GEOLABS, INC.

Geotechnical Engineering and Drilling Services

TECHNICAL MEMORANDUM

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TO:	KSF, Inc.	**FROM:**	Robin M. Lim / Glenn Barut
ATTN:	Mr. Gary Iwamoto	**NO. OF PAGES:**	14
SUBJECT:	Repairs to Wailua River Bridge Preliminary Geotechnical Recommendations		
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This technical memorandum presents our preliminary geotechnical recommendations for the Repairs to Wailua River Bridge, Federal Aid Project No. ER-23(001) project located along Kuhio Highway in the Wailua area on the Island of Kauai, Hawaii. It should be noted that the preliminary geotechnical recommendations presented herein are based on our research and review of available subsurface information and our experience in the project vicinity only. A detailed summary of our findings and recommendations will be contained in the geotechnical engineering report, which should be consulted when it becomes available.

PROJECT CONSIDERATIONS

The bridge repair project is at the Wailua River Bridge located along Kuhio Highway on the Island of Kauai, Hawaii. We understand that an assessment of the bridge foundations was conducted after the heavy rain events that occurred on March 17 and 29, 2020. Based on the assessment, it was determined that the streambed elevation was below the critical scour level; therefore, an emergency bridge closure was implemented on April 13, 2020. The bridge remains closed until a bridge repair design is finalized.

Based on the information provided, we understand that the bridge repair will consist of keeping the existing bridge deck and abutments and replacing the seven existing piers and the pile foundations with new piers and foundations. We understand the new bridge foundations will need to consider scour in the foundation design. Based on the information provided, the estimated scour elevation is at about -26 feet Mean Sea Level (MSL). The bridge foundations will be designed based on Load and Resistance Factor Design (LRFD) methods.

Since the existing bridge deck will remain in place, temporary support/shoring of the bridge will be required when the piers are demolished during construction. In addition, temporary bridge structures would need to be constructed to allow construction equipment access to the bridge piers.

REGIONAL GEOLOGY

The Island of Kauai is composed of a single basalt shield volcano built by the extrusion of lavas of the Waimea Canyon Volcanic Series during the late Pliocene Epoch (more than $2^{1}/_{4}$ million years before present). Following the cessation of this main shield building phase, renewed volcanic activity occurred with the extrusion of basaltic lavas of the post-erosional Koloa Volcanic Series and the concurrent deposition of alluvial sediments of the Palikea Formation.

The majority of the Island of Kauai is covered by lavas of the Waimea Canyon Volcanic Series. These lavas consist of four distinct formations: Napali, Olokele, Haupu, and Makaweli. These formations are comprised of thin-bedded a`a and pahoehoe flows to massive basalt flows that ponded in calderas and graben.

Rocks of the Koloa Volcanic Series cover most of the eastern half of the Island of Kauai. These rocks generally are characterized as thick flows of dense basalt extruded from groups of vents aligned in north-south trends in various locales. Associated with the vents are pyroclastic materials, which usually form low cinder cones at the vent.

During the Pleistocene Epoch (Ice Age), many sea level changes occurred as a result of widespread glaciation in the continental areas of the world. As the great continental glaciers accumulated, the level of the ocean fell since less water was available to fill the oceanic basins. Conversely, as the glaciers receded or melted, global sea levels rose because more water was available. The land mass of Kauai remained essentially stable during these changes and the fluctuations were eustatic in nature. These glacio-eustatic fluctuations resulted in stands of the sea that were both higher and lower relative to the present sea level of Kauai.

The higher sea level stands caused the accumulation of deltas and fans of terrigenous sediments in the heads of the old bays, accumulation of reef deposits at correspondingly higher elevations, and lagoonal/marine sediments in the quiet waters protected by fringing reefs.

The basaltic rock built by the extrusion of lavas of the Koloa Volcanic Series generally are characterized by flows of jointed dense vesicular basalt with interbedded thin clinker layers. The weathering process has formed a mantle of residual soils, which grade to saprolite with depth. In general, saprolite is composed of mainly silty material and is typical of the tropical weathering of volcanic rocks. The saprolite grades to basaltic rock formation with increasing depth.

ANTICIPATED SUBSURFACE CONDITIONS

Boring information was obtained from our reports and memorandum prepared for several projects in the vicinity of the bridge site. In addition, information from available plans were used. The references containing relevant subsurface information are provided below.

1. "Geotechnical Engineering Exploration, Kuhio Highway Widening, Vicinity of Leho Drive to Kuamoo Road, Lydgate to Kapaa Bike/Pedestrian Path, Project No. CMAQ-0700(49), Kapaa, Kauai, Hawaii" dated May 12, 2008.

2. "Geotechnical Engineering Exploration, Wailua River Electrical Crossing, Kuhio Highway Widening, Federal Aid Project No. NH-056-1(505), Wailua, Kauai, Hawaii" dated October 1, 2009.

3. Memorandum for the "Structural Countermeasures For Scour Critical Bridges, Project No. BR-1500(079), Wailua River Plantation Bridge - Preliminary Findings" dated February 6, 2015.

4. Plans of Kauai Belt Road (Wailua Bridge Section), Federal Aid Project No. F 12 (15), Lihue District, Island of Kauai, 1948.

Based on the geotechnical information obtained near the south end of the bridge, we anticipate that the subsurface conditions at the south end of the bridge could consist of very dense cobbles and boulders below the mudline. The boulders and cobbles are likely fill materials. The cobbles and boulders could extend to elevations of about --2 to -3.5 feet MSL. We anticipate soft to very hard basalt formation would be encountered below the layer of cobbles and boulders. Subsurface information from a boring drilled downstream near Pier No. 2 of the Wailua River Plantation Bridge indicated that soft alluvium consisting of silt with gravel and cobbles could be encountered below the mudline extending down to an elevation of about -32 feet MSL. Below the alluvium, we anticipate that medium hard basalt rock formation will be encountered.

Based on geotechnical information obtained near the center of the bridge, we anticipate that alluvium consisting of medium dense silty sands underlain by estuarine deposits consisting of soft to medium stiff clayey/sandy silts could be encountered below the mudline to about Elevation -102 feet MSL. We anticipate alluvium consisting of medium dense silty gravel would be encountered below the estuarine deposits. The medium dense silty gravel is anticipated to extend down to about Elevation -113 feet MSL.

Based on geotechnical information obtained near the northern end of the bridge, we anticipate the northern end of the bridge between Pier No. 7 and Abutment No.2 could consist of cobbles and boulders underlain by beach deposits consisting of medium dense to dense poorly graded sands. Beach deposits likely would be encountered at the ground surface and below the mudline in the southerly direction away from Abutment No. 2. We anticipate the beach deposits would extend to elevations of about -7 to -14 feet MSL. Lagoonal deposits consisting of loose to medium dense silty sand and soft to medium stiff clayey/sandy silts are anticipated below the beach deposits. The lagoonal deposits grade to stiff clayey silts at elevations of about -81 to -96 feet MSL. The stiff clayey silts are anticipated to extend down to about -123.5 feet MSL (elevation of the deepest boring at the northern end of the bridge).

Groundwater adjacent to the river is anticipated at elevations ranging from about +3.2 to -4.2 feet MSL. In addition, water levels within the river should fluctuate between +2 and -2 feet MSL. The groundwater level adjacent to the river and water level within the river likely will vary in response to tidal fluctuations, seasonal precipitation, and other factors.

SEISMIC DESIGN CONSIDERATIONS

Based on the AASHTO LRFD Bridge Design Specifications, 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, the potential for liquefaction at the project site, and the soil profile for seismic design.

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 they can be detected only by sensitive instruments. However, some of the earthquakes are strong enough to be felt, and a few cause minor to moderate damage.

In general, earthquakes associated with volcanic activity are most common on the Island of Hawaii. Because the majority of earthquakes in Hawaii (over 90 percent) are related to volcanic activity, the risk of seismic activity and degree of ground shaking diminishes with increased distance from the Island of Hawaii. The Island of Hawaii has experienced numerous earthquakes greater than Magnitude 5 (M5+); however, earthquakes are not confined only to the Island of Hawaii. To a lesser degree, the Island of Maui has experienced several earthquakes greater than Magnitude 5. Therefore, moderate to strong earthquakes have occurred in the County of Maui.

The effects of earthquakes occurring on the Islands of Hawaii and Maui may be felt on the Island of Oahu. For example, 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+).

Seismic hazards on the Island of Kauai generally are considered to be low. Earthquakes with a magnitude greater than 5 have not been recorded on the Island of Kauai.

Liquefaction Potential

Soil liquefaction is a condition where saturated cohesionless soils located near the ground surface undergo a substantial loss of strength due to the build-up of excess pore water pressures resulting from cyclic stress applications induced by earthquakes. In this process, when the loose saturated sand deposit is subjected to vibration (such as during an earthquake), the soil tends to densify and decrease in volume causing an increase in pore water pressure. If drainage is unable to occur rapidly enough to dissipate the build-up of pore water pressure, the effective stress (internal strength) of the soil is reduced.

Under sustained vibrations, the pore water pressure build-up could equal the overburden pressure, essentially reducing the soil shear strength to zero and causing it to behave as a viscous fluid. During liquefaction, the soil acquires sufficient mobility to permit both horizontal and vertical movements, and if not confined, will result in significant deformations.

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

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

We have evaluated the potential for liquefaction during a light to minor earthquake (less than M5.0) with an associated peak ground acceleration of 0.104g. Based on our analyses, it appears that the soils anticipated at the site have a factor of safety greater than 1.0 against liquefaction. Therefore, the potential for liquefaction at the site is considered low.

Soil Profile

The bridge repair will need to be designed in accordance with AASHTO 2020 LRFD Bridge Design Specifications (9th Edition) and HDOT “Design Criteria for Bridges and Structures” dated August 8, 2014.

Based on the anticipated subsurface conditions, we believe the site may be classified from a seismic analysis standpoint as being a "Stiff Soil Profile" site corresponding to a Site Class D soil profile type based on AASHTO 2020 LRFD Bridge Design Specifications, 9^{th} Edition.

Based on the AASHTO 2020 LRFD Bridge Design Specifications, the bridge repairs 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 repairs planned for the project.

SEISMIC DESIGN PARAMETERS
WAILUA RIVER BRIDGE
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)	0.065g
Spectral Response Acceleration (Site Class B), S_S	0.143g
Spectral Response Acceleration (Site Class B), S_1	0.042g
Site Class	"D"
Site Coefficient, F_{pga}	1.60
Site Coefficient, F_a	1.60
Site Coefficient, F_v	2.40
Design Peak Ground Acceleration, PGA (Site Class C) or A_s	0.104g
Design Spectral Response Acceleration, S_{DS}	0.228g
Design Spectral Response Acceleration, S_{D1}	0.100g
Seismic Design Category	"A"

DISCUSSION AND PRELIMINARY GEOTECHNICAL RECOMMENDATIONS

As mentioned previously, the bridge repair project involves keeping the existing bridge deck and abutments and replacing the seven existing piers with pile foundations (Pier Nos. 1 through 7) with new pier and foundation supports. Due to the scour potential and the relatively heavy structural load demands, we recommend supporting the existing Wailua River Bridge on a deep foundation system consisting of concrete drilled shaft foundations.

Based on the anticipated subsurface conditions, we believe drilled shaft foundations with a nominal diameter of 6 feet may be used to support the existing bridge structure. The drilled shaft foundations would derive support principally from adhesion between the drilled shaft and the soft to very hard basalt rock (south end of bridge), medium dense to dense beach deposits and medium stiff to stiff alluvium/estuarine deposits anticipated at the site. The following sections of this technical memorandum contain detailed discussions of our preliminary recommendations pertaining to the design of the bridge repair project.

Drilled Shaft Foundations

As mentioned above, we anticipate the scour potential at the existing bridge structure location and the relatively heavy structural load demands dictate supporting the existing bridge on a drilled shaft foundation system. General information and foundation loads for each pier provided by the project structural engineer are presented in the following table.

FOUNDATION LOAD DEMAND AT PIERS			
Pier No.	**Load Case**	**Compressive Load** (kips)	**No. of Drilled Shafts**
1	Strength I	1,400	2
2	Strength I	1,900	2
3	Strength I	1,800	2
4	Strength I	1,800	2
5	Strength I	1,800	2
6	Strength I	1,900	2
7	Strength I	1,400	2

Based on the anticipated subsurface conditions at the project site and the anticipated structural loads, we recommend supporting the existing bridge structure on drilled shafts having a diameter of 6 feet. Our recommendations pertaining to the drilled shaft foundation support system are presented in the following table.

DRILLED SHAFT FOUNDATIONS					
Pier No.	**Scour Elevation** (feet MSL)	**Drilled Shaft Cutoff Elevation** (feet MSL)	**Drilled Shaft Tip Elevation** (feet MSL)	**Drilled Shaft Length** (feet)	**Nominal Single Shaft Capacity** (kips)
1	-26	+10	-75	85	1,950
2	-26	+9	-135	144	2,650
3	-26	+8	-130	138	2,500

DRILLED SHAFT FOUNDATIONS					
Pier No.	**Scour Elevation** (feet MSL)	**Drilled Shaft Cutoff Elevation** (feet MSL)	**Drilled Shaft Tip Elevation** (feet MSL)	**Drilled Shaft Length** (feet)	**Nominal Single Shaft Capacity** (kips)
4	-26	+8	-125	133	2,500
5	-26	+7	-125	132	2,500
6	-26	+6	-125	131	2,650
7	-26	+6	-105	111	1,950

Because the drilled shaft cutoff elevations extend above the mudline and water level within the stream, the portion of the drilled shaft that is free-standing will need to be formed. In addition, the upper portion of the soil profile is weak and granular in nature (collapses without casing to stabilize the drilled shaft sidewalls). Therefore, we recommend installing a permanent casing from the drilled shaft cutoff elevation down to about 60 feet below the drilled shaft cutoff elevation. The permanent casing will act as form for the portion of the drilled shaft that is free-standing and also will reduce the potential for significant drop in head of the concrete while in a fluid state.

The drilled shaft foundations would derive support principally from adhesion between the sides of the drilled shaft and the soft to very hard basalt rock (southern end of bridge), medium dense to dense beach deposits, and medium stiff to stiff alluvium/estuarine deposits anticipated at the site. The contribution from end bearing was discounted in our analyses due to practical difficulties associated with cleaning the bottom of the drilled hole.

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

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

Based on our evaluation of the anticipated 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 sections address the design and construction of the drilled shaft foundations:

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

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 soft to very hard basalt rock (southern end of bridge), medium dense to dense beach deposits, and medium stiff to stiff alluvium/estuarine deposits. Total settlements of the 85 to 144-foot-deep drilled shafts are estimated to be less than 0.5 inches. Therefore, differential settlements between the drilled shafts and piers may be about 0.25 inches or less. We believe a significant portion of the settlement is elastic and should occur as the loads are applied.

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 soft to very hard basalt rock, medium dense to dense beach deposits, and medium stiff to stiff alluvium/estuarine deposits. 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.

Due to the granular nature of the beach deposits and the soft and/or loose consistency of the alluvium and estuarine/lagoonal deposits anticipated at the site, there is a high potential for caving-in of the materials during the drilling operations. To reduce the potential for significant caving-in of the drilled holes, temporary casing of the drilled holes will be required during drilled shaft installation. Care should be exercised during removal of the temporary casing to reduce the potential for "necking" of the drilled shaft concrete.

It should be noted that these soils extend down to relatively deep depths. Therefore, we recommend fully casing the drilled shaft holes to reduce the potential for caving-in of the soils. Installation and removal of large diameter temporary casing at these depths will be difficult using conventional methods. Therefore, an oscillator or rotary system should be considered to extend the drilled shaft to the recommended drilled shaft tip elevations and to remove the temporary casing during the concrete placement operations.

We anticipate that soft to very hard basalt rock formation would be encountered at the southern end of the bridge. In addition, cobbles and boulders at both ends of the bridge structure are anticipated. Therefore, some difficult drilling conditions likely will be encountered and should be expected. The drilled shaft subcontractor will need to have the appropriate equipment and tools to drill through these types of natural obstructions, where encountered. The drilled shaft subcontractor will need to demonstrate that the proposed drilling equipment (and coring tools, where appropriate) will be capable of installing the drilled shafts to the recommended depths and dimensions.

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

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

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

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

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

Method Shaft or Trial Shaft Program

A method shaft or trial shaft program normally is required and highly recommended for the bridge repair project. Considering the large diameter and high structural load capacities of the drilled shafts, we recommend undertaking a method shaft or trial 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 high-capacity drilled shafts into the existing subsurface soil deposits.

- To assess the contractor's method of placing and extracting the permanent and temporary casing for the drilled shaft.

- To assess the contractor's method of concrete placement.

To achieve these objectives, the method shaft program should consist of drilling a 6-foot diameter shaft extending down to an elevation of -135 feet MSL. The method shaft should be located at the center or northern end of the bridge. We recommend a Geolabs representative observe the installation of the method shaft to evaluate the contractor's method of drilled shaft installation and to evaluate the subsurface materials encountered in the drilled hole. Observation of the drilled shaft installation operations is a vital part of the foundation design to confirm the design assumptions.

Bi-Directional Load Test

As part of the pre-construction activities, we recommend conducting one static load test. The load test should be conducted on 6-foot diameter drilled shaft extending down to an elevation of about -135 feet MSL. The results of the load test will be used to confirm or modify the estimated tip elevations of the production drilled shafts. The load test shaft should be structurally reinforced and instrumented with embedment strain gauges for load testing purposes. As a minimum, two embedment strain gauges should be placed at each level, starting near the load cell location at an elevation of about 5 feet above and below the load cell and subsequently at about 10-foot intervals.

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

The expandable load cell should be capable of applying a load of at least 2,500 kips in each direction for the 6-foot diameter load test shaft. The expandable load cell will need to be attached to the reinforcing steel cage prior to lowering the cage into the drilled hole. The expandable load cell should be placed at an elevation of about -105 feet MSL.

The drilled shaft load test should be performed in general accordance with the Quick Load Test Method of ASTM D1143. The load test shaft should be loaded to failure to evaluate the ultimate side shear resistance and end-bearing components of the shaft. Installation of the expandable load cells, installation of the embedment strain gauges, performance of the bi-directional axial load tests, and presentation of the load test data should be performed by a professional experienced in these types of load testing procedures. The load test shaft should be loaded at increments of about 200 kips and should be held for a minimum of 4 hours (each hold) at the load intervals of 100 percent, 150 percent, and 200 percent of the design capacity of shaft capacity for the 6-foot diameter test shaft to evaluate the potential for creep effects.

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

Non-Destructive Integrity Testing

Based on the critical nature of the drilled shaft foundations, we recommend conducting non-destructive integrity testing on all the production drilled shafts. One of the non-destructive integrity testing methods, Crosshole Sonic Logging (CSL), has gained widespread use and acceptance.

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

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

To reduce the potential de-bonding between the access tube and the surrounding concrete, we recommend that the access tubes consist of standard steel pipe with a minimum inside diameter of 2 inches. In addition, the access tube should be equipped with watertight coupling. In general, the access tubes should be securely attached to the interior of the reinforcing cage as near to parallel as possible in the drilled shaft. We recommend casting a minimum of six access tubes at equal distance from each other into the concrete of the 6-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. The bottom of the access tube should be permanently capped. It is imperative that joints required to achieve the full length of the access tubes be watertight. The contractor is responsible for taking extra care to prevent damaging the access tubes during the placement of the reinforcing cage into the drilled hole. The tubes should be filled with potable water as soon as possible, but no later than 4 hours after the concrete placement. Subsequently, the top of the access tubes should be capped with watertight caps.

The CSL test of drilled shafts should be conducted after at least 5 days of curing time, but no later than 20 days after concrete placement. In addition, the CSL test of drilled shafts should be performed in general accordance with ASTM D 6760. In the event that a drilled shaft is observed 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. A Geolabs representative should determine the coring location and should be present to observe the coring of the drilled shaft. After completion of the crosshole sonic logging of the drilled shafts, all access tubes should be filled with grout of the same strength as the drilled shaft concrete.

LIMITATIONS

The analyses and preliminary geotechnical recommendations submitted in this technical memorandum are based, in part, upon information obtained from visual observations, boring information from previous projects in the vicinity, and our experience in the area. Variations of conditions between and beyond our observations and existing field exploration points may occur, and the nature and extent of these variations may not become evident until construction is underway. If variations then appear evident, it will be necessary to re-evaluate the recommendations presented herein.

This technical memorandum has been prepared for the exclusive use of KSF, Inc. for specific application to the Repairs to Wailua River Bridge project in accordance with generally accepted geotechnical engineering principles and practices. No warranty is expressed or implied.

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

The owner/client should be aware that unanticipated subsurface conditions are commonly encountered. Unforeseen subsurface conditions, such as 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.

CLOSURE

If you have questions regarding this technical memorandum or need additional information, please contact our office.