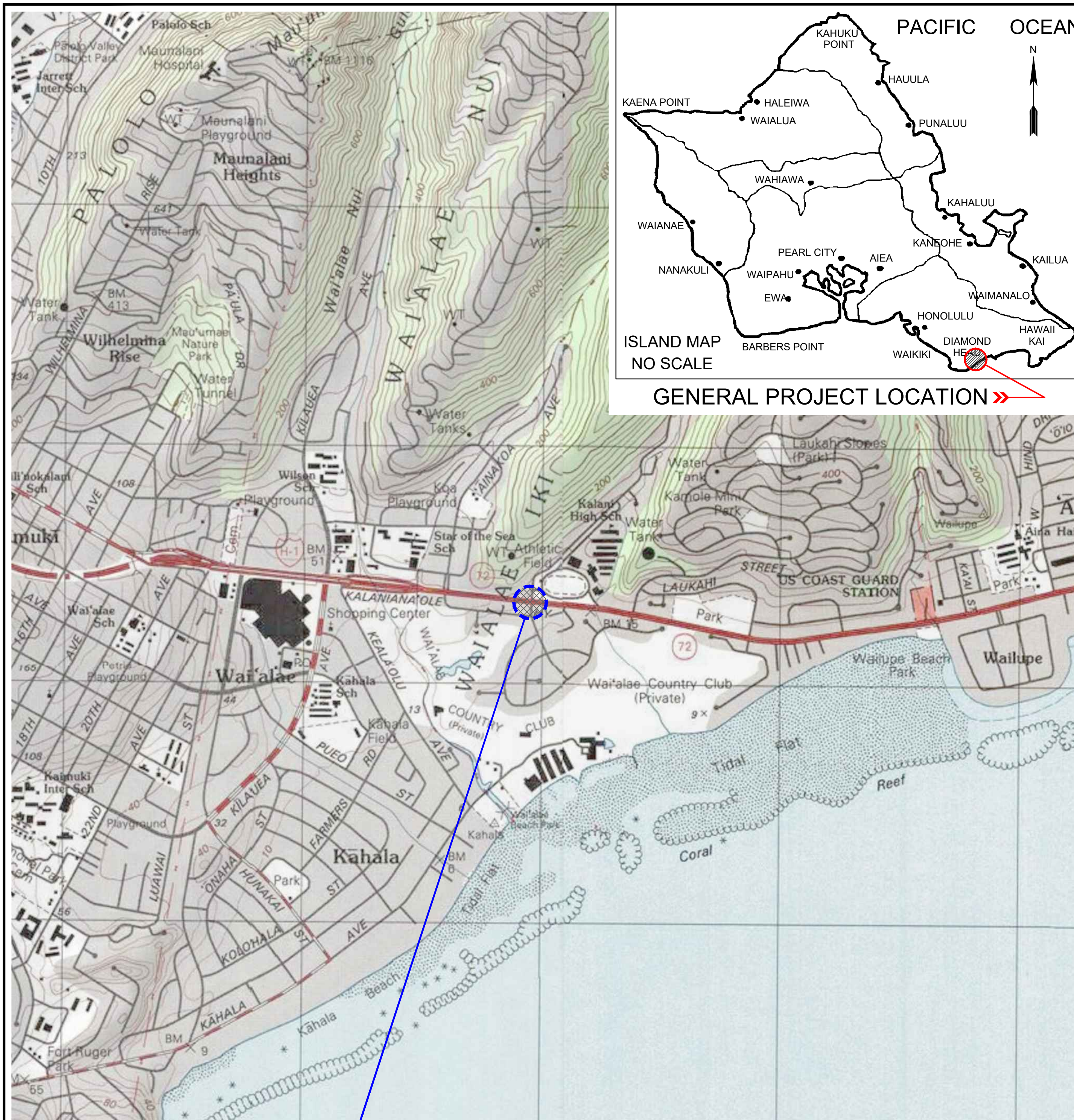
PLATES



PROJECT LOCATION »

PROJECT LOCATION MAP

TRAFFIC SIGNAL MODERNIZATION PROJECT
KALANIANAOLE HIGHWAY &
KALANIIKI STREET INTERSECTION
HONOLULU, OAHU, HAWAII

2000 1000 0 2000 FT.

GRAPHIC SCALE

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

GEOLABS, INC.

Geotechnical Engineering

DATE	DRAWN BY	PLATE
MAY 2024	ASP	
SCALE	W.O.	1
1" = 2,000'	7328-20(B)	

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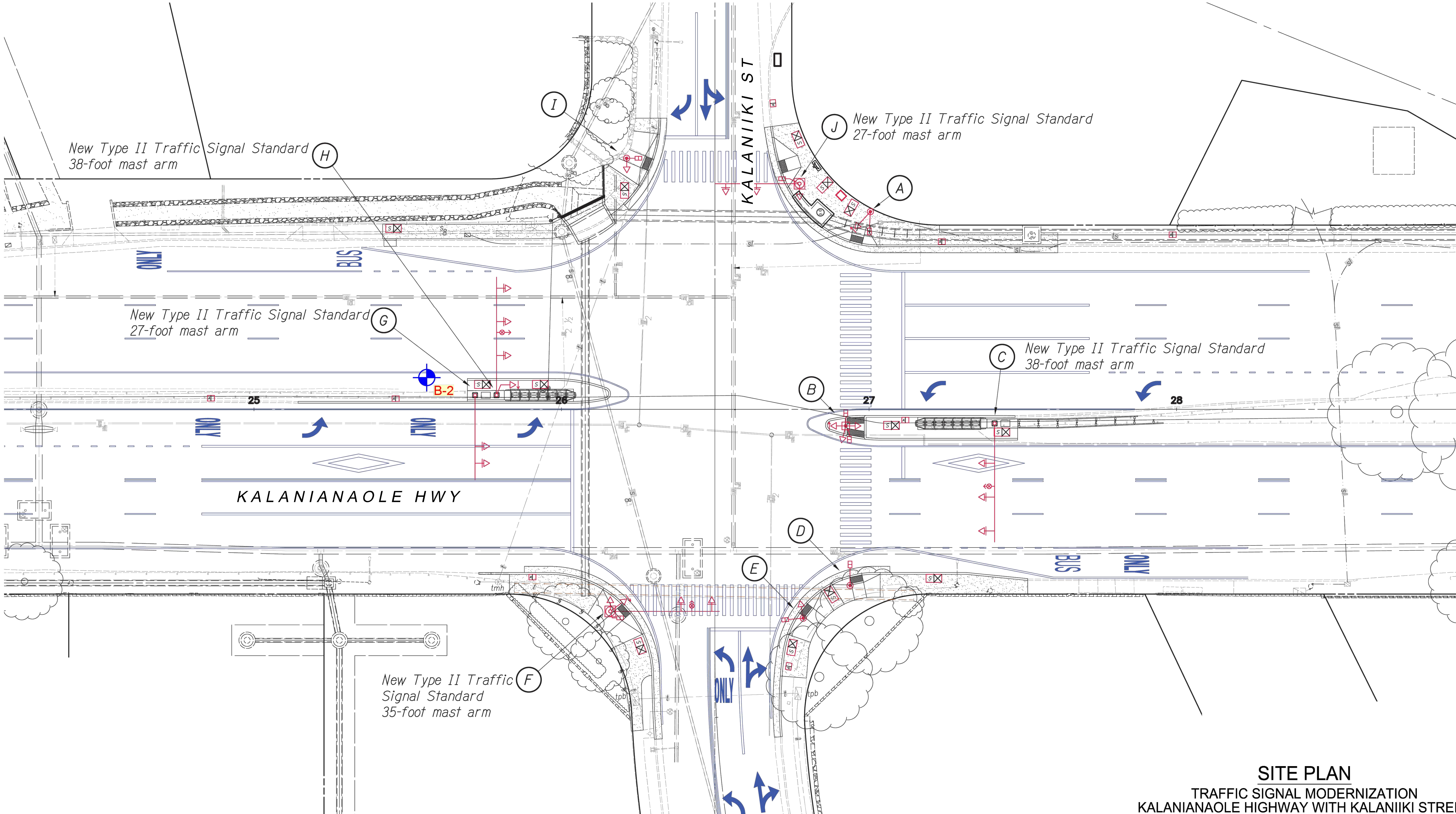

SITE PLAN

TRAFFIC SIGNAL MODERNIZATION
KALANIANAOLE HIGHWAY WITH KALANIIKI STREET
ISLAND OF OAHU, HAWAII

LEGEND:

APPROXIMATE BORING LOCATION (FROM REPORT BY GEOLABS, INC. ENTITLED "GEOTECHNICAL ENGINEERING EXPLORATION, TRAFFIC SIGNAL MODERNIZATION PROJECT, KALANIANAOLE HIGHWAY AND KALANIIKI STREET INTERSECTION, HONOLULU, OAHU, HAWAII" DATED AUGUST 12, 2019. W.O. 7328-00(C)

REFERENCE: SIGNAL PLAN TRANSMITTED BY ENGINEERING CONCEPTS, INC. ON MAY 20, 2024.

0 10 20 30 60 90 FT.
GRAPHIC SCALE

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GEOLABS, INC.
Geotechnical Engineering

DATE	DRAWN BY	PLATE
JUNE 2024	ASP	
SCALE	W.O.	
1" = 30'	7328-20(B)	2

CAD User: ASPASIONJR File Last Updated: May 31, 2024 4:16:20pm Plot Date: May 31, 2024 - 4:28:57pm
File: A:\Drafting\DraftingWorking\7328-20(B)_Traffic_Signal_Modernization\7328-20(B)SitePlan.dwg\2.0 SitePlan
Plotter: DWG To PDF-GEO.pc3 Plotstyle: GEO-No-Dithering-Blue-Boring.ctb

APPENDIX A

APPENDIX A

Field Exploration

We explored the subsurface conditions at the project site by drilling and sampling one boring, designated as Boring No. 2, extending to a depth of about 26.7 feet below the existing ground surface. The approximate boring location is shown on the Site Plan, Plate 2. The boring was drilled using a truck-mounted drill rig equipped with continuous flight augers and rotary coring tools.

Our geologist classified the materials encountered in the boring 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 Log of Boring, Plate A-1.

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 Log of Boring at the appropriate sample depths. The penetration resistance shown on the Log of Boring indicates the number of blows required for the specific sampler type used. The blow counts may need to be factored to obtain the Standard Penetration Test (SPT) blow counts.

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

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

Plate

A-0.1

Soil Classification Log Key
(with deviations from ASTM D2488)

GEOLABS, INC. CLASSIFICATION*

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

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

RELATIVE DENSITY / CONSISTENCY

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

MOISTURE CONTENT DEFINITIONS

Dry: Absence of moisture, dry to the touch

Moist: Damp but no visible water

Wet: Visible free water

ABBREVIATIONS

WOH: Weight of Hammer

WOR: Weight of Drill Rods

SPT: Standard Penetration Test Split-Spoon Sampler

MCS: Modified California Sampler

PP: Pocket Penetrometer

GRAIN SIZE DEFINITION

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

*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 7328-00(C).GPJ GEOLABS.GDT 8/1/19

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

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Laboratory			Field								Approximate Ground Surface Elevation (feet MSL): 19.5 *
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)	Depth (feet)	Sample	Graphic	USCS	Description
											5-inch **ASPHALTIC CONCRETE**
										CH	6-inch **CONCRETE**
UC	25	87			38	4.5					Brown **SILTY CLAY** with some sand and gravel, stiff to very stiff, moist (fill)
LL=66 PI=46	20				11						
	30				55/4"		5				
UC			63	63							Gray to reddish gray vesicular **BASALT**, severely to moderately fractured, moderately to highly weathered, medium hard to hard (pahoehoe basalt)
							10				
					8						grades with seams of weathered clinker
			100	43							
							15				
	28				72						
UC			100	29							
							20				
			100	52							
							25				
					30/2"						
											Boring terminated at 26.67 feet * Elevation estimated from Signal Plan transmitted by Engineering Concepts, Inc. on January 31, 2019.
							30				
							35				

Date Started:	May 10, 2019	Water Level:	not encountered	Plate A - 1
Date Completed:	May 10, 2019			
Logged By:	D. Gremminger	Drill Rig:	CME-45C TRUCK	
Total Depth:	26.67 feet	Drilling Method:	4" Solid Stem Auger & PQ Coring	
Work Order:	7328-20(B)	Driving Energy:	140 lb. wt., 30 in. drop	

APPENDIX B

A P P E N D I X B

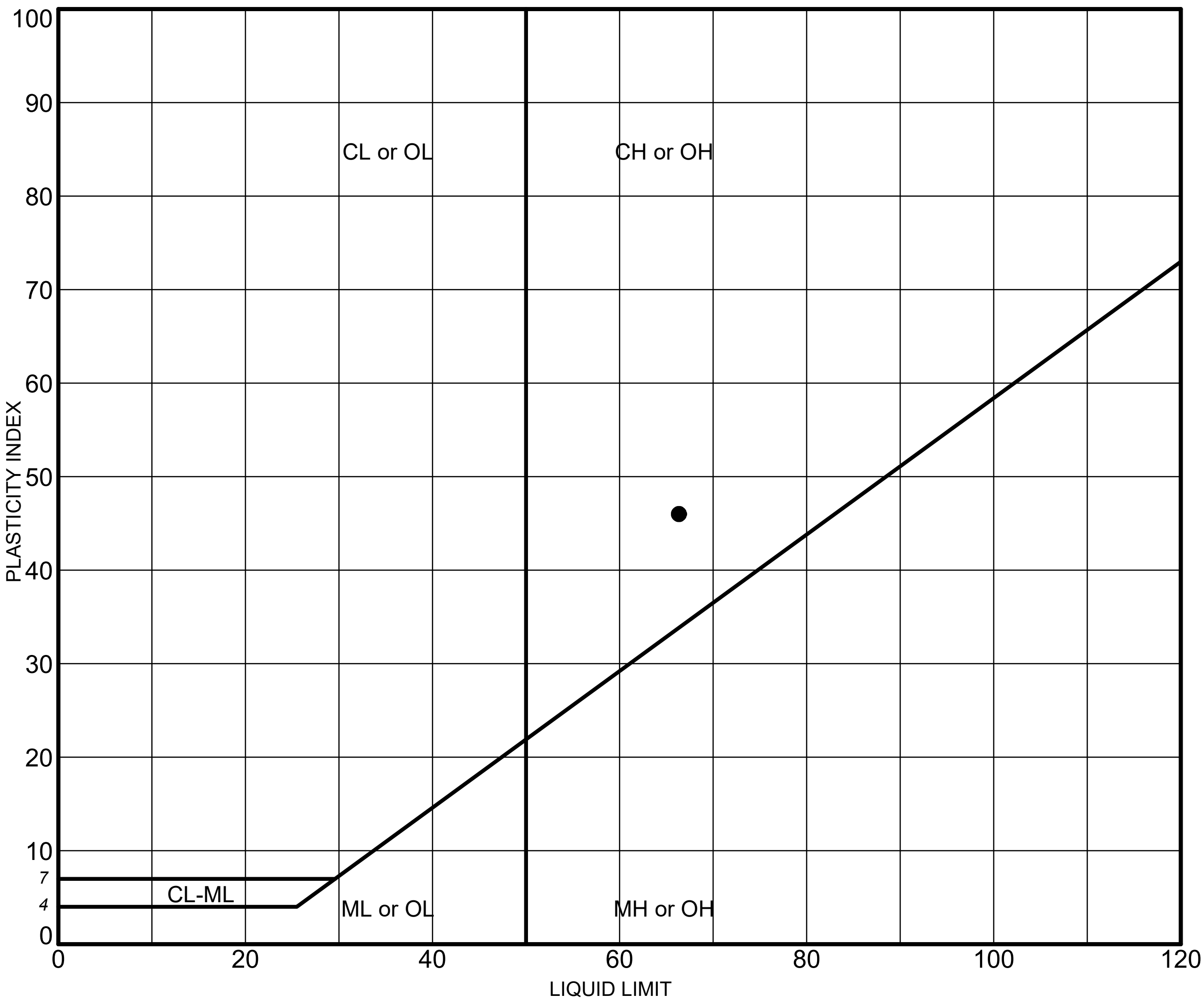
Laboratory Tests

Moisture Content (ASTM D2216) and Unit Weight (ASTM D2937) determinations were performed on selected 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.

One Atterberg Limits test (ASTM D4318) was performed on a selected soil sample to evaluate the liquid and plastic limits and to aid in soil classification. The test results are summarized on the Log of Boring at the appropriate sample depth. A graphic presentation of the test results is provided on Plate B-1.

One Unconfined Compression test (ASTM D2166) was performed on a selected in-situ cohesive soil sample to evaluate the unconfined compressive strength of the soil. The test results are provided on Plate 2.

Two Uniaxial Compression Strength tests (ASTM D7012 Method C) were performed on selected rock cores to evaluate the unconfined compressive strength of the rock formation encountered. Results of the uniaxial compression tests are presented on Plate B-3.



	Sample	Depth (ft)	LL	PL	PI	Description
●	B-2	2.5-4.0	66	20	46	Brown silty clay (CH) with some sand and gravel

NP = NON-PLASTIC

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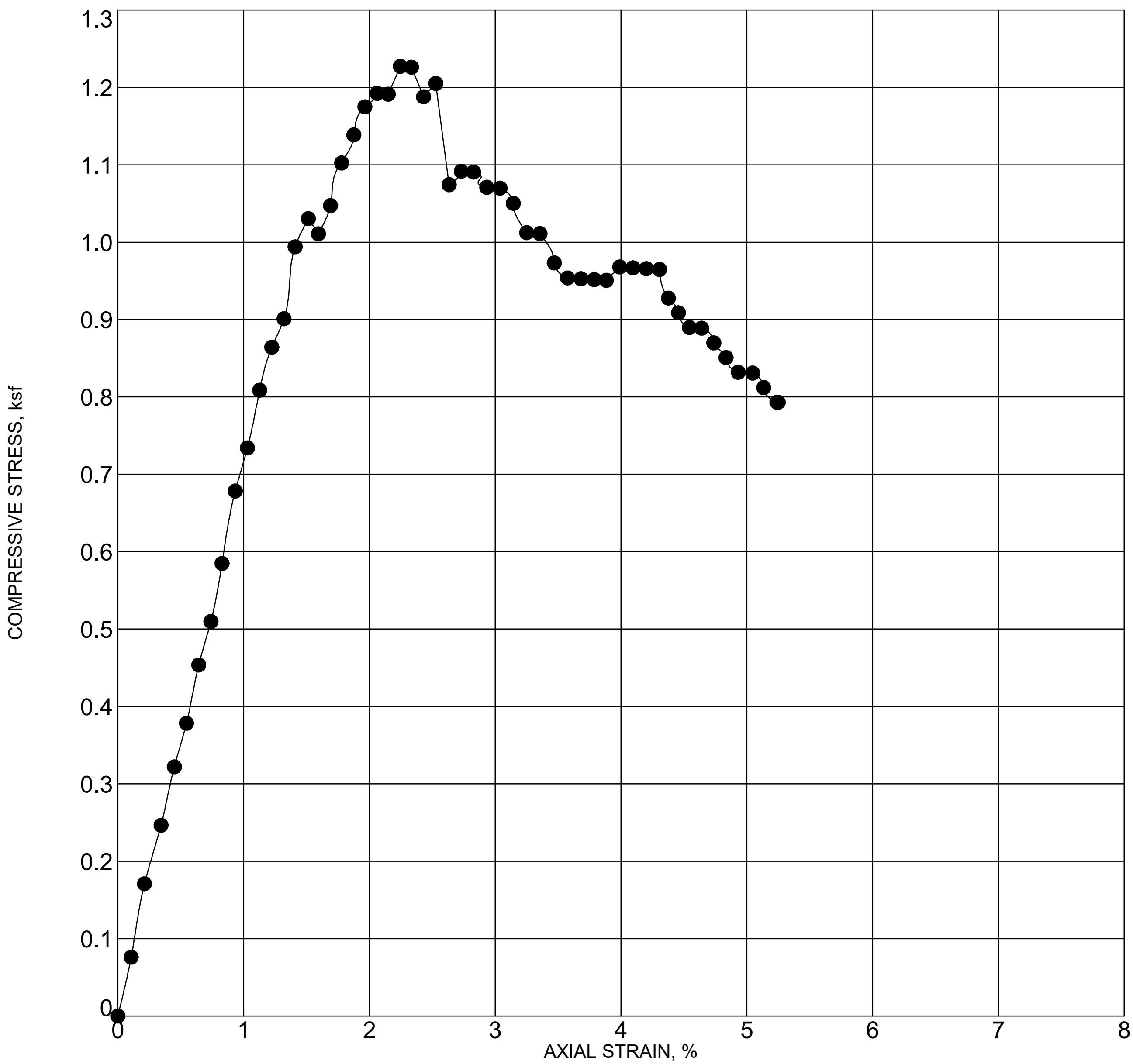
ATTERBERG LIMITS TEST RESULTS - ASTM D4318

TRAFFIC SIGNAL MODERNIZATION PROJECT
KALANIANAOLE HIGHWAY &
KALANIIKI STREET INTERSECTION
HONOLULU, OAHU, HAWAII

Plate
B - 1



Unconfined Compressive Strength (ksf):	1.23
Axial Strain at Failure (%):	2.2
Strain Rate (% / minute):	0.94

Location: B-2
Depth: 1.0 - 2.5 feet
Description: Brown silty clay with some sand and gravel
Test Date: 6/3/2019

Dry Density (pcf)	86.7	Sample Diameter (inches)	2.390
Moisture (%)	24.9	Sample Height (inches)	5.000

G UC 7328-00(C).GPJ GEOLABS.GDT 8/12/19

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UNCONFINED COMPRESSION TEST - ASTM D2166

TRAFFIC SIGNAL MODERNIZATION PROJECT
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KALANIIKI STREET INTERSECTION
HONOLULU, OAHU, HAWAII

Plate
B - 2

Location	Depth	Length	Diameter	Length/ Diameter Ratio	Density	Load	Compressive Strength
	(feet)	(inches)	(inches)		(pcf)	(lbs)	(psi)
B-2	6 - 11	6.900	3.300	2.09	141.6	35,870	4,190
B-2	17.5 - 21	6.900	3.300	2.09	111.0	23,410	2,740

ASTM D7012 (METHOD C)

APPENDIX C

TRAFFIC SIGNAL MODERNIZATION PROJECT
KALANIANAOLE HIGHWAY & KALANIIKI STREET INTERSECTION
HONOLULU, OAHU, HAWAII

B-2 6.0' TO 26.0'

6.0' 12.5' 17.5' 21.0'

11.0' 16.0' 21.0' 26.0'