GEOTECHNICAL ENGINEERING EXPLORATION
INTERSTATE ROUTE H-1 (EB) IMPROVEMENTS
OLA LANE OVERPASS TO KALIHI STREET INTERCHANGE
HONOLULU, OAHU, HAWAII
W.O. 8049-00 & 10(B) NOVEMBER 21, 2022

SUMMARY OF FINDINGS AND RECOMMENDATIONS

Our field exploration at the project site generally encountered a surface fill layer about 1 to 17 feet thick underlain by saprolite or recent alluvium followed by a basalt rock formation extending to depths of about 10 to 67 feet below the existing ground surface. Below the basalt rock formation, older alluvial deposits were encountered extending to the maximum depth explored of about 122.5 feet below the existing ground surface.

We encountered groundwater in one of our borings at depths of about 19.5 and 29.6 feet below the existing ground surface, corresponding to elevations of about +21.4 and +31.5 feet MSL. It should be noted that due to the depths of the borings and the basalt formation encountered, rotary rock coring techniques utilizing drilling water were required to advance the deeper borings. Therefore, accurate in-situ groundwater readings could not be taken in most of our borings due to the time it took for the drilling water to dissipate. In addition, it should be noted that groundwater levels can fluctuate depending on tidal fluctuations, storm surge conditions, seasonal precipitation, and other factors.

Based on the information provided, we understand that new abutments will need to be constructed for the Gulick Avenue Overpass to widen Interstate Route H-1 in the eastbound direction and the westbound direction in the future. Due to the limited construction area available, the installation of drilled shaft retaining walls for the northern and southern abutments is planned for the overpass. Drilled shaft retaining walls typically consist of a series of closely spaced drilled shafts along the retaining wall alignment.

To evaluate the lateral load resistance of the new bridge retaining structures, stiffness modeling parameters were estimated based on the subsurface conditions encountered in the drilled borings. The analysis was carried out to generate non-linear “p-y” curves to represent soil moduli at various depths to be used in the structural model. In addition, lateral load analysis of the drilled shafts was performed using the LPILE software.

The center pier structure supported on continuous spread footing for the Gulick Avenue Overpass will be subjected to additional loading due to the bridge widening. Based on our field exploration and the structural loading, we believe the existing center pier foundation is adequate for the additional structural loading. Due to the high foundation bearing pressures and the potential for cavities and/or voids in the underlying basalt rock formation, we recommend the implementation of a probing and grouting program on the sides of the existing pier footing.

Artesian groundwater conditions may be present at the project site within the basalt rock formation and/or older alluvium layers. Therefore, additional measures may be required to maintain the integrity of all cast-in-place concrete structures below the groundwater elevations because artesian groundwater pressures may cause the cement matrix to be washed away considering the high slump concrete used and the retarding admixtures that are normally introduced into the drilled shaft concrete.

We understand that deep foundations are desired to support the Kalihi Stream Bridge expansion on both sides of Kalihi Stream. To develop the required bearing and lateral load resistances, the proposed bridge expansion may be supported by a foundation system consisting of 60-inch diameter cast-in-place concrete drilled shafts.

We understand that spread footings are desired on the west side of Richard Lane to support the Kalihi Stream Bridge widening improvements. Based on the available as-built drawings, we understand that the existing Kalihi Stream Bridge is supported by shallow foundations bearing on the underlying basalt rock formation. Our boring on Richard Lane encountered basalt rock formation at a depth of about 5 feet below the existing pavement surface. Therefore, we believe that shallow spread and/or continuous footings bearing on the underlying basalt rock formation may be utilized for foundation support of the proposed bridge widening improvements.

Based on the information provided, we understand that both cut and fill walls are required on the makai side of Interstate Route H-1 for the freeway widening project. The majority of the new retaining walls will be in cut conditions with the exception of the retaining walls near the Kalihi Stream/Richard Lane Bridge where the existing ground slopes downward and will be in a fill condition. Recommendations regarding the design of these retaining structures are provided in the body of this report.

Since the new foundation installations will be performed in the underlying basalt rock formation that will require hard excavation, we recommend performing a pre-construction condition survey to document the existing conditions before the start of construction. In addition, we recommend implementing an instrumentation and monitoring program for excavation movement. Furthermore, vibration monitoring should be performed during construction.

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 performed for the *Interstate Route H-1 (EB) Improvements, Ola Lane Overpass to Kalihi Street Interchange* project located 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 retaining structures, bridge foundations, site grading, and underground utilities. The findings and recommendations presented herein are subject to the limitations noted at the end of this report.

1.1 Project Considerations

The congestion improvements project spans approximately 3,100 linear feet along Interstate Route H-1, starting from the Ola Lane Overpass to the Kalihi Street Interchange. Generally, the project involves widening the highway in the eastbound direction through this section. Widening the highway will affect two significant structures: the Kalihi Stream/Richard Lane Bridge and the Gulick Avenue Overpass.

The congestion improvements project generally involves widening the Kalihi Stream/Richard Lane crossing and lengthening the Gulick Avenue Overpass. In addition, new retaining walls will be constructed to accommodate the highway widening on the southern (makai) side of Interstate Route H-1. The installation of new highway pavements will also be required as part of the project.

The Kalihi Stream/Richard Lane bridge structure consists of a one-span bridge structure about 86 feet in length crossing Kalihi Stream with a back span structure about 48 feet long crossing Richard Lane. The existing Kalihi Stream/Richard Lane bridge structure is supported on shallow continuous footing foundations.

The Gulick Avenue Overpass structure consists of a two-span structure about 56-feet wide and 116.5 feet in total length with spans of about 58.23 feet long. Based on as-built drawings, the abutments and center pier structures are supported on shallow continuous footings bearing on the underlying basalt rock formation.

The Kalihi Stream/Richard Lane bridge will be widened about 11.5 feet on the makai side of the bridge. Lengthening of the existing abutment structures will be required to accommodate the widening.

The Gulick Avenue Overpass structure will be lengthened about 18 feet on the makai and mauka sides of the existing bridge and provide about 18 feet of additional width on Interstate Route H-1 Freeway on the makai side of the overpass and for the future widening on the mauka side of the overpass. The new abutments will consist of a drilled shaft retaining wall structures with a concrete facing. The existing abutment structure will be demolished.

1.2 Purpose and Scope

The purpose of our field exploration was to obtain an overview of the subsurface conditions to develop a soil/rock data set to formulate geotechnical engineering recommendations for the design of the congestion improvements project. The work was performed in general accordance with our fee proposal dated January 8, 2020. The scope of work for this exploration included the following tasks and work efforts:

1. Research and review of readily available as-built information in the vicinity of the project site.
2. Compile the available subsurface information and engineering properties of the subsurface geomaterials to perform engineering analyses to determine the capacity of the existing foundations to support the viaduct and bridge widening alternatives.
3. Application for excavation permits from the applicable agencies and coordination of site access and underground utility toning by our engineer or geologist. One-Call Center was notified following the field boring layout.
4. Preparation and submittal of a traffic control plan in support of our field exploration activities on the highway or roadways.
5. Preparation and submittal of an accident prevention plan with activity hazard analysis in support of our field exploration activities on the highway/roadways and during nighttime hours.

6. Mobilization and demobilization of a truck-mounted drill rig, water truck, and two operators to the project site and back.

7. Drilling and sampling of 18 borings extending to depths of about 10 to 122.5 feet below the existing ground surface for a total of about 1,011.5 lineal feet of field exploration.

8. Performance of two seismic shear wave velocity profiling tests to evaluate the seismic site classification and to determine the shear wave velocities of the subsurface materials at the project site.

9. Provision of traffic control devices, signs, and special duty police officers during the geotechnical field exploration activities.

10. Coordination of the field exploration and logging of the borings by our field engineer and geologists.

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

12. Analyses of the field and laboratory data to formulate geotechnical recommendations for the project.

13. Preparation of this report summarizing our work and presenting our findings and geotechnical recommendations.

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

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

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

Detailed descriptions of our field exploration methodology and the Logs of Borings are presented in Appendix A. Results of the seismic shear wave velocity testing are presented in Appendix B. Results of the laboratory tests performed on selected soil samples are presented in Appendix C. Photographs of the core samples retrieved are presented in Appendix D. The analytical corrosivity test reports are presented in Appendix E.

END OF GENERAL

SECTION 2. SITE CHARACTERIZATION

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

2.1 Regional Geology

The Island of Oahu was built by the extrusion of basaltic lava from two shield volcanoes, Waianae and Koolau. The older volcano, Waianae, is estimated to be middle to late Pliocene in age, and the younger shield, 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 located 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 the heads of valleys, the erosional processes are dominated by the 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. In summary, the project site is situated on the southern flank of the Koolau Volcano and is underlain by fill, alluvial deposits, and basalt rock formation.

2.2 Existing Site Conditions

The project site encompasses approximately 0.7 miles of the eastbound lanes of the Interstate Route H-1 in the Kalihi area of Honolulu on the Island of Oahu, Hawaii. The project limits approximately extend from the Ola Lane Overpass to the Likelike Highway off-ramp (Exit 20A). The existing freeway generally consists of four travel lanes in each direction paved with asphaltic concrete (AC) pavement. Two bridge structures will be affected within the project limits: Interstate H-1 Highway over Kalihi Stream and Richard Lane crossing (referred to in this report as the Kalihi Stream/Richard Lane Bridge) on the western side of the project site, and the Gulick Avenue Overpass on the eastern side of the project site. Richard Lane is a relatively narrow two-lane AC roadway running parallel to Kalihi Stream.

Based on the topographic survey map provided, the elevation of the existing roadway surface ranges from about +28 feet Mean Sea Level (MSL) near Ola Lane, slopes up towards the Kalihi Stream Bridge (about +57.5 feet MSL), slopes down towards the Gulick Avenue Overpass (about 51.5 feet MSL), and then back up towards the Kalihi Street Interchange (about +60 feet MSL). The roadway elevation of the Gulick Avenue Overpass ranges from about +72.5 feet MSL on the north side of the bridge to about +67.5 feet MSL on the south side.

2.3 Subsurface Conditions

We explored the subsurface conditions at the project site by drilling and sampling sixteen borings, designated as Boring Nos. 1 through 16, extending to depths of about 10 to 122.5 feet below the existing ground surface. In addition, two borings, designated as Boring Nos. 101 and 101A, were drilled on the north side of the Gulick Avenue Overpass near the temporary pedestrian bridge. Five bulk samples of the near-surface soils, designated as Bulk-1 through Bulk-5, were obtained to evaluate the moisture density relationship and pavement support characteristics of the near-surface soils. The approximate boring and bulk sample locations are shown on the Overall Site Plan, Plate 2, and the Site Plans, Plates 3.1 and 3.2.

Our borings on Interstate Route H-1 generally encountered a surface fill layer about 1 to 8.5 feet thick underlain by recent alluvium, saprolite, basalt rock formation and older alluvium extending to the maximum depth explored of about 122.5 feet below the existing ground surface. The surface fill layer consisted of about 4 to 8 inches of asphaltic concrete, dense to very dense sandy gravel, silty/gravelly sand and cobbles, and medium stiff to hard silts and clays with sand and gravel. In two of the borings, Portland cement concrete about 21 inches thick was encountered.

Below the surface fill layer, saprolite and recent alluvium were encountered to about 5 to 15.5 feet below the existing pavement surface. The saprolite consisted of dense to very dense silty gravel/sand with some cobbles and medium stiff to hard silty/sandy clay and clayey silt with sand and gravel. The recent alluvium was composed of stiff to hard silty clay and sandy silt with gravel and cobbles.

The saprolite and recent alluvium were underlain by severely fractured to massive, un-weathered to moderately weathered, and medium hard to very hard basalt rock formation extending to depths of 10 to 67 feet. Beneath the basalt rock formation, older alluvial deposits were encountered extending to the maximum depth explored of 122.5 feet below the existing pavement surface. The older alluvium consisted of medium stiff to hard clays and silts with sands and gravels and medium dense gravelly/silty sand.

Our boring performed on Richard Lane (Boring No. 7) generally encountered a surface fill layer about 5 feet thick consisting of about 3 inches of asphaltic concrete, dense cobbles, and stiff silty clay with some sand. The surface fill layer was underlain by severely fractured to massive, slightly weathered, very hard basalt rock formation extending to about 51 feet below the existing pavement surface. Below the basalt rock formation, older alluvial deposits were encountered extending to the maximum boring depth of about 122 feet below the existing pavement surface. The older alluvium generally consisted of medium stiff to hard silty clay/clayey silt with sand and gravel, and medium dense gravelly sand.

The borings drilled on both sides of the Gulick Avenue Overpass abutments (Boring Nos. B-11 and B-13) generally encountered a surface fill layer about 2 to 5 feet thick underlain by severely fractured to massive, unweathered to moderately weathered, and medium hard to very hard basalt rock formation to 55 to 57 feet deep. Below the basalt rock formation, older alluvial deposits were encountered extending to the maximum boring depths of 122.5 feet below the existing ground surface. A 6.5 feet thick welded clinker layer was encountered as part of the basalt rock formation in one of the borings. In addition, a 10.5 feet thick pocket of older alluvium was encountered within the basalt rock formation in one of the borings.

The surface fill layer consisted of very stiff to hard silty clay and medium dense to dense silty gravel. The older alluvium was composed of medium stiff to hard gravelly/clayey/sandy silts, silty clay with some sand and silt, and medium dense silty sand.

The first boring drilled for the temporary pedestrian bridge (Boring No. 101) encountered concrete and a 4.5-foot tall void within the top 15 feet. The boring generally encountered a surface fill layer about 15.5 feet thick underlain by a soft to very hard basalt rock formation extending to the maximum depth explored of about 31 feet below the existing ground surface. The surface fill layer consisted of medium dense to very dense silty/gravelly sand and cobbles and boulders, concrete, and a void. Due to the unusual materials encountered in the upper section of Boring No. 101, a second boring (Boring No. 101A) was drilled approximately 10 feet east of Boring No. 101. Boring No. 101A encountered approximately 17 feet of medium dense to dense silty sand with some gravel overlying basalt rock formation.

We encountered groundwater in one of our borings at depths of about 19.5 and 29.6 feet below the existing ground surface, corresponding to elevations of about +21.4 to +31.5 feet MSL. It should be noted that due to the depths of the borings and the basalt formation encountered, rotary rock coring techniques utilizing drilling water were required to advance the deeper borings. Therefore, accurate in-situ groundwater readings could not be taken in most of our borings due to the time it took for the drilling water to dissipate. In addition, it should be noted that groundwater levels can fluctuate depending on surface water runoff, storm surge conditions, seasonal precipitation, perched groundwater, and other factors.

Detailed descriptions of the materials encountered from our field exploration are presented on the Logs of Borings, Plates A-1 through A-18, in Appendix A. The results of the seismic shear wave velocity tests are presented in Appendix B. Results of the laboratory tests performed on selected samples obtained from our field exploration are presented in Appendix C. Photographs of the core samples retrieved from our field exploration are presented in Appendix D. Corrosion test results are presented in Appendix E.

2.4 Seismic Design Considerations

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

2.4.1 Earthquakes and Seismicity

In general, earthquakes that occur throughout the world are caused by shifts in the tectonic plates. In contrast, earthquake activity in Hawaii is linked primarily to volcanic activity. Therefore, earthquake activity in Hawaii generally occurs before or during volcanic eruptions. In addition, earthquakes may result from the underground movement of magma that comes close to the surface but does not erupt. The Island of Hawaii experiences thousands of earthquakes each year, but most are so small that only sensitive instruments can detect them. However, some of the earthquakes are strong enough to be felt, and a few cause minor to moderate damage.

In general, earthquakes associated with volcanic activity are most common on the Island of Hawaii. Earthquakes that are directly associated with the movement of magma are concentrated beneath the active Kilauea and Mauna Loa Volcanoes on the Island of Hawaii. Because the majority of the earthquakes in Hawaii (over 90 percent) are related to volcanic activity, the risk of high seismic activity and degree of ground shaking diminishes with increased distance from the Island of Hawaii. The Island of Hawaii has experienced numerous earthquakes greater than Magnitude 5 (M5+); however, earthquakes are not confined only to the Island of Hawaii.

To a lesser degree, the Island of Maui has experienced several earthquakes greater than Magnitude 5. Therefore, moderate to strong earthquakes have occurred in the County of Maui. The effects of earthquakes occurring on the Islands of Hawaii and Maui may be felt on the Island of Oahu. For example, several small landslides occurred on the Island of Oahu as a result of the Maui Earthquake of 1938 (M6.8). In addition, some houses on the Island of Oahu were reportedly damaged as a result of the Lanai Earthquake of 1871 (M7+).

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

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

Tripler areas, reported more intense ground shaking, severe enough to have cracked reinforced concrete.

2.4.2 Liquefaction Potential

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

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

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

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

Based on the subsurface conditions encountered, the phenomenon of soil liquefaction is not a design consideration for this project site. The risk for potential liquefaction is low based on the subsurface conditions encountered (stiff and dense fill, basalt rock formation and relatively stiff and dense alluvium within the depths of our borings).

2.4.3 Soil Profile Type for Seismic Design

Seismic shear wave velocity profiling was conducted near the Kalihi Stream crossing and the Gulick Avenue Overcrossing to analyze the subsurface conditions more closely at the two bridge locations for seismic design considerations. Shear wave velocity profiling was performed at two selected locations (Boring Nos. 7 and 13) using seismic cone penetration testing (SCPT) equipment. Based on the subsurface conditions encountered in our field exploration, the weighted average shear wave velocity for the materials within the upper 100 feet of the soil profile at Boring Nos. 7 and 13 is on the order of about 1,628 and 1,629 feet per second, respectively. Results of the seismic shear wave velocity tests are provided in Appendix B.

Based on the subsurface materials encountered at the project site and the shear wave velocity profiling performed, we believe the project site may be classified from a seismic analysis standpoint as being a “Very Dense Soil and Soft Rock” site corresponding to a Site Class C soil profile type based on AASHTO 2020 LRFD Bridge Design Specifications, 9th Edition.

Based on the AASHTO 2020 LRFD Bridge Design Specifications, the two bridge structures will need to be designed based on an earthquake return period of 1,000 years. Based on a 1,000-year return period and the anticipated Site Class, the following seismic design parameters were estimated and may be used for the seismic analysis of the bridge structures planned for the project.

SEISMIC DESIGN PARAMETERS AASHTO 2020 LRFD BRIDGE DESIGN SPECIFICATIONS 1,000-YEAR RETURN PERIOD (~7% PROBABILITY OF EXCEEDANCE IN 75 YEARS)	
Parameter	**Value**
Peak Bedrock Acceleration, PBA (Site Class B)	0.174g
Spectral Response Acceleration (Site Class B), S_S	0.398g
Spectral Response Acceleration (Site Class B), S_1	0.109g
Site Class	"C"
Site Coefficient, F_{pga}	1.20
Site Coefficient, F_a	1.20
Site Coefficient, F_v	1.69
Design Peak Ground Acceleration, PGA (Site Class C) or A_s	0.209g
Design Spectral Response Acceleration, S_{DS}	0.477g
Design Spectral Response Acceleration, S_{D1}	0.184g
Seismic Design Category	"B"

END OF SITE CHARACTERIZATION

SECTION 3. DISCUSSION AND RECOMMENDATIONS

Our field exploration generally encountered a surface fill layer about 1 to 17 feet thick underlain by saprolite or recent alluvium followed by a basalt rock formation extending to depths of about 10 to 67 feet. Below the basalt rock formation, older alluvial deposits were encountered extending to the maximum depth explored about 122.5 feet below the existing ground surface. Groundwater was encountered in one of the drilled borings at depths of about 19.5 and 29.6 feet below the existing ground surface, corresponding to elevations of about +21.4 and +29.6 feet MSL. It should be noted that accurate in-situ groundwater readings could not be taken in most of the borings due to the time it took for the drilling water to dissipate.

Based on the information provided, we understand that new abutments will need to be constructed for the Gulick Avenue Overpass to widen Interstate Route H-1 in the eastbound direction and the westbound direction in the future. The use of drilled shaft retaining walls is planned for the northern and southern abutments for the overpass.

Based on the available as-built drawings, we understand that the existing Kalihi Stream/Richard Lane Bridge is supported by shallow foundations bearing on the underlying basalt rock formation. However, we understand that deep foundations are desired to support the Kalihi Stream Bridge expansion on the makai side of the bridge. To develop the required bearing and lateral load resistances, the proposed bridge expansion may be supported by a foundation system consisting of 60-inch diameter cast-in-place concrete drilled shafts.

Based on the information provided, we understand that both cut and fill walls are required on the makai side of Interstate Route H-1 for the freeway widening project. The majority of the new retaining walls will be in a cut condition with the exception of the retaining walls near the Kalihi Stream/Richard Lane Bridge, which will be constructed in a fill condition.

A detailed discussion of these items and other geotechnical aspects of the project are presented in the following sections.

3.1 Gulick Avenue Overpass

Based on the information provided, we understand that new abutments will need to be constructed for the Gulick Avenue Overpass structure to widen Interstate Route H-1 in the eastbound direction and the future widening in the westbound direction. Therefore, drilled shaft retaining walls are being considered for the northern and southern abutment structures for the overpass. Drilled shaft retaining walls typically consist of a series of closely spaced drilled shafts along the retaining wall alignment. The drilled shafts will serve as retaining structures, and temporary shoring would not be required during construction. After drilled shaft installation, a concrete facing will be installed in front of the drilled shaft wall.

An axial load demand of 260 kips per drilled shaft was provided by the project structural engineer. However, the length of the drilled shafts will be governed by the lateral load demands on the drilled shaft retaining walls. Therefore, we have provided the axial capacities for the drilled shaft length required to satisfy the lateral load demands. The drilled shaft foundations would derive support principally from adhesion between the drilled shaft and the basalt rock formation encountered during our field exploration near the proposed new abutment locations.

Based on our engineering analyses and the above assumptions, we recommend using drilled shafts with the compressive load capacities for the strength limit states based on Load and Resistance Factor Design (LRFD) methods for design of highway bridges provided in the table below. For the strength limit state, a resistance factor of 0.50 has been applied to the strength limit state capacities within the basalt rock formation for design of the drilled shaft foundations.

In general, we anticipate that the drilled shafts for the new abutments will be closely spaced, with a minimum spacing of 1.2 times the diameter of the shaft measured from center-to-center. Therefore, the effect of group action was considered in our axial load analyses. The compressive axial load capacities of the proposed 5-foot diameter drilled shafts for the bridge abutments are presented in the table below.

SUMMARY OF COMPRESSIVE AXIAL CAPACITIES (STRENGTH I LEVEL) FOR INDIVIDUAL DRILLED SHAFTS			
		Compressive Load Capacity **Per Drilled Shaft** (kips)	
Shaft Diameter (feet)	**Shaft Length** (feet)	**Unfactored Single Shaft Capacity**	**Strength Limit State**
5	30*	600	300
*Note: Drilled shaft length assumes that the shaft extends about 15 feet below the existing Interstate Route H-1 pavement elevation and achieves 10 feet of embedment into basalt rock formation.			

As noted above, the recommended length of the drilled shafts is based on a 10-foot minimum embedment into the basalt rock formation encountered in our borings. If the top of the basalt rock formation is found to be deeper during construction, the drilled shafts should be extended to achieve the minimum required embedment.

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

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

3.1.1 Lateral Load Resistance

In general, lateral load resistance for the drilled shaft is a function of the stiffness of the surrounding soil/rock, the stiffness of the shaft, allowable deflection at the top of the shaft, and induced moment in the shaft. To evaluate the lateral load resistance of the new bridge structures, stiffness modeling parameters were estimated based on the subsurface conditions encountered in the drilled borings. The stiffness modeling parameters were obtained using the program LPILE 2019 for Windows, which is a microcomputer adaptation of a finite difference, laterally loaded pile program. The program solves for a deflection and bending moment along a pile under lateral loads as a function of the depth. The analysis was carried out to generate non-linear "p-y" curves to represent soil moduli at various depths.

It should be noted that the project structural engineer has designed the new Gulick Avenue Overpass bridge structure to act as a rigid frame. Therefore, the lateral load demands on the drilled shaft retaining wall abutments were decreased due to the framing effect of the bridge system. Based on the information provided by the project structural engineer, the lateral load demands per drilled shaft are 110 kips in the longitudinal direction and 120 kips in the transverse direction for both abutments.

Due to the close spacing of the drilled shaft foundations, the effect of group action was considered in our lateral load analyses by including an efficiency factor in the direction of loading. These values assume that drilled shafts in the direction of loading are spaced at 6 feet on center for the 5-foot diameter drilled shafts. Results of our lateral load analyses are summarized in the table below.

SUMMARY OF LATERAL LOAD ANALYSES

Analysis Scenario	**Maximum Lateral Deflection** (inches)	**Maximum Shear** (kips)	**Maximum Induced Moment** (kip-feet)	**Depth to Maximum Moment** (feet)
Makai Abutment Longitudinal Loading	0.79	546	3,155	19.0
Makai Abutment Transverse Loading	0.60	349	2,286	19.0
Mauka Abutment Longitudinal Loading	1.54	813	4,535	22.5
Mauka Abutment Transverse Loading	0.91	421	2,652	22.0

NOTE: Analyses based on concrete compressive strength of 5,000 psi and a minimum of 2% longitudinal steel reinforcement.

3.1.2 Foundation Settlements

Settlement of the drilled shaft foundation will result from elastic compression of the shaft and subgrade response of the foundation embedded in the basalt formation. Total settlements of the drilled shafts are estimated to be on the order of about 0.5 inches. Therefore, differential settlements between the drilled shafts may be on the order of about 0.25 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

Groundwater was encountered in one of our nearby borings at a relatively high elevation. Therefore, we believe that artesian groundwater conditions may be present near the Gulick Avenue Overpass. The contractor should be prepared to contain the artesian water during drilled shaft construction.

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 the drilling operations, are important. The contractor should exercise care in drilling the shaft holes and in 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 Obstructions, Boulders, and Basalt Rock Formation

Where obstructions, boulders, and/or basalt rock formation are anticipated, some difficult drilling conditions will likely 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 shafts to the recommended depths and dimensions.

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

3.1.3.b Shallow Groundwater & Artesian Groundwater Conditions

Groundwater conditions are anticipated within the depths of the drilled shaft excavations and, therefore, concrete placement by tremie methods will be required during drilled shaft construction. The concrete should be placed in a suitable manner by displacing the water in an upward fashion from the bottom of the drilled hole. A low-shrink concrete mix with high slump (7 to 9 inches 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.

As mentioned above, artesian groundwater conditions may be present at the project site within the basalt rock formation and/or older alluvium layers. Therefore, additional measures may be required to maintain the integrity of all cast-in-place concrete structures below the groundwater elevations because artesian groundwater pressures may cause the cement matrix to be washed away considering the high slump concrete used and the retarding admixtures that are normally introduced into the drilled shaft concrete.

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 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 placement of concrete. Therefore, Geolabs observation of the drilled shaft installation operations is necessary to confirm the assumed subsurface conditions.

3.1.4 Method Shaft Program

Considering the large diameters and difficult drilling conditions for the drilled shafts, we recommend undertaking a method shaft program as part of pre-construction activities at a selected location to fulfill the following objectives:

- To examine the adequacy of the methods and equipment proposed by the contractor to install the large diameter drilled shafts into the existing subsurface materials.
- To assess the contractor's method of placing and extracting the temporary casing for the drilled shaft.
- To assess the contractor's method of concrete placement.

To achieve these objectives, we recommend the method shaft program consist of drilling a method shaft extending to the estimated tip elevation of the drilled shafts near the southern abutment location of the Gulick Avenue Overpass. We recommend a Geolabs representative observe the method shaft installation program to evaluate the contractor's method of drilled shaft installation and to evaluate the subsurface materials encountered in the drilled holes. Observation of the drilled shaft installation operations is a vital part of the foundation design to confirm the design assumptions.

3.1.5 Non-Destructive Integrity Testing

Based on the critical nature of the drilled shaft foundations for the new bridge abutments, we recommend conducting non-destructive integrity testing on the method shaft and production drilled shafts for the project. Crosshole Sonic Logging

(CSL) is one of the non-destructive integrity testing methods that has gained widespread use and acceptance for integrity testing of drilled shafts.

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

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

In general, the access tubes should be securely attached to the interior of the reinforcing cage as near to parallel as possible in the drilled shaft. We recommend casting a minimum of five access tubes into the concrete of the 5-foot diameter drilled shafts.

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

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

3.1.6 Wall Drainage

The drilled shaft retaining wall should be properly drained to reduce the potential for hydrostatic pressure acting against the wall. Vertical prefabricated geocomposite drains should be installed between the drilled shafts. Subsurface water within the geocomposite drains should be drained by weep holes extending to the wall face near the bottom of the wall. It is critical that these vertical drains be installed soon after excavation of the front of the drilled shafts to avoid drying and raveling of the existing soil between the shafts and difficulties that will be encountered during placement on an irregular surface. In addition, it is recommended that the vertical drains be covered with shotcrete soon after drain placement to avoid drying and raveling of the cut soil face.

3.2 Gulick Avenue Overpass – Center Pier

Based on the available as-built drawings, we understand that the existing Gulick Avenue Overpass center pier columns are supported by shallow foundations bearing on the underlying basalt rock formation. Based on the structural information provided, we understand that the Strength I axial load demand on each pier column footing will increase to 58,000 psf. Our field exploration encountered hard to very hard basalt in the vicinity of the existing pier footings with RQD values of 82 and 100 below the bottom of the existing center pier footing level. Based on the result of our laboratory testing, the uniaxial compressive strength of the basalt rock formation at the center pier location ranged from

17,550 to 18,480 pounds per square inch (psi). Based on our field exploration, laboratory testing, and engineering analyses, we recommend the following design values for the existing center pier structure based on LRFD methods.

GULICK AVENUE OVERPASS – CENTER PIER			
	Extreme Event Limit State	**Strength Limit State**	**Service Limit State**
Bearing Pressure (psf)	130,000	58,500	43,300
Coefficient of Sliding Friction	0.70	0.60	N/A

Due to the high foundation bearing pressures and the potential for cavities and/or voids in the underlying basalt formation, we recommend implementing a program of cavity probing and grouting on the sides of the existing pier footings.

We recommend drilling probe holes at 10-foot on centers along the sides of the pier footings. The center of the probe holes should be offset about 1-foot from the outside edge of the footings. The probe holes should be at least 3 inches in diameter and should extend to a depth of at least 10 feet below the planned bottom of the foundation. Geolabs should review the proposed probing hole layout to evaluate whether the above requirements are met.

If cavities and/or voids are encountered or suspected during the probing operation, additional probe holes should be drilled at closer spacing to help delineate the vertical and lateral extent of the cavity and/or void. The probe holes and cavities discovered should be backfilled with cement grout with a minimum 28-day compressive strength of 5,000 pounds per square inch (psi) and a slump of about 6 to 9 inches. The cement grout should be injected at low to moderate pressures.

Because of the potential for encountering cavities and/or voids at the project site, we recommend obtaining unit prices for additional probing and grouting during bidding. In addition, the probe drill should be available on-site until the probing and grouting operations are completed. The contractor also should be made aware that a longer lag time between probing/grouting operations and foundation construction might be required in the construction schedule.

The probing and grouting program should be conducted under the observation of a Geolabs representative. This would allow our firm to monitor the presence of cavities and/or voids and to allow additional recommendations to be made if excess grout take and/or changed conditions are observed.

3.3 Gulick Avenue Traffic Signal Pole Foundations

Based on the information provided, we understand that new 30-foot and 35-foot mast arm traffic signal poles are planned to replace the existing traffic signal poles at the intersection of Gulick Avenue and Beckley Street. Based on the structural loads provided and the anticipated subsurface soil conditions, we recommend supporting the new traffic signal poles on single cast-in-place drilled shaft foundations.

To develop the required bearing and lateral load resistances, the proposed new traffic signal pole structures may be supported by a foundation system consisting of cast-in-place concrete drilled shafts. The following structural loads were utilized to design the cast-in-place concrete drilled shaft foundations for the 30-foot and 35-foot mast arm traffic signal pole. An axial load demand of 2.6 kips was provided for each traffic signal pole and each load case.

30-FOOT AND 35-FOOT MAST ARM TRAFFIC SIGNAL POLES STRUCTURAL LOADS				
Mast Arm Length	**Load Case**	**Resultant Shear Force** (kips)	**Resultant Bending Moment** (kip-feet)	**Torsion** (kip-feet)
30-foot	1	3.1	60.9	35
	2	1.7	29.0	35
	3	2.6	63.0	35
35-foot	1	2.9	66.3	61
	2	1.2	47.0	61
	3	2.4	63.0	61

Based on the typical dimensions of the base plate and anchor bolts, we envision that 36-inch diameter cast-in-place concrete drilled shafts would be required for the proposed traffic signal poles. The cast-in-place concrete drilled shafts would derive vertical support principally from skin friction between the shafts and the surrounding soils. Our recommendations pertaining to the drilled shaft capacities are presented in the following table.

SUMMARY OF COMPRESSIVE AXIAL CAPACITIES FOR TRAFFIC SIGNAL POLE DRILLED SHAFTS			
		Compressive Load Capacity Per Drilled Shaft (kips)	
Shaft Diameter (feet)	**Shaft Length** (feet)	**Extreme Event Limit State**	**Strength I Limit State**
3	10	180	79

Uplift loads may be resisted by a combination of the dead weight of the drilled shaft and shear along the shaft surface area and adjacent soils. An ultimate uplift load capacity (Extreme Event Limit State) of 68 kips may be used for each of the traffic signal pole drilled shafts. The project structural engineer should check the capacity of the drilled shaft in tension.

The load bearing capacities of the drilled shafts will depend largely on the consistency of the soils. Because local variations in the subsurface materials likely will occur, it is imperative that our representative is present during the shaft drilling operations to confirm the subsurface conditions encountered during the drilled shaft construction and to observe the installation of the drilled shafts. In addition, contract documents should include provisions (unit prices) for additional drilling and extension of the drilled shafts during construction to account for unforeseen subsurface conditions. The following subsections address the design and construction of the drilled shaft foundations, which include:

- Lateral Load Resistance
- Foundation Settlements
- Drilled Shaft Construction Considerations

3.3.1 Lateral Load Resistance

The lateral load resistance of the drilled shafts is a function of the stiffness of the surrounding soil, the stiffness of the shafts, allowable deflection at the top of the shafts, and the induced moment in the shafts. The lateral load analyses were performed using the program LPILE 2019 for Windows, a microcomputer adaptation of a finite difference laterally loaded deep foundation program originally developed at the University of Texas at Austin. The program solves for deflection and bending moment along a deep foundation under lateral loads as a function of depth. The analysis was carried out with the use of non-linear “p-y” curves to represent soil moduli. The lateral deflection was then computed using the appropriate soil moduli at various depths.

Based on the structural loads provided, results of our lateral load analyses for the concrete drilled shaft foundation are presented in the following table. The top of the shaft was assumed to be free against rotation.

SUMMARY OF LATERAL LOAD ANALYSES				
Load Case (feet)	**Maximum Lateral Deflection** (inches)	**Maximum Shear** (kips)	**Maximum Induced Moment** (kip-feet)	**Depth to Maximum Moment** (feet)
30-foot Case 1	0.01	23	77	5.0
30-foot Case 2	0.01	12	38	5.0
30-foot Case 3	0.01	23	77	5.0
35-foot Case 1	0.01	24	81	5.0
35-foot Case 2	0.01	16	53	5.0
35-foot Case 3	0.01	23	75	5.0
NOTE: Analyses based on concrete compressive strength of 4,000 psi and a minimum of 1% longitudinal steel reinforcement.				

3.3.2 Foundation Settlements

Settlement of the drilled shaft foundation will result from elastic compression of the shaft and subgrade response of the foundation embedded in the subsurface soils. Total settlement of the drilled shaft is estimated to be on the order of less than 0.5 inches. We believe that a significant portion of the settlement is elastic and should occur as the loads are applied.

3.3.3 Drilled Shaft Construction Considerations

In general, the performance of the drilled shafts will depend 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 the drilled shaft depend, to a significant extent, on the frictional resistance 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 hole and in placing concrete into the drilled hole.

We understand that fill is planned in the areas around the proposed traffic signal poles. In addition, the in-situ subsurface materials generally consist of very stiff to hard clayey fill material overlying basalt rock formation. Therefore, some difficult drilling conditions should be expected when drilling into the existing basalt rock formation. The drilled shaft contractor will need to have the appropriate equipment and tools to drill through the hard rock material encountered during drilled shaft installation operations.

Based on our field exploration and the estimated length of the drilled shafts, groundwater is generally not expected in the drilled hole during the shaft installation work. Due to the relatively short length of the drilled shaft, concrete placement using the free fall method should be acceptable provided the concrete does not flow on the reinforcing cage. In the event of seasonal rainfall and/or perched groundwater, water may be encountered in the drilled hole and concrete placement by tremie method would be required.

A low-shrinkage concrete mix with a high slump (6 to 9-inch slump range) should be used to provide close contact between the drilled shaft and the surrounding soils. In addition, the concrete should be placed promptly after drilling (within 24 hours after drilling of the holes) to reduce the potential for softening of the sidewalls of the drilled hole.

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. Although the drilled shaft design is primarily based on skin friction, the bottom of the drilled hole should be relatively free of loose materials prior to placement of the concrete. Therefore, it is necessary for Geolabs to observe the drilled shaft installation operations to confirm the assumed subsurface conditions.

3.4 Kalihi Stream/Richard Lane Bridge – Deep Foundations

We understand that deep foundations are desired to support the Kalihi Stream Bridge expansion on the makai side of the bridge on either side of Kalihi Stream.

To develop the required bearing and lateral load resistances, the proposed bridge expansion may be supported by a foundation system consisting of cast-in-place concrete drilled shafts. The following structural loads were provided for the Kalihi Stream Bridge abutments for the Strength I Limit State.

KALIHI STREAM BRIDGE EXPANSION STRUCTURAL LOADS			
Abutment	**Axial Load** (kips)	**Maximum Shear Force** (kips)	**Maximum Bending Moment** (kip-feet)
West	850	140	3,400
East	850	300	4,800

Based on the information provided, we understand that 60-inch diameter cast-in-place concrete drilled shafts are desired. The cast-in-place concrete drilled shafts would derive vertical support principally from skin friction between the shafts and the surrounding soils. A top-of-shaft elevation of +32 feet Mean Sea Level (MSL) was used in our design. Our recommendations pertaining to the drilled shaft capacities are presented in the following table.

SUMMARY OF COMPRESSIVE AXIAL CAPACITIES (STRENGTH I LEVEL) FOR KALIHI STREAM BRIDGE DRILLED SHAFTS			
Shaft Diameter (feet)	**Shaft Length** (feet)	**Compressive Load Capacity Per Drilled Shaft** (kips)	
		Unfactored Single Shaft Capacity	**Strength I Limit State**
5	19	2,040	850
*Note: Drilled shaft length assumes that the top of shaft elevation is +32 feet MSL.			

The load bearing capacities of the drilled shafts will depend largely on the consistency of the soils. Because local variations in the subsurface materials likely will occur, it is imperative that our representative is present during the shaft drilling operations to confirm the subsurface conditions encountered during the drilled shaft construction and to observe the installation of the drilled shafts. In addition, contract documents should include provisions (unit prices) for additional drilling and extension of the drilled shafts during construction to account for unforeseen subsurface conditions. The following subsections address the design and construction of the drilled shaft foundations, which include:

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

3.4.1 Lateral Load Resistance

The lateral load resistance of the drilled shafts is a function of the stiffness of the surrounding soil, the stiffness of the shafts, allowable deflection at the top of the shafts, and the induced moment in the shafts. The lateral load analyses were performed using the program LPILE 2019 for Windows, a microcomputer adaptation of a finite difference laterally loaded deep foundation program originally developed at the University of Texas at Austin. The program solves for deflection and bending moment along a deep foundation under lateral loads as a function of depth. The analysis was carried out with the use of non-linear “p-y” curves to represent soil moduli. The lateral deflection was then computed using the appropriate soil moduli at various depths.

Based on the structural loads provided and a fixed-head condition for the tops of the drilled shafts, the results of our lateral load analyses for the concrete drilled shaft foundations are presented in the following table.

SUMMARY OF LATERAL LOAD ANALYSES				
Abutment	**Maximum Lateral Deflection** (inches)	**Maximum Shear** (kips)	**Maximum Induced Moment** (kip-feet)	**Depth to Maximum Moment** (feet)
West	0.04	585	3,421	0.3
East	0.11	886	4,886	0.5
NOTE: Analyses based on concrete compressive strength of 4,000 psi and a minimum of 1% longitudinal steel reinforcement.				

3.4.2 Foundation Settlements

Settlement of the drilled shaft foundation will result from elastic compression of the shaft and subgrade response of the foundation embedded in the basalt formation and older alluvium. Total settlements of the drilled shafts are estimated to be on the order of about 0.5 inches. Therefore, differential settlements between the drilled shafts may be on the order of about 0.25 inches. We believe a significant portion of the settlement is elastic and should occur as the loads are applied.

3.4.3 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 the drilling operations, are important. The contractor should exercise care in drilling the shaft holes and in 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.

3.4.3.a Cobbles, Boulders, and Basalt Rock Formation

The subsurface materials generally consist of relatively stiff and/or dense fill material overlying basalt rock formation. Alluvium, consisting of very stiff to hard silts and clays with some cobbles and boulders, may also be encountered above the basalt rock formation. Therefore, some difficult drilling conditions should be expected when drilling through the in-situ alluvial soils and into the existing basalt rock formation. The drilled shaft contractor will need to have the appropriate equipment and tools to drill through the cobbles/boulders and hard rock material encountered during drilled shaft installation operations.

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

3.4.3.b Shallow Groundwater Conditions

Based on our field exploration and the vicinity of the planned drilled shaft foundations to Kalihi Stream, groundwater conditions are anticipated within the depths of the drilled shaft excavations. Therefore, concrete placement by tremie methods will be required during drilled shaft construction. The concrete should be placed in a suitable manner by displacing the water in an upward fashion from the bottom of the drilled hole. A low-shrink concrete mix with high slump (7 to 9 inches 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 should not commence until the concrete for the recently constructed drilled shaft has cured for a minimum of 24 hours.

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. Although the drilled shaft design is primarily based on skin friction, the bottom of the drilled hole should be relatively free of loose materials prior to placement of the concrete. Therefore, it is necessary for Geolabs to observe the drilled shaft installation operations to confirm the assumed subsurface conditions.

3.4.4 Non-Destructive Integrity Testing

Based on the critical nature of the drilled shaft foundations for the bridge expansion, we recommend conducting non-destructive integrity testing on the production drilled shafts for the project. Crosshole Sonic Logging (CSL) is one of the non-destructive integrity testing methods that has gained widespread use and acceptance for integrity testing of drilled shafts.

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

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

In general, the access tubes should be securely attached to the interior of the reinforcing cage as near to parallel as possible in the drilled shaft. We recommend casting a minimum of five access tubes into the concrete of the 5-foot diameter drilled shafts.

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

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

3.5 Kalihi Stream/Richard Lane Bridge – Shallow Foundations

We understand that spread footings are desired on the west side of Richard Lane to support the bridge widening improvements. Based on the available as-built drawings, we understand that the existing Kalihi Stream Bridge is supported by shallow foundations bearing on the underlying basalt rock formation. Our boring on Richard Lane encountered basalt rock formation at a depth of about 5 feet below the existing pavement surface. Therefore, we believe that shallow spread and/or continuous footings bearing on the underlying basalt rock formation may be utilized for foundation support of the proposed bridge widening improvements. Based on our analyses, the following values may be used for the design of shallow foundations based on LRFD methods.

KALIHI STREAM/RICHARD LANE BRIDGE – SHALLOW FOUNDATIONS			
	Extreme Event Limit State	**Strength Limit State**	**Service Limit State**
Bearing Pressure (psf)	60,000	27,000	20,000
Coefficient of Sliding Friction	0.70	0.60	N/A
Passive Pressure Resistance (Rock Conditions) (psi)	50	25	N/A
Passive Pressure Resistance (Soil Conditions) (pcf)	290	145	N/A

The passive pressure resistances for rock conditions (in pounds per square inch) presented in the table above are for a rectangular distribution of uniform pressure. It is assumed that the footing will be poured neat against the near-vertical face excavation into the basalt rock formation. If the footings will be backfilled with well-compacted fill material, the passive pressure resistances for soil conditions should be used. The passive resistance should be reduced if future utility installation within the passive wedge is anticipated. Unless covered by pavements or slabs, the passive resistance in the upper 12 inches should be neglected.

Soft and/or loose materials (or less competent basalt rock formation, such as clinker seams) encountered at the bottom of the footing excavations should be over-excavated to expose the underlying dense basalt rock formation. The less competent basalt rock formation includes the closely to severely fractured basalt and clinker seams that may be encountered at the site. The over-excavation should be backfilled with concrete or the bottom of the footing may be extended deeper to bear on the more competent basalt rock surface. In addition, concrete for the footings should be placed neatly against the sides of the foundation excavations.

In general, the bottom of the footings should be embedded a minimum of 18 inches below the lowest adjacent finished grade. Footings constructed near tops of slopes or on sloping ground should be embedded deep enough to provide a minimum horizontal set-back distance of 8 feet measured from the outside edge of the footings to the slope face.

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

If foundations are designed and constructed in strict accordance with the recommendations presented herein, we estimate total settlements of the foundations to be between less than 0.5 inches. Differential settlements between adjacent footings supported on similar materials may be on the order of about 0.25 inches or less.

Lateral loads acting on the retaining walls may be resisted by frictional resistance developed between the bottom of the foundation and the bearing soil and by passive earth pressure acting against the near-vertical faces of the foundation system. The coefficient of sliding friction and passive pressure resistance are provided in the table above. The passive earth pressure value assumes that the soils around footings are well-compacted. Unless covered by pavements or slabs, the passive pressure resistance in the upper 12 inches of the soils should be neglected.

A Geolabs representative should observe the footing excavations prior to the placement of reinforcing steel and concrete to confirm the foundation bearing conditions and the required embedment depths.

3.6 Kalihi Stream/Richard Lane Bridge – Probing and Grouting

Cavities and/or voids are commonly present in basalt rock formation. To reduce the potential for loss of foundation support resulting from the collapse of cavities below the new Kalihi Stream/Richard Lane Bridge shallow foundations, we recommend implementing a program of cavity probing and grouting of the foundations.

We recommend drilling probe holes at 10-foot on centers for the continuous strip footings. In addition, probe holes should be drilled at each isolated spread footing location (one probe hole per 50 square feet of footing area). The probe holes should be at least 3 inches in diameter and should extend to a depth of at least 10 feet below the planned bottom of the foundation. Geolabs should review the proposed probing hole layout to evaluate whether the above requirements are met.

If cavities and/or voids are encountered or suspected during the probing operation, additional probe holes should be drilled at closer spacing to help delineate the vertical and lateral extent of the cavity and/or void. The probe holes and cavities discovered should be backfilled with sand-cement grout or cement grout with a minimum 28-day compressive strength of 4,000 psi and a slump of about 6 to 9 inches for the probe holes. The grout should be injected at low to moderate pressures.

Because of the potential for encountering cavities and/or voids at the project site, we recommend obtaining unit prices for additional probing and grouting during bidding. In addition, the probe drill should be available on-site until the probing and grouting operations are completed. The contractor also should be made aware that a longer lag time between probing/grouting operations and foundation construction might be required in the construction schedule.

The probing and grouting program should be conducted under the observation of a Geolabs representative. This would allow our firm to monitor the presence of cavities and/or voids and to allow additional recommendations to be made if excess grout take and/or changed conditions are observed.

3.7 Retaining Walls

Based on the information provided, we understand that both cut and fill retaining walls are required on the makai side of Interstate Route H-1 for the freeway widening project. We understand that conventional retaining wall structures will be used. In addition, a drilled shaft retaining wall structure will be used at the Gulick Avenue

Overpass. The majority of the new conventional retaining walls will be in cut conditions with the exception of the retaining walls near the Kalihi Stream/Richard Lane bridge where the existing ground slopes downward and will be in a fill condition.

The following sections provide our foundation and lateral earth pressure recommendations for the conventional retaining structures planned for the project site.

3.7.1 Retaining Wall Foundations

Based on the subsurface conditions encountered during our field exploration, we believe that the proposed retaining walls on the southern side of the Interstate Route H-1 freeway in the eastbound direction will be bearing on relatively stiff/dense fill, saprolite, alluvium, and basalt formation. Based on our drilled borings, we believe the retaining walls from the vicinity of Ola Lane to approximately Sta. 34+50 and at the Kalihi Stream/Richard Lane Bridge will be underlain by the relatively stiff/dense soil materials encountered in our borings.

Based on our analyses, the following values may be used for the design of the retaining walls bearing on soil material based on LRFD methods.

RETAINING WALL FOUNDATIONS BEARING ON SOIL MATERIAL **Approx. Vicinity of Ola Lane Overpass to Sta. 34+50 and Kalihi Stream/Richard Lane Bridge**			
	Extreme Event Limit State	**Strength Limit State**	**Service Limit State**
Bearing Pressure (psf)	12,000	5,400	4,000
Coefficient of Sliding Friction	0.35	0.28	N/A
Passive Pressure Resistance (pcf)	330	165	N/A

The passive earth pressure values in the table above assume that the soils around footings are well compacted. The passive resistance should be reduced if future utility installation within the passive wedge is anticipated. Unless covered by pavements or slabs, the passive pressure resistance in the upper 12 inches of the soils should be neglected.

Soft and/or loose materials encountered at the bottom of footing excavations should be over-excavated until dense materials are exposed in the footing excavation. The over-excavation should be backfilled with select granular fill materials, moisture-conditioned to above the optimum moisture content, and compacted to a minimum of 90 percent relative compaction, or may be backfilled with lean concrete or flowable fill.

Based on the subsurface conditions encountered during our field exploration, we anticipate that the retaining structures on the southern side of the Interstate Route H-1 freeway in the eastbound direction may be underlain by basalt formation from approximately Sta. 34+50 to the vicinity of the Kalihi Street Overpass.

Based on our analyses, the following values may be used for the design of the retaining walls bearing on basalt formation based on LRFD methods.

RETAINING WALL FOUNDATIONS BEARING ON BASALT FORMATION Approx. Sta. 34+50 to Vicinity of Kalihi Street Overpass			
	Extreme Event Limit State	**Strength Limit State**	**Service Limit State**
Bearing Pressure (psf)	60,000	27,000	20,000
Coefficient of Sliding Friction	0.70	0.60	N/A
Passive Pressure Resistance (Rock Conditions) (psi)	50	25	N/A
Passive Pressure Resistance (Soil Conditions) (pcf)	290	145	N/A

The passive pressure resistances for rock conditions (in pounds per square inch) presented in the table above are for a rectangular distribution of uniform pressure. It is assumed that the footing will be poured neat against the near-vertical face excavation into the basalt rock formation. If the footings will be backfilled with well-compacted fill material, the passive pressure resistances for soil conditions should be used. The passive resistance should be reduced if future utility installation within the passive wedge is anticipated. Unless covered by pavements or slabs, the passive resistance in the upper 12 inches should be neglected.

Soft and/or loose materials (or less competent basalt formation, such as clinker seams) encountered at the bottom of the footing excavations should be over-excavated to expose the underlying dense basalt formation. The less competent basalt formation includes the closely to severely fractured basalt and clinker seams that may be encountered at the site. The over-excavation should be backfilled with concrete or the bottom of footing may be extended deeper to bear on the more competent basalt rock surface. In addition, concrete for the footings should be placed neatly against the sides of the foundation excavations.

The bottom of wall footings should be embedded at a minimum depth of 24 inches below the lowest adjacent finished grade. Wall footings oriented parallel to the direction of the slope should be constructed in stepped footings. Foundations located next to utility trenches or easements should be embedded below a 45-degree imaginary plane extending upward from the bottom edge of the utility trench, or the bottom of footing should be extended to a depth as deep as the inverts of the utility lines. This requirement is necessary to avoid surcharging adjacent below-grade structures with additional structural loads and to reduce the potential for appreciable foundation settlement.

3.7.2 Static Lateral Earth Pressures

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

We anticipate that the retaining walls may be retaining imported select granular fill and/or in-situ soil material. The recommended lateral earth pressures for the design of the cut and fill retaining walls on the makai side of Interstate Route H-1 Eastbound are presented in the table below.

LATERAL EARTH PRESSURES **Cut and Fill Retaining Structures Along Makai Side of Interstate Route H-1 Eastbound**			
Backfill Condition	**Earth Pressure Component**	**Active** (pcf)	**At-Rest** (pcf)
Level Backfill	Horizontal	40	60
	Vertical	None	None
Maximum 2H:1V Sloping Backfill	Horizontal	60	80
	Vertical	16	21

The values provided above assume that on-site soils or select granular fill materials will be used to backfill behind the retaining walls. It is assumed that the backfill behind the retaining wall will be compacted to between 90 and 95 percent relative compaction. Over-compaction of the backfill should be avoided.

Based on our field exploration program, we anticipate that the retaining walls at the Kalihi Stream/Richard Lane Bridge will be retaining fill and alluvial soils. The recommended lateral earth pressures for the design of the retaining structures at the Kalihi Stream/Richard Lane Bridge are presented in the table below.

LATERAL EARTH PRESSURES **Retaining Structures at Kalihi Stream/Richard Lane Bridge**			
Backfill Condition	**Earth Pressure Component**	**Active** (pcf)	**At-Rest** (pcf)
Level Backfill	Horizontal	38	56
	Vertical	None	None

LATERAL EARTH PRESSURES Retaining Structures at Kalihi Stream/Richard Lane Bridge			
Backfill Condition	**Earth Pressure Component**	**Active** (pcf)	**At-Rest** (pcf)
Maximum 2H:1V Sloping Backfill	Horizontal	63	80
	Vertical	15	20

The values provided above assume that the walls will be retaining the fill and alluvial soils encountered during our field exploration.

At the Gulick Avenue Overpass, we anticipate that the drilled shaft retaining walls will be retaining a surface layer of fill and residual soils about 17 feet thick underlain by basalt rock formation. The recommended lateral earth pressures for the design of the drilled shaft retaining walls at the Gulick Avenue Overpass Abutments are presented in the table below.

LATERAL EARTH PRESSURES Drilled Shaft Retaining Walls at Gulick Avenue Overpass Abutments			
Backfill Condition	**Earth Pressure Component**	**Active** (pcf)	**At-Rest** (pcf)
Level Backfill (Soil)	Horizontal	38	56
	Vertical	None	None
Maximum 2H:1V Sloping Backfill (Soil)	Horizontal	61	77
	Vertical	30	38
Basalt Rock Formation	Horizontal	5	5
	Vertical	None	None

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

Surcharge stresses due to areal surcharges, line loads, and point loads within a horizontal distance equal to the depth of the retaining walls should be considered in the design. The following table presents our recommended surcharge stresses for each retaining structure. The surcharge stresses should be treated as rectangular distributions acting on the entire height of the wall and are presented as percentages of the vertical surcharge pressure.

LATERAL EARTH PRESSURES DUE TO VERTICAL SURCHARGE STRESSES		
Retaining Wall Location	**Active** (percent)	**At-Rest** (percent)
Makai Side of Interstate Route H-1	33	50
Kalihi Stream/Richard Lane Bridge	36	53
Gulick Avenue Overpass	36	53

Additional analyses during design may be needed to evaluate the surcharge effects of point loads and line loads.

3.7.3 Dynamic Lateral Earth Pressures

Dynamic lateral earth forces due to seismic loading (a_{max}= 0.209g) for the different retaining structures at the project site are presented in the table below.

DYNAMIC LATERAL EARTH PRESSURES DUE TO SEISMIC LOADING			
Retaining Wall Location	**Backfill Condition**	**Active Condition** (H^2)	**At-Rest Condition** (H^2)
Makai Side of Interstate Route H-1	Level	4.0	6.8
	2H:1V Sloping Backfill	13.0	21.0
Kalihi Stream/Richard Lane Bridge	Level	3.6	6.2
	2H:1V Sloping Backfill	17.5	28.0

DYNAMIC LATERAL EARTH PRESSURES DUE TO SEISMIC LOADING			
Retaining Wall Location	**Backfill Condition**	**Active Condition** (H^2)	**At-Rest Condition** (H^2)
Gulick Avenue Overpass (Soil Conditions Only)	Level	3.6	6.2
	2H:1V Sloping Backfill	17.5	28.0

Notes:
- Values above multiplied by H^2 per linear foot of wall length, where H is the height of the wall in feet.
- The dynamic lateral earth pressures for the Gulick Avenue Overpass only apply to the surface soil layer encountered behind the existing abutments (about 17 feet thick measured from the existing ground surface at the time of our field exploration).

The active condition assumes that the walls will be allowed to move laterally by up to about 1 inch in the event of an earthquake. The at-rest condition should be used when walls are restrained. The resultant force of the dynamic lateral earth pressures should be assumed to act through the mid-height of the wall. It should be noted that the forces due to dynamic lateral earth pressures presented above are in addition to the static lateral earth pressures. An appropriately reduced factor of safety may be used when dynamic lateral earth forces are accounted for in the design of the retaining structures.

3.7.4 Drainage

The retaining walls should be well-drained to reduce the potential for build-up of hydrostatic pressures. A typical drainage system would consist of a 12-inch wide zone of permeable material, such as No. 3B Fine gravel (ASTM C33, No. 67 gradation), placed directly around a perforated pipe (perforations facing down) at the base of the wall discharging to an appropriate outlet or weepholes. As an alternative, a prefabricated drainage product, such as MiraDrain or EnkaDrain, may be used instead of the drainage material. The prefabricated drainage product also should be hydraulically connected to a perforated pipe at the base of the wall.

The backfill from the bottom of the wall to the bottom of the perforated pipe or weep hole should consist of relatively impervious materials to reduce the potential for significant water infiltration into the subsurface. In addition, the upper 12 inches of the retaining structure backfill should consist of relatively impervious materials to reduce the potential for significant water infiltration behind the retaining structure unless covered by concrete slabs at the surface.

3.8 Gulick Avenue Overpass Barrier Retaining Walls

Based on the information provided, we understand that barrier retaining walls are planned at the base of the new mauka Gulick Avenue Overpass abutment. The barrier walls will be placed in front of the drilled shaft retaining walls and backfilled with select granular fill at a 1.5H:1V inclination. Grouted Rubble Paving (GRP) will then be placed on top of the select granular fill backfill at an inclination of 1H:1V. We understand that the barrier wall near the Mauka abutment will be up to 6.5 feet in height.

We anticipate that new walls will be embedded at least 2 feet below the existing ground surface. Based on our field exploration, we anticipate that the new walls will be bearing on basalt rock formation. Therefore, the foundation design parameters from Section 3.7.1 for retaining walls bearing on basalt formation may be used.

Based on the above information, the following static lateral earth pressures may be used to design the new barrier retaining walls.

LATERAL EARTH PRESSURES Barrier Retaining Walls at Base of Gulick Avenue Overpass Drilled Shaft Retaining Wall Abutment			
Wall Location	**Backfill Conditions**	**Earth Pressure Component**	**Active** (pcf)
Mauka Abutment	1.5H:1V Select Granular Fill with 1H:1V GRP	Horizontal	97
		Vertical	39

Dynamic lateral earth forces due to seismic loading (a_{max}= 0.209g) for the barrier retaining walls are presented in the table below.

DYNAMIC LATERAL EARTH PRESSURES Barrier Retaining Walls at Base of Gulick Avenue Overpass Drilled Shaft Retaining Wall Abutment		
Wall Location	**Backfill Conditions**	**Active Condition** (H^2)
Mauka Abutment	1.5H:1V Select Granular Fill with 1H:1V GRP	9.5

3.9 Instrumentation and Monitoring

Because the proposed widening project is located within a developed area and in close proximity to existing structures and vibrations are anticipated during rock excavation and drilling, we believe that a pre-construction survey and construction monitoring of vibrations and movements should be performed for the project. A pre-construction survey of the existing building structures adjacent to the new structures should be conducted, including photographs and detailed descriptions of pre-existing distresses, to document the existing conditions prior to the commencement of construction. Displacement monitoring points should be installed on structures (buildings, walls, etc.) and on the ground adjacent to structures in close proximity to the construction areas. Before the start of construction, the monitoring points should be surveyed to establish baseline readings for the monitoring points. Benchmarks should be established for the survey work. Readings of the monitoring points should be performed on a daily basis during rock excavation or drilling near existing structures. In addition, vibration monitoring should be performed during construction especially during rock excavation and drilling.

3.10 Temporary Pedestrian Bridge Foundations

We understand that a temporary pedestrian bridge structure will be constructed on the east side of the Gulick Avenue Overpass. In addition, we understand that the temporary bridge structure will be designed by others.

Boring No. 13 is located in the vicinity of the south end of the temporary bridge. Boring Nos. 101 and 101A are located in the vicinity of the north end of the temporary pedestrian bridge. Concrete and an approximately 4.5 feet tall void were encountered in Boring No. 101.

3.11 Site Grading

Grading plans were not available at the time this report was prepared. However, we anticipate that site grading for the project will primarily consist of cutting existing slopes on the makai side of Interstate Route H-1 eastbound, excavations for bridge and retaining wall foundations, backfilling retaining structures, and underground utility installation. Items of site grading that are addressed in the subsequent subsections include the following:

1. Site Preparation
2. Fills and Backfills
3. Fill Placement and Compaction Requirements
4. Excavation

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

3.11.1 Site Preparation

At the onset of earthwork, areas within the contract grading limits should be thoroughly cleared and grubbed. Vegetation, debris, demolished manmade structures, and other unsuitable materials should be removed and disposed of properly off-site to reduce the potential for contamination of the excavated materials designated to be reused as fill and/or backfill. If soft or wet soils are encountered during clearing, over-excavation may be required to remove the soft or wet materials to expose firm and/or dense soils. The resulting over-excavation should be backfilled with compacted fill material.

After clearing and grubbing, the existing ground surface should be scarified to a depth of 8 inches, moisture-conditioned to at least 2 percent above the optimum moisture content, and compacted to a minimum of 90 percent relative compaction. For pavement subgrades, the compaction requirement should be a minimum of 95 percent relative compaction. Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same soil established in accordance with AASHTO T-180 (or ASTM D1557). Optimum moisture is the water content (percentage by dry weight) corresponding to the maximum dry density.

Where scarification of the subgrade is not possible (subgrades of basalt rock formation), we recommend proof-rolling the subgrades with a large vibratory drum roller (minimum 15 tons static weight) for a minimum of eight passes to help detect and collapse near-surface cavities and/or voids. The vibratory drum roller should be operated at a speed of about 300 feet per minute (about 3.5 miles per hour). Yielding areas, loose areas, or cavities disclosed during the clearing and proof-rolling operations should be over-excavated and backfilled with compacted fill materials. The depth of over-excavation should extend until dense underlying materials are exposed and should be evaluated by our field representative.

3.11.2 Fills and Backfills

In general, we anticipate the excavations will likely encounter fill, alluvium, residual soil, saprolite, and basalt rock formation at relatively shallow depths. The excavated on-site soil and basalt rock formation may be used as a source of fill material provided that the material meets the following requirements.

In general, the on-site soil and basalt rock formation encountered during our field exploration should be suitable for use as general fill materials, provided that the maximum particle size is less than 3 inches in largest dimension. The excavated on-site materials may be used as general fill or backfill materials if they are screened of the over-sized materials and/or processed to meet the gradation requirements (less than 3 inches in largest dimension). In addition, fill materials should be free of vegetation and deleterious materials. Excavated soft and wet soils may not be re-used as a source of fill and backfill materials.

Imported materials to be used as select granular fill should consist of non-expansive granular material, such as crushed coral or basalt. The select granular fill should be well-graded from coarse to fine with particles no larger than 3 inches in largest dimension. The material should also contain between 10 and 30 percent particles passing the No. 200 sieve. The material should have a laboratory CBR value of 20 or more and should have a maximum swell value of 1 percent or less. Imported fill materials should be tested for conformance with these recommendations prior to delivery to the project site for the intended use.

3.11.3 Fill Placement and Compaction Requirements

Fills and backfills should be moisture-conditioned to at least 2 percent 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. 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 (AASHTO T180) test procedures. Optimum moisture is the water content (percentage by dry weight) corresponding to the maximum dry density.

Compaction should be accomplished by sheepsfoot rollers, vibratory rollers, multiple-wheel pneumatic-tired rollers, or other types of acceptable compaction equipment. Where compaction is less than required, additional compactive effort should be applied with adjustment of moisture content as necessary, to obtain the specified compaction.

3.11.4 Excavation

Based on our field exploration, hard basalt rock formation was encountered at relatively shallow depths throughout most of the project alignment. Therefore, the contractor will likely encounter difficult excavation conditions during construction where the hard basalt rock formation is encountered.

We anticipate that the surface fill, alluvium, residual soil, and saprolite encountered during our field exploration along the project alignment may be excavated readily with normal heavy excavation equipment, such as excavators. However, cobbles and boulders are frequently encountered in these types of soil deposits and should be expected. Excavations that encounter cobbles and boulders within the on-site soils and excavations extending into the underlying basalt rock formation may require the use of hoerams or chipping.

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

3.12 Underground Utility Structures

Based on the current design concepts, we understand that underground utility structures will be constructed along Interstate Route H-1 eastbound and Richard Lane near the Kalihi Stream/Richard Lane Bridge. Based on the subsurface conditions encountered in our borings along the project alignment, we anticipate most of the new underground utility structures will be situated within the fill, basalt formation, or alluvial deposits.

3.12.1 Underground Utility Structure Foundations

Based on our drilled borings, we believe the underground utility structures planned along Interstate Route H-1 eastbound from the vicinity of Ola Lane to approximately Sta. 34+50 may bear on/within the relatively stiff/dense fill and alluvial materials encountered in our borings.

Based on our analyses, the following values may be used for the design of the underground utility structures bearing on/within the stiff/dense fill and alluvial materials based on LRFD methods.

UNDERGROUND UTILITY STRUCTURE FOUNDATIONS BEARING ON STIFF/DENSE FILL AND ALLUVIAL MATERIAL Interstate Route H-1 Eastbound from Approx. Vicinity of Ola Lane Overpass to Sta. 34+50			
Description	**Extreme Event Limit State**	**Strength Limit State**	**Service Limit State**
Bearing Pressure (psf)	12,000	5,400	4,000
Coefficient of Sliding Friction	0.35	0.28	N/A
Passive Pressure Resistance (pcf)	330	165	N/A

In general, the exposed soil subgrades should be scarified to a depth of at least 8 inches, moisture-conditioned to at least 2 percent above the optimum moisture, and recompacted to at least 95 percent relative compaction to provide a relatively firm and smooth bearing surface prior to the placement of aggregate subbase, reinforcing steel or concrete.

Based on the subsurface conditions encountered during our field exploration, we anticipate that underground utility structures: on the southern side of the Interstate Route H-1 freeway in the eastbound direction from approximately Sta. 34+50 to the vicinity of the Kalihi Street Overpass; near the abutments of the Gulick Avenue overpass; and along Richard Lane near the Kalihi Stream/Richard Lane Bridge, may bear on/within medium hard to very hard basalt formation.

Based on our analyses, the following values may be used for the design of the underground utility structures bearing on basalt formation based on LRFD methods.

UNDERGROUND UTILITY STRUCTURE FOUNDATIONS BEARING ON MEDIUM HARD TO VERY HARD BASALT FORMATION

- **Interstate Route H-1 Eastbound from Approx. Sta. 34+50 to Vicinity of Kalihi Streat Overpass**
- **Near the Gulick Avenue Overpass Abutments**
- **Richard Lane near the Kalihi Stream/Richard Lane Bridge**

	Extreme Event Limit State	Strength Limit State	Service Limit State
Bearing Pressure (psf)	60,000	27,000	20,000
Coefficient of Sliding Friction	0.70	0.60	N/A
Passive Pressure Resistance (Rock Conditions) (psi)	50	25	N/A
Passive Pressure Resistance (Soil Conditions) (pcf)	290	145	N/A

The passive pressure resistances for rock conditions (in pounds per square inch) presented in the table above are for a rectangular distribution of uniform pressure. These values assume that the underground utility structure footings and walls will be poured neat against the near-vertical face excavation into the basalt rock formation. However, we anticipate that the underground utility structures may consist of precast concrete structures. If this is the case, the footings/walls will be backfilled with well-compacted fill material and the passive pressure resistances for soil conditions should be used. The passive resistance should be reduced if future utility installation within the passive wedge is anticipated. Unless covered by pavements or slabs, the passive resistance in the upper 12 inches should be neglected.

Soft and/or loose materials encountered at the bottom of footing excavations should be over-excavated until dense materials are exposed in the footing excavation. The over-excavation should be backfilled with concrete.

We recommend providing a cushion layer consisting of at least 12 inches of aggregate subbase below the drainage structures to provide uniform bearing support. The aggregate subbase layer would also serve as a working platform during construction. Aggregate subbase cushion layer should be compacted to at least 95 percent relative compaction. Aggregate subbase materials below the drainage structures should conform to the requirements stipulated in Subsection 703.17 of the State of Hawaii, Standard Specifications for Road and Bridge Construction, 2005.

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

3.12.2 Static Lateral Earth Pressures

The underground utility structures should be designed to resist the lateral earth pressure due to adjacent soil and surcharge effects. In general, the underground utility structure walls should be designed for the at-rest condition. The lateral earth pressure does not include hydrostatic pressures that might be caused by groundwater trapped behind the walls. The recommended lateral earth pressure for the design of the underground utility structures, expressed in equivalent fluid pressures of pounds per square foot per foot of depth (pcf), is presented in the following table.

LATERAL EARTH PRESSURES FOR DESIGN OF UNDERGROUND UTILITY STRUCTURES	
Backfill Condition	**At-Rest** (pcf)
Level Backfill Above Water	60

The value provided above assumes that granular soils will be used to backfill around the underground utility structures. It is assumed that the backfill around the drainage structures will be compacted to between 90 percent and 95 percent relative compaction. However, over compaction of the drainage structure backfill should be avoided. Compaction should be accomplished by suitable compaction equipment.

Surcharge stresses due to areal surcharges, line loads, and point loads within a horizontal distance equal to the depth of the underground utility structures should be considered in the design. For uniform surcharge stresses imposed on the loaded side of the structure, a rectangular distribution with a uniform pressure equal to 33 percent of the vertical surcharge pressure acting on the entire height of the structures may be used for design. For underground utility structures that are restrained, a rectangular distribution equal to 50 percent of the vertical surcharge pressure acting over the entire height of the structures may be used for design. Additional analyses during design may be needed to evaluate the surcharge effects of point loads and line loads.

3.12.3 Dynamic Lateral Earth Pressures

Dynamic lateral earth forces due to seismic loading will need to be considered in the design of the underground utility structures based on LRFD design methods. An appropriately reduced factor of safety (or resistance factor) may be used when dynamic lateral earth forces are accounted for in the design of retaining structures. For restrained conditions, dynamic lateral earth forces due to seismic loading (a_{max} = 0.209g) may be estimated by using $6.8H^2$ pounds per linear foot of wall length for level backfill conditions, where H is the height of the wall in feet. The resultant force should be assumed to act through the mid-height of the wall. The dynamic lateral earth forces are in addition to the static lateral earth pressures provided above. An appropriately reduced factor of safety may be used when dynamic lateral earth forces are accounted for in the design of the retaining structures.

3.13 Underground Utility Lines

We anticipate that new underground utilities will be installed for the project. We envision that most of the trenches for utilities will be excavated in the near-surface soils encountered in the borings drilled. Some of the trench excavations may also extend into the basalt rock formation encountered during our field exploration. In general, granular bedding consisting of 6 inches of open-graded gravel (AASHTO M43, No. 67 gradation materials) is recommended below the pipes for uniform support. Free-draining granular materials, such as open-graded gravel (AASHTO M43, No. 67 gradation materials), should also be used for the initial trench backfill up to about 12 inches above the pipes to provide adequate support around the pipes and to reduce the compaction effort of the backfill. It is critical to use free-draining materials around the pipes 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, which could result in backfill settlement.

The upper portion of the trench backfill from the level 12 inches above the pipes to the top of the subgrades or finished grade may consist of the on-site soils generally less than 3 inches in maximum particle size. The backfill material should be moisture-conditioned to above the optimum water content, placed in maximum 8-inch level loose lifts, and mechanically compacted to no less than 90 percent relative compaction to reduce the potential for appreciable future ground subsidence. Where trenches are below pavement areas, the compaction requirement for the upper 3 feet of the trench backfill below the pavement grade should be increased to at least 95 percent relative compaction.

3.14 Corrosion Potential

Two sets of laboratory corrosion tests, including pH, minimum resistivity, chloride content, and sulfate content, were performed on selected samples obtained during our field exploration to evaluate the corrosivity of the near-surface soils at the project site. The test results are summarized and presented in Appendix C. Detailed results of the Chloride Content (EPA 300.0) and Sulfate Content (EPA 300.0) tests performed by TestAmerica Laboratories, Inc. are presented in Appendix E.

Design of metallic substructures, such as metallic piping, should consider the effects of the corrosive environment on the substructure. Resistivity is generally recognized as one of the most significant soil characteristics regarding the corrosivity of the soil to buried metallic objects. In general, the lower the resistivity, the greater the potential for corrosion of the buried metallic structure. Conversely, the higher the resistivity, the less likely the soil will contribute to the corrosion of metallic objects. Results of the resistivity testing indicate that the on-site soils have resistivity values ranging from 1,100 to 1,600 ohm-cm with pH values varying from 7.97 to 8.34. Therefore, the on-site near-surface soils may be considered very corrosive based on the Board of Water Supply, City and County of Honolulu Water System External Corrosion Control Standards dated 1991.

In addition, chloride content and sulfate content were performed by TestAmerica Laboratories, Inc. to evaluate the corrosivity of the on-site soils encountered. Based on the chloride and sulfate content tests performed on the on-site soils, the test values are generally relatively low. It may be appropriate to consult with a professional corrosion engineer to review the test results and provide detailed recommendations for corrosion protection.

3.15 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 earthwork recommendations provided herein. If this review is not made, Geolabs cannot be responsible for the misinterpretation of our recommendations.

3.16 Post-Design Services/Services During Construction

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

1. Review of drilled shaft foundation installation submittals
2. Observation of the method drilled shaft installation
3. Observation of the production drilled shaft
4. Observation of shallow foundation excavations
5. Observation of probing and grouting
6. Observation of proof-rolling
7. Observation of the subgrade soil preparation
8. Observation of fill placement and compaction

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

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

END OF DISCUSSION AND RECOMMENDATIONS

SECTION 4. LIMITATIONS

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

The field boring locations indicated herein are approximate, having been estimated using a handheld GPS device. Elevations of the borings were estimated from contours and spot elevations shown on the Topographic Survey Map transmitted by Jacobs on January 5, 2021. The field boring locations and elevations should be considered accurate only to the degree implied by the methods used.

The stratification breaks shown on the graphic representations of the borings depict the approximate boundaries between soil types and, as such, may denote a gradual transition. Water level data from the borings were measured at the times shown on the graphic representations and/or presented in the text of this report. This data has been reviewed and interpretations made in the formulation of this report. However, it should be noted that groundwater levels can fluctuate depending on surface water runoff, storm surge conditions, seasonal precipitation, perched groundwater, and other factors.

This report has been prepared for the exclusive use of Jacobs and their client, the State of Hawaii Department of Transportation – Highways Division for specific application to the design of the *Interstate Route H-1 (EB) Improvements, Ola Lane Overpass to Kalihi Street Interchange* project in Honolulu on the Island of Oahu, Hawaii 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 engineers in the preparation of the design documents for the highway improvements project. Therefore, this report may not contain sufficient data, or the proper information, for use to form the basis for the preparation of construction cost estimates or contract bidding. A contractor wishing to bid on this project should retain a competent geotechnical engineer to assist in the interpretation of this report and/or performance of site-specific exploration for bid estimating purposes.

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

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

END OF LIMITATIONS

CLOSURE

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

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Respectfully submitted,

GEOLABS, INC.

By ______________________
Nicholas Kam, P.E.
Project Engineer

By ______________________
Gerald Y. Seki, P.E.
Vice President

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