SECTION 3. DISCUSSION AND RECOMMENDATIONS

Our field exploration indicates the surface conditions along the majority of the project alignment consist of 9.0 to 12.0 inches of concrete pavement underlain by 6 to 10 inches of base course material. We encountered 4.0 to 11.0 inches of asphaltic concrete pavement overlying 12 to 24 inches of aggregate base material in our borings near the end of the project alignment.

Variable subsurface conditions were encountered below the pavement sections along the project alignment, generally consisting of medium dense to very dense and medium stiff to very stiff fills, soft and medium dense to dense alluvium, and weathered to hard basalt formations extending to the maximum depth explored of about 16.5 feet below the existing ground surface. We did not encounter groundwater in the drilled borings during our field exploration, except for Boring No. 11, where groundwater was encountered at about 13-foot depth.

New concrete barrier walls, end posts, an impact attenuator, and light poles will be constructed along the project alignment. Based on the subsurface soil conditions encountered along the project alignment, we believe that the new structures may be supported on shallow foundations bearing on either on-site soil or rock encountered along the project alignment. Consideration may also be given to the use of a single drilled shaft to support each of the new light pole structures. In general, retaining structures, including end posts and barrier walls should be designed to resist the lateral earth pressures due to the adjacent soils and surcharge effects.

Detailed discussions of these items and our geotechnical recommendations for design of the new structures for the project are presented in the following sections.

3.1 Structure Foundations

We understand that several new concrete end posts, concrete barrier walls, an impact attenuator, and light poles will be constructed along the project. Locations of the new structures are provided in the table below.

NEW STRUCTURE	NEARBY CROSSING FEATURE	STRUCTURE LOCATION	
		Site ID	Station Nos.
Guardrail to Wall and Concrete Barrier	1st and 2nd Avenue Pedestrian Overpass	1IB	55+00 to 60+00
Concrete Barrier	2nd Avenue Pedestrian Overpass	2OB	59+00 to 60+00
Guardrail Connection	4th Avenue Pedestrian Overpass	2IB	65+80 to 66+40
Concrete Barrier	4th Avenue Pedestrian Overpass	3OB	66+10 to 67+10
Concrete Barrier	4th Avenue Pedestrian Overpass	3IB	66+10 to 67+10
Guardrail to Abutment	6th Avenue Overpass	4OB	76+70
Guardrail to Abutment	6th Avenue Overpass	5IB	77+10
Guardrail to Abutment	7th Avenue Overpass	5OB	81+40
Guardrail to Wall and Abutment	Between 6th Avenue and 8th Avenue	6IB	80+00, 81+50, 82+90, 84+90
Railings Structure	Between 9th Avenue and 11th Avenue	6OB	94+60 to 95+30
Railings Structure	Between 9th Avenue and 11th Avenue	7IB	94+60 to 95+25
Guardrail to Abutment	Koko Head Avenue Overpass	7OB	108+40
New Lights	Between Koko Head Avenue and Waialae On-Ramp	8OB & 9IB	109+00 to 115+00
New Lights	Between Koko Head Avenue and Waialae On-Ramp	9OB & 10IB	110+00 to 124+50
New Lights	Between Koko Head Avenue and Waialae On-Ramp	10OB & 11IB	124+50 to 132+70
Impact Attenuator	Between Koko Head Avenue and Waialae On-Ramp	10OB	128+40 to 129+30
IB – Inbound (Westbound) OB – Outbound (Eastbound)			

Design of the new structures foundations should be based on the parameters presented in the following sections.

3.1.1 Shallow Foundation

Based on the information provided, we understand that end posts, barrier walls, an impact attenuator, and light poles will be required for the project. We believe that shallow foundations bearing on on-site soil and rock anticipated at the project site may be utilized for support of the planned structures. Based on our analysis, we believe that the following values may be used to evaluate the bearing support, sliding resistance, and passive pressure resistance of the planned structures based on LRFD design methods.

SHALLOW FOUNDATIONS			
Soil Condition			
Description	**Extreme Event Limit State**	**Strength Limit State**	**Service Limit State**
Bearing Pressure	9,000 psf	4,500 psf	3,000 psf
Coefficient of Sliding Friction	0.35	0.30	N/A
Passive Pressure Resistance	270 pcf	135 pcf	N/A
Rock Condition			
Description	**Extreme Event Limit State**	**Strength Limit State**	**Service Limit State**
Bearing Pressure	15,000 psf	6,750 psf	5,000 psf
Coefficient of Sliding Friction	0.60	0.48	N/A
Passive Pressure Resistance	640 pcf	320 pcf	N/A

The bottom of the footing excavations in the soil condition areas should be scarified to a depth of at least 8 inches, moisture-conditioned to at least 2 percent above the optimum moisture, and recompacted to at least 95 percent relative compaction to provide a relatively firm and smooth bearing surface prior to the placement of reinforcing steel or concrete. Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same soil established in accordance with ASTM D1557. Optimum moisture is the water content (percentage by dry weight) corresponding to the maximum dry density.

Soft and/or loose materials encountered at the bottom of the footing excavations should be over-excavated to expose the underlying firm materials. The over-excavation may be backfilled with the on-site soils compacted to a minimum of 95 percent relative compaction, or the bottom of footing may be extended down to the underlying competent materials. For footings bearing on new compacted fills, the bottom of the footing excavations should also be recompacted to a minimum of 95 percent relative compaction prior to the placement of reinforcing steel or concrete.

Imported materials required for the project 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. In addition, the fill material should contain 10 to 30 percent fines (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. The material should be moisture-conditioned to above the optimum moisture, placed in level lifts not exceeding 8 inches in loose thickness, and compacted to at least 95 percent relative compaction.

For footings bearing directly on rock, the bottom of the footings should be relatively level and free of sharp points that may cause a concentration of loading resulting in potential distress to the foundation system. A thin layer of compacted aggregate subbase material may be placed at the bottom of footings to create a relatively level surface to provide uniform loading.

In general, the bottom of foundations should be embedded a minimum of 18 inches below the lowest adjacent finished grades. Foundations next to utility trenches or easements should be embedded below a 45-degree imaginary plane extending upward from the bottom edge of the utility trench, or they should extend 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.

For sloping ground conditions, the bottom of footing should extend deeper to obtain a minimum 6-foot setback distance measured horizontally from the outside edge of the footing to the face of the slope. Footings oriented parallel to the direction of the slope should be constructed in stepped footings.

Based on service limit state bearing pressures of 3,000 and 5,000 pounds per square foot (psf) for soil and rock conditions, respectively, we estimate that foundation settlements under the anticipated design loads for foundations bearing on the recompacted soil and rock to be less than 1 inch.

Lateral loads acting on the structures may be resisted by friction between the base of the foundation and the bearing soil and by passive earth pressure developed against the near-vertical faces of the embedded portion of the foundation. The values presented in the tables above, expressed in pounds per square foot per foot of embedment (pcf), may be used to evaluate the passive pressure resistance for footings embedded and bearing on the stiff soil or rock. Unless covered by pavements or slabs, the passive resistance in the upper 12 inches should be neglected.

A Geolabs representative should monitor footing excavations prior to placement of reinforcing steel and concrete to confirm the foundation soil/rock conditions.

3.1.2 Drilled Shaft Foundation

The following structural loading information for the new light pole structures was provided to our office and used in our engineering analyses.

DRILLED SHAFT FOUNDATIONS		
Light Pole Loading Information		
Load	Wind Load	Barrier Crash Load
Axial	40 kips	40 kips
Shear	1.5 kips	54 kips
Overturning Moment	35 kip-feet	243 kip-feet

Based on our analysis, we believe that a single cast-in-place concrete drilled shaft may be used to support each light pole. The cast-in-place concrete drilled shaft would derive vertical support from friction between the concrete shaft and the surrounding weathered and hard basalt rock formation.

Based on our field exploration, structural loading provided, and our engineering analyses, we recommend using a 2.0-foot diameter drilled shaft extending to a minimum depth of 10 feet below the bottom of the footing to support each light pole structure. The drilled shaft may be designed with an allowable compressive load capacity of 80 kips. The allowable compressive load capacity for the drilled shaft is for dead-plus-live loads. The compressive load capacity may be increased by one-third (1/3) when considering transient loads, such as wind or seismic forces. A factor of safety of 2.0 was used for the allowable compressive load capacity.

3.1.2.a Uplift Load Resistance

In general, uplift loads may be resisted by a combination of the dead weight of the drilled shaft and by shear along the shaft surface and the adjacent basalt rock formation. Considering that the drilled shafts are designed based on adhesion between the shaft and the surrounding weathered and hard basalt rock formation, we believe that an ultimate uplift load capacity of 160 kips may be used in the design. This value includes the weight of the drilled shaft and should be used for transient loads only.

For sustained uplift loads acting on the foundations, the provided uplift load capacity should be reduced by a factor of safety of 3.0. The project structural engineer should check the structural capacity of the shaft member in tension.

3.1.2.b Lateral Load Resistance

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

Based on the anticipated lateral load acting at the top of the drilled shaft, the lateral deflection at the top of the drilled shaft, the maximum induced moment, the maximum induced shear, and the depth below the existing ground surface at which the maximum moment and shear would act for the free-head and fixed-head conditions are presented in the table below.

SUMMARY OF LATERAL LOAD ANALYSES

Pile Head Connection	**Lateral Deflection** (inches)	**Max. Induced Moment** (kip-ft)	**Depth to Max. Moment** (feet)	**Max. Induced Shear** (kips)	**Depth to Max. Shear** (feet)
Free	0.30	504	5.2	188	6.8
Fixed	0.04	177	0.0	54	0.0

NOTE: Analyses based on concrete compressive strength of 4,000 psi.

3.1.2.c Foundation Settlement

Settlement of the drilled shaft foundation will result from elastic compression of the shaft and subgrade response of the foundation embedded in the weathered and hard basalt formation encountered at the site. The total settlement of the drilled shaft is estimated to be less than 0.5 inch. We believe that a significant portion of the settlement will be elastic and should occur as the loads are applied.

3.1.2.d Drilled Shaft Construction Considerations

The performance of the shaft will significantly depend upon the contractor's method of construction and construction procedures. As a result, a Geolabs representative should be present to observe the installation of the drilled shaft during construction. In our opinion, the following may have a significant impact on the effectiveness and cost of the drilled shaft foundation.

The load carrying capacity of the drilled shaft depends, to a large extent, on the friction between the shaft and the surrounding soils/formation. Therefore, proper construction techniques are important. The contractor should exercise care in drilling or excavating the shaft hole and in placing concrete in the hole.

Based on our field exploration, difficult drilling conditions likely will be encountered at the project site and should be expected. The drilled shaft subcontractor will need to have the appropriate equipment and tools to drill through hard to very hard basalt rock formations.

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 residual soil and rock formation. Concrete should be placed in a suitable manner to reduce the potential for segregation of the aggregates from the concrete mix. In addition, concrete should be placed promptly after drilling (within 24 hours after drilling of the holes) to reduce the potential for softening/unraveling the sides of the drilled holes.

A Geolabs representative should be present at the project site to observe the drilling and installation of drilled shafts during construction. Although the drilled shafts are primarily designed 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 in accordance with Section 1704 of the International Building Code (2006).

3.2 Retaining Structures

Based on the project drawings, retaining and impact structures will be required for the project. Based on the subsurface conditions encountered during our field exploration, the following general guidelines may be used for the design of the retaining and impact structures at the project site. In general, we believe retaining structures may be designed in accordance with the recommendations and parameters presented in the "Shallow Foundations" section herein. In addition, retaining structure foundations should be a least 18 inches wide and should be embedded a minimum of 24 inches below the lowest adjacent finish grades.

3.2.1 Static Lateral Earth Pressures

Retaining structures should be designed to resist the lateral earth pressures due to the adjacent soils and surcharge effects caused by loads adjacent to the retaining structures. The recommended lateral earth pressures for design of retaining structures, expressed in equivalent fluid pressures, are presented in the following table.

LATERAL EARTH PRESSURES FOR DESIGN OF RETAINING STRUCTURES			
Backfill Condition	**Earth Pressure Component**	**Active** (pcf)	**At-Rest** (pcf)
Level Backfill	Horizontal	36	53
	Vertical	None	None
Maximum 2H:1V Sloping Backfill	Horizontal	58	73
	Vertical	29	37

Type A Structure Backfill Material conforming to Section 703.20 of the Hawaii Standard Specifications for Road and Bridge Construction, 2005 (HSS) should be used to backfill behind the retaining structures. The backfill behind retaining structures should be compacted to at least 95 percent relative compaction in accordance with HSS. In general, an active condition may be used for gravity retaining walls or walls that are free to deflect by as much as 0.5 percent of the wall height. If the tops of walls are not free to deflect beyond this degree or are restrained, the walls should be designed for the at-rest condition. These lateral earth pressures do not include hydrostatic pressures that might be caused by groundwater trapped behind the walls.

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 uniform pressure equal to 36 percent of the vertical surcharge pressure acting over the entire height of the wall, which is free to deflect (cantilever), may be used in design. For walls that are restrained, a rectangular distribution equal to 53 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.

Lateral impact loads acting into the retained soil on the retaining structures may be resisted by passive earth pressure acting against the near-vertical faces of the wall system. Resistance due to passive earth pressure may be estimated using an equivalent fluid pressure of about 1,000 pounds per square foot per foot of depth (pcf). This assumes that the lateral load acting on the retaining structure is due to vehicular impact into the wall structure retaining a minimum 2H:1V sloping backfill. In addition, it is assumed that backfill behind the structure is well compacted (minimum of 95 percent relative compaction). Unless covered by pavements or slabs, the passive pressure resistance in the upper 12 inches of soil should be neglected.

3.2.2 Dynamic Lateral Earth Pressures

Dynamic lateral earth forces due to seismic loading (amax= 0.224g) may be estimated by using $9.2H^2$ pounds per lineal 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 is allowed to move laterally by up to about 1.5 to 2 inches in the event of an earthquake. The resultant force should be assumed to act through the mid-height of the wall. An appropriately reduced factor of safety may be used when dynamic lateral earth forces are accounted for in the design of the retaining structures.

If the estimated amount of lateral movement is not acceptable, 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 $14.0H^2$ pounds per lineal foot of wall (H measured in feet) for level backfill conditions.

3.2.3 Drainage

Retaining structures should be well drained to reduce the potential for the build-up of hydrostatic pressures. A typical drainage system would consist of a 12-inch wide zone of permeable material, such as drain rock (AASHTO M43 Size No. 67), directly adjacent to the wall with 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 should also be hydraulically connected to a perforated pipe at the base of the wall.

Backfill behind the permeable drainage zone should consist of Type A Structure Backfill Material conforming to Section 703.20 of the HSS (a minimum of 95 percent relative compaction). Unless covered by concrete slabs or pavements, the upper 12 inches of backfill should consist of relatively impervious material to reduce the potential for water infiltration behind the walls. In addition, the backfill below the drainage outlet (or weepholes) should consist of the relatively impervious material to reduce the potential for water infiltration into the footing subgrade. The relatively impervious material should be compacted to not less than 90 percent relative compaction.

3.3 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. This review is necessary to evaluate 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 misinterpretation of our recommendations.

3.4 Post-Design Services/Services During Construction

Geolabs should be retained to provide Geotechnical Engineering Services during construction. The critical items of construction monitoring consist of verifying the assumed subgrade conditions used in the design of the new structure foundations. If the actual exposed subsurface conditions encountered during construction differ from those assumed or considered herein, revisions to the geotechnical recommendations presented herein may be required.

END OF DISCUSSION AND RECOMMENDATIONS