3. DISCUSSION AND RECOMMENDATIONS

Seismic evaluation of Kaholo Bridge was performed using the soil information obtained from our borings. Based on our evaluation, the Hilo shallow abutment foundations appear to be slightly above highly weathered to moderately weathered basalt. The Hilo shallow wingwall foundations appear to be bearing on stiff residual soils consisting of clayey silt. The Honokaa shallow abutment and wingwall foundations appear to be bearing on stiff to very stiff clayey silt. In general, our analyses for the stiffness modeling parameters of the foundations included the following:

- Estimation of the ultimate bearing capacity of the shallow foundations.
- Estimation of the lateral load resistance of the shallow foundations.
- Estimation of the static and dynamic lateral earth pressures acting on the bridge structure.

Based on the seismic evaluation of the bridge structure by the project structural engineer, we understand that appreciable lateral deflections of the bridge structure would occur during a seismic event. The lateral deflection of the bridge structure in the longitudinal direction would be reduced by the passive pressure resistance of the existing shallow bridge foundations and the stiffness of the abutment fills. A group of battered micropiles would be installed to provide resistance to the transverse lateral load and reduce the amount of transverse lateral deflection of the bridge structure.

To provide the lateral load resistance in the transverse direction during a seismic event, we recommend using a 7.625-inch diameter cased micropile system with a minimum grout bulb diameter of 7.625 inches for the battered micropiles. The uplift and lateral supporting capacities of the micropile would be derived primarily from skin friction between the micropile bonded zone and the surrounding saprolite soils and highly to moderately weathered soft to medium hard basalt formation. The bonded zone of the micropile should be embedded a minimum of 30 and 45 feet for the Hilo and Honokaa abutments, respectively. A detailed discussion of these items and our geotechnical recommendations for the design of the bridge seismic retrofit are presented in the following sections of this report.

3.1 Stiffness Modeling Analysis

In order to evaluate the lateral load resistance of the existing bridge structure, foundation capacities and stiffness modeling parameters were estimated based on the soil descriptions obtained from our borings and our laboratory test results.

We understand that soil foundation parameters consisting of the foundation bearing pressure, friction resistance, and static and dynamic lateral earth pressures of the existing bridge structure are required; therefore, an evaluation was conducted of the subsurface conditions and available as-built information. Based on our evaluation, Hilo abutment foundations appear to be slightly above highly weathered to moderately weathered basalt. The Hilo wing wall foundations appear to be bearing on stiff residual soils consisting of clayey silt. The Honokaa abutment and wing wall foundations appear to be bearing on stiff to very stiff clayey silt.

Based on the anticipated bearing conditions of the existing bridge abutment and wing wall foundations, the foundation bearing pressures and friction resistance for the extreme event limit state are provided in the following tables.

<table>
<tr><th colspan="6">HILO ABUTMENTS AND WINGWALLS</th></tr>
<tr><td colspan="2" rowspan="2">Location</td><td>Hilo</td><td>Hilo</td><td>Hilo</td><td>Hilo</td></tr>
<tr><td>North Abutment</td><td>South Abutment</td><td>West Wingwall</td><td>East Wingwall</td></tr>
<tr><td colspan="2">Size of Footing</td><td>19' x 10.5'</td><td>11.5' x 10'</td><td>12' x 11'</td><td>10' x 6.5'</td></tr>
<tr><td colspan="2">Bottom of Footing</td><td>+711</td><td>+721</td><td>+719</td><td>+726</td></tr>
<tr><td colspan="2">Soil Profile Type (Seismic Analysis)</td><td colspan="4">Site Class C, A_s = 0.456g</td></tr>
<tr><td colspan="2">Estimated Extreme Event Bearing Capacity (ksf)</td><td>35.2</td><td>36.9</td><td>7.9</td><td>7.6</td></tr>
<tr><td>Extreme Event Sliding Resistance</td><td>Friction, $\tan\delta$</td><td>0.43</td><td>0.43</td><td>0.40</td><td>0.40</td></tr>
</table>

HONOKAA ABUTMENTS AND WINGWALLS					
Location		Honokaa North Abutment	Honokaa South Abutment	Honokaa West Wingwall	Honokaa East Wingwall
Size of Footing		11.5' x 10'	19' x 10'	10' x 6.5'	11' x 6'
Bottom of Footing		+735	+727	+740	+730
Soil Profile Type (Seismic Analysis)		Site Class C, A_s = 0.456g			
Estimated Extreme Event Bearing Capacity (ksf)		7.8	11.4	11.6	11.4
Extreme Event Sliding Resistance	Friction, $\tan\delta$	0.40	0.43	0.43	0.43

The recommended static lateral earth pressures acting on the abutment and wing wall structures are presented in the following table.

STATIC LATERAL EARTH PRESSURES	
Active (pcf)	**At-Rest** (pcf)
38	58

Dynamic lateral earth forces due to seismic loading are based on a seismic loading peak ground acceleration (As) of 0.456g. The table below summarizes the dynamic lateral earth forces acting on the retaining structures in the event of an earthquake versus the estimated wall displacements.

DYNAMIC LATERAL EARTH FORCES	
Lateral Movement (inches)	**Dynamic Lateral Earth Forces** (H^2 pounds per linear foot)
1	47
2	33
3	20

Please note that the above table only applies to 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 above dynamic lateral earth forces are in addition to the static lateral earth pressures provided previously.

3.2 Micropiles

As mentioned previously, a group of battered micropiles will be installed at each abutment to provide lateral load resistance in the transverse direction during a seismic event. In general, a micropile consists of a small diameter (usually less than 12 inches) drilled and grouted pile with steel reinforcing. A micropile is typically constructed by drilling a borehole, placing reinforcing steel in the hole, and grouting the borehole. Micropiles are desirable because they can be readily installed in access restrictive environments, such as low headroom areas, and in numerous soil types and ground conditions. Micropile equipment is also more compact, making transport on narrow roadways feasible. In addition, the installation of the micropiles generally causes minimal disturbance to adjacent structures, the adjacent soils, and the environment.

Based on our analyses and the availability of equipment, we envision a cased micropile system with a minimum grout bulb diameter of 7.625 inches may be used. The load-supporting capacity (tension) of the micropile would be derived primarily from skin friction between the micropile bonded zone and the surrounding saprolite soils and highly to moderately weathered soft to medium hard basalt rock formation. The micropile capacities and recommendations are summarized in the following tables.

AXIAL (TENSION) LOAD CAPACITIES OF MICROPILE	
Extreme Event Limit State (kips)	Strength Limit State (kips)
255	175

CASED MICROPILE SYSTEM RECOMMENDATIONS	
Micropile Outside Diameter of Casing	7.625 inches minimum
Micropile Casing Thickness	0.430 inches minimum
Micropile Unbonded Length	10 feet

CASED MICROPILE SYSTEM RECOMMENDATIONS	
Diameter of Micropile Bonded Length	7.625 inches minimum
Micropile Bonded Length (Hilo Abutment)	30 feet minimum
Micropile Total Length (Hilo Abutment)	~40 feet from the bottom of the pile cap
Micropile Bonded Length (Honokaa Abutment)	45 feet minimum
Micropile Total Length (Honokaa Abutment)	~55 feet from the bottom of the pile cap
Center Reinforcing Bar (Full Depth)	1.75-Inch Grade 150 ksi Bar
Grout Minimum Compressive Strength	5,000 psi (water-cement ratio of 0.40 or less)

To facilitate the micropile drilling and ensure the quality of the grouting, we recommend advancing the steel casing to the bottom of the micropile during the drilling operation. The steel casing may be withdrawn during the grouting operation while a minimum of 5 feet of grout head is maintained above the bottom of the casing at all times. The casing should be withdrawn to above the specified permanent casing tip elevation (minimum 3 feet) and plunged back to the design casing tip elevation to ensure proper grout cover around the permanent casing.

3.2.1 Lateral Load Resistance

The lateral load capacity of a battered micropile will depend on the vertical load capacity and the batter angle. In this case, the micropiles will be subjected to a tension load. Based on the extreme event axial capacity and a batter of four horizontal to twelve vertical (4H:12V), a lateral load resistance of up to 120 kips per micropile may be used to resist the lateral load acting on the bridge structure.

3.2.2 Micropile Load Test Program

It should be noted that the bond stress between the grout bulb and the soil is highly dependent on the drilling procedures and the grouting methods employed by the contractor to install the micropile. Therefore, the bond stress between the grout bulb and the soil may vary between different contractors and micropile foundation systems. In order to determine whether the contractor’s methods of micropile installation are adequate and to determine the ultimate axial load capacity, we recommend performing one pre-production tension load test on a sacrificial micropile at the Hilo abutment and one pre-production tension load test on a sacrificial micropile at the Honokaa abutment location for a total to two pre-production load tests for the project. In general, the purpose of the pre-production load tests on a micropile is to fulfill the following objectives:

- To examine the adequacy of the methods and equipment proposed by the contractor to install the micropiles to the depths required.
- To confirm or modify the estimated minimum depth of the micropiles by determining the ultimate grout-to-soil bond stress.
- To assess the contractor's method of drilling and grouting.

In general, the pre-production load tests should be performed in accordance with ASTM D3689, Standard Test Methods for Deep Foundations Under Static Axial Tensile Load. Based on experience, we believe the load test should be conducted no earlier than 7 days after completion of the micropile installation to allow the grout adequate time to cure.

The load test micropile should be loaded gradually to the maximum test load of at least 380 kips. The pre-production load test is an integral part of the design of the micropile foundation system. Therefore, we recommend a Geolabs representative observe the pre-production load tests.

In addition to the pre-production load tests, we also recommend performing pullout tests (proof tests) on selected micropiles during construction to confirm the load-carrying capacity of the installed micropiles. We recommend testing a minimum of 2 production micropiles at each abutment for pullout. The pullout tests should consist of subjecting the micropile to at least 255 kips. The micropile should be loaded in 25-kip load increments, and each load should be held for at least 5 minutes. The maximum test load should be held for a minimum of 10 or 60 minutes, depending on the recorded movements of the tested micropile. Pullout test on the selected micropiles is an integral part of the design of the micropile foundation system. Therefore, we also recommend conducting the pullout tests under the observation of a Geolabs representative.

Due to the specialized nature of the micropile foundation construction, observation and testing of the micropile foundation system should be designated a "Special Inspection" item. Therefore, a Geolabs representative (Special Inspector) should be present to observe the geotechnical aspects of the micropile foundation installation and testing.

3.2.3 Micropile Construction Considerations

A specialty contractor experienced in the construction of a micropile foundation system (minimum five projects) should perform the installation of the micropiles. Saprolite and basalt formation were encountered in the borings within the embedment depths of the micropiles. The micropile contractor should anticipate hard drilling conditions during micropile construction.

It should be noted that the bond stress between the grout bulb and the soil is highly dependent on the drilling procedures and the grouting methods employed by the contractor to install the micropile. Therefore, the bond stress between the grout bulb and the soil may vary considerably between different contractors and micropile foundation systems.

Due to the specialized nature of the micropile foundation construction, observation, and testing of the micropile foundation system should be designated as a "Special Inspection" item. Therefore, a Geolabs representative (Special Inspector) should be present to observe the geotechnical aspects of the micropile foundation installation and testing.

3.3 Soil Nails

Based on the information provided, we understand it is desired to retain the existing soil within the space between the abutment columns at both ends of the bridge. Therefore, we recommend constructing a soil-nailed retaining wall to retain the existing soil between the abutment columns. Construction of the permanent soil-nailed wall system should be performed by a specialty contractor experienced in the construction of soil-nailed walls. Due to the specialized nature of the soil-nailed wall construction, a Geolabs representative should be present to observe the geotechnical aspects of the soil-nailed wall and test the soil nails.

Items pertaining to the permanent soil nailed wall system are addressed in the subsequent subsections and include the following:

1. Soil Nailed Wall
2. Soil Nail Installation
3. Soil Nail Testing
4. Shotcrete Facing
5. Drainage

3.3.1 Soil Nailed Wall

The soil-nailed wall system consists of a series of individual reinforcing bars grouted into drilled holes used to stabilize the near vertical slope. Design of the soil-nailed wall system will need to consider both the internal and external stability of the reinforced mass. The design of the internal stability includes establishing the size, spacing, orientation, and length of the grouted reinforcing bars. The external stability includes slope stability of the reinforced mass.

The soil nails should be installed by drilling a minimum 6-inch diameter hole with an inclination of approximately 15 degrees from horizontal. The soil nail bar should consist of ASTM A615 Grade 75 threaded bar with a minimum bar diameter of 1.0 inches. We anticipated that the existing subsoil at the project site may be very corrosive. Therefore, we recommend using a double corrosion protection system for the nails. Galvanized or epoxy coated bars surrounded by neat cement grout or sand-cement mixture with a minimum 28-day compressive strength of 4,000 pounds per square inch (psi) may be considered.

Based on our soil nail analysis, we recommend using a design embedment nail length of 30 feet for the nails extending into the fill and residual/saprolite soils encountered in our field exploration. The soil nails should be spaced 5 feet on-center horizontally and vertically. The first nail should be installed about 2 feet below the bottom of the existing abutment beam.

3.3.2 Soil Nail Installation

Potentially difficult drilling conditions may be encountered during the installation of the soil nails due to the potential presence of medium hard to hard rock (in the form of relatively unweathered cobbles and boulders) within the residual and saprolite soils. In addition, utilizing a temporary casing may be required to maintain an open hole for the soil nail installation when encountering zones of very moist, medium stiff soils.

3.3.3 Soil Nail Testing

Due to the limited number of soil nails that will be installed, we believe the performance of pre-production verification testing on sacrificial soil nails is not needed. Proof tests should be performed on at least 10 percent of the production soil nails or a minimum of one proof test at each abutment during construction to confirm the bond stresses used in the design.

The proof tests should consist of subjecting the soil nail to at least 133 percent of the design load of 24 kips, and the load should be held for at least 10 minutes (until stable). The proof test nails may be incorporated into the permanent soil-nailed wall, provided the nail satisfies the test criteria. Pullout tests on the soil nails are integral parts of the design of the soil-nailed wall system. Therefore, a Geolabs representative should observe the pullout tests.

3.3.4 Shotcrete Facing

Shotcrete placement should be performed by an experienced nozzleman certified as a nozzleman for shotcrete placement by the American Concrete Institute (ACI). Prior to production shotcreting, it is recommended that unreinforced test panels (4-foot by 4-foot size by 4-inch-thick panels) of shotcrete be constructed for inspection.

3.3.5 Drainage

The soil-nailed wall should be well-drained to reduce the potential for the build-up of hydrostatic pressures. A drainage system consisting of 2-foot wide strips of a prefabricated drainage composite product should be installed on the face of the slope before the application of the shotcrete facing. The prefabricated drainage composite product should be installed extending from the top of the slope to the base of the slope and be hydraulically connected to weep holes at the base of the wall. In addition, the drainage strips should be spaced a minimum of about 8 feet on-center.

3.4 Site Preparation

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

Soft and yielding areas encountered during clearing and grubbing below areas designated to receive fill and/or future improvements should be over-excavated to expose firm natural material, and the resulting excavation should be backfilled with well-compacted fill. The excavated soft soils should not be reused as fill materials and should be properly disposed of off-site.

After clearing and grubbing, the exposed subgrades and areas designated to receive fills should be scarified to a depth of about 8 inches, moisture-conditioned to above the optimum moisture content, and recompacted to a minimum of 90 percent relative compaction. Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density as determined by ASTM D1557. Optimum moisture is the water content (percentage by dry weight) corresponding to the maximum dry density.

In general, the excavated on-site materials should be suitable for use as general fill materials, provided that the maximum particle size is less than 3 inches in the largest dimension. General fills and backfills should be moisture-conditioned to above the optimum moisture content, placed in level lifts not exceeding 8 inches in loose thickness, and compacted to at least 95 percent relative compaction.

Imported materials, if required, should consist of select granular fill such as crushed basalt. The select granular fill should be well-graded from coarse to fine with no particles larger than 3 inches in the largest dimension. The material should have a California Bearing Ratio (CBR) value of 20 or higher and a swell potential of 1 percent or less when tested in accordance with AASHTO T193 (ASTM D1883). The material should also contain between 10 and 30 percent particles passing the No. 200 sieve. Imported fill materials should be tested for conformance with these recommendations prior to delivery to the project site for the intended use. Select granular fills should be moisture-conditioned to above the optimum moisture content, placed in level lifts not exceeding 8 inches in loose thickness, and compacted to at least 95 percent relative compaction. Imported fill materials should be tested and approved prior to delivery to the project site for the intended use.

3.5 Cut and Fill Slopes

Based on the subsurface conditions encountered in the borings, it appears that permanent cut slopes near the existing ground surface would expose the soil-like materials (residual soils and saprolite). In general, permanent cut slopes exposing the soil-like materials may be designed with a slope inclination of 2H:1V or flatter. Where cut slopes expose the dense basalt formation, the cut slope may be steepened to a slope inclination as steep as 0.5H:1V, if desired. Cavities that may be exposed on the cut slope face should be backfilled and grouted. We recommend that cut slopes exposing soil-like materials be immediately protected by appropriate slope planting or other means to reduce the potential for erosion of the exposed soils.

Permanent fill slopes constructed with either general fill materials or imported fill materials may be designed with a slope inclination of 2H:1V or flatter. Fills placed on slopes steeper than 5H:1V should be keyed and benched into the existing slope to provide stability for the new fill against sliding. The filling operations should start at the lowest point and continue up in level horizontal compacted layers in accordance with the above fill placement recommendations. Fill slopes should be constructed by overfilling and cutting back to the design slope ratio to obtain a well-compacted slope face. Water should be diverted away from the tops of slopes, and slope planting should be provided as soon as possible to reduce the potential for significant erosion of the finished slopes.

3.6 Design Review

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

3.7 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 Inspections":

1. Review of micropile and soil nail installation submittals
2. Observation of the load test micropiles installation
3. Observation of the micropile load testing
4. Observation of the production micropile installation
5. Observation of the production soil nail installation
6. Observation of the micropile and soil nail proof testing
7. Observation of the subgrade soil preparation
8. Observation of fill placement and compaction

A Geolabs representative should monitor 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