SECTION 3. DISCUSSION AND RECOMMENDATIONS

Our field exploration at the project site generally encountered a thin surface fill and/or alluvial soils layer about 0.5 to 4 feet thick underlain by beach deposit, alluvium, clinker materials, and basalt rock formation extending to the maximum depth explored of about 71.5 feet below the existing ground surface. The surface fill layer consisted of about 7 and 8 inches of asphaltic concrete in paved areas and about 0.5 to 3 feet of medium dense to dense silty sand and sandy gravel. The surface alluvial soil layer, about 0.5 to 1 foot thick, consisted of medium dense silty sand. Beach deposit consisting of loose to medium dense poorly graded sand was encountered at depths of about 0.5 to 18 feet below the existing ground surface. Beach deposit was not encountered in the borings drilled along approximately the northern half of the project site. Below the beach deposit, alluvium about 9 feet thick, consisting of soft to hard silty clay and clayey silt with cobbles and boulders, was encountered and underlain by interbedded layers of basalt formation and clinker materials to the maximum depth explored of about 71.5 feet below the existing ground surface. Basalt formation encountered ranged from hard to very hard and moderately to slightly weathered. Clinker materials encountered generally consisted of medium dense to very dense silty/sandy gravel and silty sand.

We encountered groundwater in the drilled borings at depths of about 9.9 to 12.3 feet below the existing ground surface at the time of our field exploration. The groundwater levels measured generally correspond to about Elevations +0.7 to +2.2 feet MSL, respectively. Due to the proximity of the project site to the Pacific Ocean, groundwater levels can fluctuate depending on tidal fluctuations, storm surge conditions, seasonal precipitation, groundwater withdrawal and/or injection, and other factors.

Based on the information provided, we understand that the relatively heavy structural load demands will require supporting the new bridge on a deep foundation system, such as cast-in-place concrete drilled shafts. The drilled shaft foundations would derive support primarily from adhesion between the drilled shaft and the hard to very hard basalt formation and medium dense to very dense clinker materials encountered in our borings drilled. Based on the anticipated subsurface soil/rock conditions and structural load demands provided, drilled shafts with diameter of 3 feet and embedment lengths varying from 48 to 54 feet are analyzed and recommended for the new bridge foundations. Structural load demands for drilled shafts supporting the proposed wingwalls were not available at the time of this report preparation. Additional analysis and recommendations for these drilled shafts will be provided when structural demands become available.

Retaining walls (not structurally connected to the bridge structure) may be supported on a shallow footing foundation bearing on the recompacted on-site soils. In the event that soft soils are encountered at the footing subgrade elevations, the exposed soft soils within the limits of the footing foundations should be removed and replaced with compacted fills.

Based on the information provided, we understand that site grading consisting of both cut and fill are required for the proposed project. In general, we anticipate the excavations during site grading operations likely will encounter surface fills, alluvial soils and/or beach deposits. The excavated materials may be used as a source of general fill and backfill materials provided that the materials are processed to meet the gradation requirements discussed herein. Cut and fill slopes should be designed with a maximum inclination of three horizontal to one vertical (3H:1V) or flatter.

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

3.1 Drilled Shaft Foundations

Based on the information provided and the anticipated subsurface conditions, we believe drilled shaft foundations with a nominal diameter of 3 feet may be used to support the abutments and the center pier of the new bridge structure at Lauhulu Stream. The drilled shaft foundations would derive support primarily from adhesion between the drilled shaft and the basalt formation and clinker materials encountered in our borings. It should be noted that scour evaluation and protection should be considered and provided for the drilled shaft foundations.

It is our understanding that drilled shaft foundations will also be used as foundation supports for the proposed wingwalls. Structural demands for these drilled shafts were not available at the time of this report preparation. Additional analysis and recommendations will be provided when the structural demands become available.

Based on our engineering analyses and the above assumptions, we recommend using drilled shafts with the following compressive load capacities for the strength limit state based on Load and Resistance Factor Design (LRFD) methods for design of the new bridge provided in the table below.

SUMMARY OF COMPRESSIVE AXIAL CAPACITIES FOR INDIVIDUAL DRILLED SHAFTS					
Shaft Location	**Shaft Diameter** (feet)	**Shaft Length*** (feet)	**Drilled Shaft Tip Elevation** (feet MSL)	**Compressive Load Capacity Per Drilled Shaft** (kips)	
				Extreme Event Limit State	**Strength Limit State**
Abutment No. 1	3	52	-49	1384	692
Abutment No. 2	3	48	-45	1352	676
Center Pier	3	54	-51	1522	761
*Shaft length is based on design shaft cutoff elevation at +3 feet MSL.					

In general, we anticipate that the drilled shafts with a minimum spacing of 4 times the diameter of the shaft measured from center-to-center will be provided. Therefore, the effect of group action was not considered in our axial load analyses. For the strength limit state, a resistance factor of 0.50 has been applied to the extreme event limit state capacities for design of the drilled shaft foundations.

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. Test 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 structure, 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.

Due to the relatively close spacing of the drilled shaft foundations, the effect of group action was considered in our lateral load analyses by including an efficiency factor of p-multiplier in the direction of loading. Results of the generated non-linear "p-y" curves are summarized and presented on Plates 4.1 through 4.12.

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 and clinker materials. 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 our borings at a relatively high elevation. Therefore, we believe that the contractor should be prepared to contain the groundwater during drilled shaft construction. In addition, beach sand deposits were encountered in our drilled borings, therefore, temporary casing to prevent caving-ins of the beach sand will be necessary during the 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 rock 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 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 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.

In addition, the concrete should be placed promptly after drilling (within 24 hours after substantial completion of the holes) to reduce the potential for softening of the sides of the drilled holes. Furthermore, drilling adjacent to a recently constructed shaft (within 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 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 Test Shaft Program

A test shaft program is normally required and highly recommended for bridge foundation projects. Considering the diameter and structural load capacities of the drilled shafts, we recommend performing a test shaft program, including the performance of an instrumented load test at the bridge site 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 confirm or modify the estimated tip elevations of the drilled shafts.
- 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 that the test shaft program consist of drilling one 3-foot diameter test shaft extending to a depth of about 75 feet below the existing ground surface. The location of the test shaft should be near, but outside of, the planned Abutment No. 2 foundation location. In general, the load test shaft should be structurally reinforced and instrumented with embedment strain gauges for load testing purposes. The embedment strain gauges should be placed starting from an elevation of about 5 feet above and below the load cell and subsequently at the pre-determined intervals, as shown on the Drilled Shaft Load Test Detail, Plate 5.

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 a bi-directional axial load test on the reinforced load test shaft using an expandable base load cell (Osterberg Load Cell). The expandable base load cell will need to be installed within the load test shaft reinforcing cage prior to lowering the cage in place.

The drilled shaft load test should be performed in general accordance with the Quick Load Test Method of ASTM D1143. In general, the load test shaft should be loaded at increments of about 50 to 100 kips and should be held for a minimum of 12 hours at or near failure to evaluate the potential for creep effects. The load test shaft should be loaded to failure to evaluate the ultimate side shear resistance of the shaft. Installation of the expandable base load cell and embedment strain gauges, performance of the bi-directional axial load test, and analyses of the load test data should be performed by a qualified professional experienced in these types of load testing procedures.

Considering the specialized nature of the test shaft program, we recommend that a Geolabs representative be present during the test shaft program to evaluate the contractor's method of drilled shaft installation and to evaluate the subsurface materials encountered. In addition, Geolabs should observe the instrumented load test on the reinforced load test shaft. It should be noted that the drilled shaft design was developed from our analysis using limited field exploration data. Therefore, observation of the drilled shaft installation operations by Geolabs is a vital part of the foundation design to confirm our design assumptions.

3.1.5 Non-Destructive Integrity Testing

Based on the critical nature of the drilled shaft foundations for the new bridge abutments and center pier, we recommend conducting non-destructive integrity testing on the test 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 four access tubes into the concrete of the 3-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.

3.2 Structural Approach Slabs

To reduce the potential for appreciable abrupt differential settlements between the drilled shaft supported bridge structure and the compacted backfills behind the abutment structures, we recommend providing structure approach slabs at the abutment locations. In general, the structure approach slabs should be at least 10 feet long.

The structure approach slabs should be supported on a minimum of 8 inches of aggregate subbase course placed on a prepared subgrade. The subgrade should be scarified to a depth of about 8 inches, moisture-conditioned to above the optimum moisture content, and compacted to no less than 95 percent relative compaction. The aggregate subbase course should also be moisture-conditioned to above the optimum moisture content and compacted to at least 95 percent relative compaction. Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density 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.

3.3 Retaining Walls

Based on the information provided, we understand that retaining structures, including the abutment wall and wingwalls, should be designed to resist the lateral earth pressures due to the adjacent soils and surcharge effects. Parameters for the design of foundations for the abutment walls supported on drilled shafts have been provided in the "Drilled Shaft Foundations" section herein. Structural load demands for drilled shafts supporting the proposed wingwalls were not available at the time of this report preparation. Additional analysis and recommendations for these drilled shafts will be provided when structural demands become available. Design of retaining walls (not structurally connected to the bridge structure) should be based on the parameters presented in the following subsections.

3.3.1 Retaining Wall Foundations

Based on the subsurface conditions encountered during our field exploration, we believe that conventional retaining walls may be supported by a shallow footing foundation bearing on recompacted on-site soils consisting of loose to medium dense sand and/or stiff silty clay.

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			
	Extreme Event Limit State	**Strength Limit State**	**Service Limit State**
Bearing Pressure (psf)	9,000	4,050	3,000
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. 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.

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.

3.3.2 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.

LATERAL EARTH PRESSURES			
Backfill Condition	**Earth Pressure Component**	**Active** (pcf)	**At-Rest** (pcf)
Level Backfill	Horizontal	37	55
	Vertical	None	None
Maximum 2H:1V Sloping Backfill	Horizontal	56	73
	Vertical	14	18

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.

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 wall should be considered in the design. For uniform surcharge stresses imposed on the loaded side of the wall, a rectangular distribution with a uniform pressure equal to 33 percent of the vertical surcharge pressure acting over the entire height of the wall, which is free to deflect (cantilever), may be used in the design. For walls that are restrained, a rectangular distribution equal to 50 percent of the vertical surcharge pressure acting over the entire height of the wall may be used for design. Additional analyses during design may be needed to evaluate the surcharge effects of point loads and line loads.

3.3.3 Dynamic Lateral Earth Forces

Dynamic lateral earth forces due to seismic loading will need to be considered in the design of the retaining wall 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 wall structures. Dynamic lateral earth forces due to seismic loading (a_{max} = 0.236g) may be estimated by using $4.2H^2$ pounds per linear foot of wall length for level backfill conditions, where H is the height of the wall in feet. It should be noted that the dynamic lateral earth forces provided assume that the wall will be allowed to move laterally by up to about 1 to 2 inches in the event of an earthquake. 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.

If the estimated amount of lateral movement is not attainable or the retaining structure is restrained, the retaining structure should be designed with higher dynamic lateral forces for a restrained condition. For a restrained condition (less than 0.5 inches of lateral movement), dynamic lateral forces due to seismic loading may be estimated using $15.7H^2$ pounds per linear foot of wall (H measured in feet) for level backfill conditions.

3.3.4 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.

3.4 Site Grading

Based on the information provided, we anticipate that cuts of about 5 feet deep and fills up to about 8 feet high may be required for the proposed project. 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
5. Cut and Fill Slopes

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.

3.4.1 Site Preparation

At the onset of earthwork, areas within the contract grading limits should be thoroughly cleared and grubbed. Vegetation, debris, demolished man-made 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.

3.4.2 Fills and Backfills

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

In general, the on-site soil 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 reused 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.

3.4.3 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 the AASHTO T180 (or ASTM D1557) test procedures. Optimum moisture is the water content (percentage by dry weight) corresponding to the maximum dry density.

Imported fill materials should be moisture-conditioned to above the optimum moisture content, placed in level lifts of about 8 inches in loose thickness, and compacted to a minimum of 90 or 95 percent relative compaction, as appropriate. Aggregate base course and subbase materials should be moisture-conditioned to above the optimum moisture content, placed in level lifts not exceeding 8 inches in loose thickness, and compacted to a minimum of 95 percent relative compaction.

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.

3.4.4 Excavation

The project site generally is underlain by a thin surface fill/alluvial soil layer over beach deposits, recent alluvium, clinker materials and hard basalt formation. It is anticipated that the fills, alluvial soils, and beach deposits near the ground surface may be readily excavated with normal heavy excavation equipment, such as excavators, and ripped with large bulldozers. However, cobbles and boulders are frequently encountered in fills and alluvial soil deposits and should be expected. Excavations that encounter cobbles and boulders within the on-site soils and deeper 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.

3.4.5 Cut and Fill Slopes

Based on the anticipated grading and our field exploration, permanent cut and fill slopes for the drainage and safety improvements project should be designed with an inclination of three horizontal to one vertical (3H:1V) or flatter. Fills that are to be placed on existing ground steeper than 5H:1V should be benched. The filling operation 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. In addition, slope planting or other means of slope protection should be provided as soon as possible to reduce the potential for significant erosion of the finished slopes.

3.5 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. 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.

3.6 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.

3.7 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 Inspection":

1. Observation of the test drilled shaft installation and testing
2. Observation of the production drilled shaft installation
3. Observation of shallow foundation excavations
4. Observation of the subgrade soil preparation
5. 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