GEOLABS, INC.

Geotechnical Engineering and Drilling Services

TECHNICAL MEMORANDUM

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TO:	Nakamura, Oyama & Assoc.	**FROM:**	Robin M. Lim / Tim Roy
ATTN:	Mr. Calvin Kwan	**NO. OF PAGES:**	12
SUBJECT:	Highway Lighting Improvements, Moanalua Freeway Halawa Heights Off-Ramp to Middle Street Overpass F.A.P. No. NH-H201(005) Geotechnical Recommendations		
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This technical memorandum presents our geotechnical recommendations in support of the Highway Lighting Improvements, Moanalua Freeway, Halawa Heights Off-Ramp to Middle Street Overpass project located in Honolulu on the island of Oahu, Hawaii. The geotechnical recommendations presented herein are based on our experience in the project vicinity and limited subsurface information available.

PROJECT CONSIDERATIONS

The proposed lighting improvements project is located along Moanalua Freeway on the Island of Oahu, Hawaii. The approximate limits of the project discussed in this technical memorandum extend from the vicinity of Halawa Heights Off-Ramp (Mile Post 1.12) to the vicinity of the Middle Street Overpass (Mile Post 4.09) for a distance of about 15,700 linear feet.

The project consists of upgrading the existing lighting system to meet the current State, AASHTO, and Federal lighting standards. We understand new impact barriers will be designed for the project and some of the light poles will be constructed atop the impact barriers. In addition, some of the impact barriers are along the hillside and will function as retaining walls. Also, installation of ITS conduits for the lighting system is anticipated for this project. It should be noted that light poles located within the elevated portions of the highway (viaduct) are specifically excluded from our scope of work for this project.

SUBSURFACE CONDITIONS

Based on available information, the subsurface conditions at the project site may be generalized into four different categories: (1) Soft soil; (2) Stiff Soil; (3) Tuff; and (4) Basalt. We estimate the distribution of the soil condition categories along the project alignment is as follows:

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Moanalua Freeway Stationing		**Estimated Ground Conditions**
From Station (Approx.)	To Station (Approx.)	
205+00	215+00	Stiff Soil
215+00	222+00	Basalt
222+00	255+00	Tuff
255+00	277+50	Stiff Soil
277+50	281+00	Tuff
281+00	298+00	Stiff Soil
298+00	322+50	Soft Soil
322+50	340+50	Stiff Soil
340+50	351+00	Soft Soil
351+00	366+00	Basalt
NOTE: The stations provided in this table are approximations only based on regional geologic maps. Variations from those indicated in this table are likely and should be expected.		

Stiff Soil

In this area, we anticipate dense granular materials and stiff cohesive materials. Based on the surface elevations in this area, groundwater is not anticipated within the foundation excavation depths.

Basalt

In this area, we anticipate a relatively thin layer of fill material overlying hard to very hard basalt. Based on the surface elevations in this area, groundwater is not anticipated within the foundation excavation depths.

Tuff

In this area, we anticipate a relatively thin layer of fill material overlying hard to very hard volcanic tuff. Based on the surface elevations in this area, groundwater is not anticipated within the foundation excavation depths.

Soft Soil

In this area, we anticipate subsurface conditions consisting of loose granular materials and soft cohesive materials. Based on available subsurface information, groundwater in this area may be encountered at shallow depths of about 5 feet or less beneath the existing ground surface.

SEISMIC DESIGN CONSIDERATIONS

Based on the subsurface conditions anticipated at the project site and the geologic setting, the tuff and basalt areas of the project site may be classified from a seismic analysis standpoint as Site Class C, and the soil areas (soft soils and stiff soils) of the project site may be classified from a seismic analysis standpoint as Site Class E in accordance with AASHTO LRFD Bridge Design Specifications (Table No. 3.10.3.1-1), 2012 Edition. Based on these considerations, the following seismic design parameters were estimated and may be used for seismic analysis of the project.

SEISMIC DESIGN PARAMETERS 1,000-YEAR RETURN PERIOD Tuff and/or Basalt Conditions Stiff Soil Conditions	
Parameter	**Value**
Mapped MCE Spectral Response Acceleration, S_S =	0.397 g
Mapped MCE Spectral Response Acceleration, S_1 =	0.109 g
Site Class =	"C"
Site Coefficient, F_a =	1.200
Site Coefficient, F_v =	1.691
Design Spectral Response Acceleration, S_{DS} =	0.476 g
Design Spectral Response Acceleration, S_{D1} =	0.184 g
Design Peak Bedrock Acceleration, PBA (Site Class B) =	0.174 g
Design Peak Ground Acceleration, PGA (Site Class C) =	0.208 g

SEISMIC DESIGN PARAMETERS 1,000-YEAR RETURN PERIOD Soft Soil Condition	
Parameter	**Value**
Mapped MCE Spectral Response Acceleration, S_S =	0.397 g
Mapped MCE Spectral Response Acceleration, S_1 =	0.109 g
Site Class =	"E"

SEISMIC DESIGN PARAMETERS 1,000-YEAR RETURN PERIOD Soft Soil Condition	
Parameter	**Value**
Site Coefficient, F_a =	2.030
Site Coefficient, F_v =	3.473
Design Spectral Response Acceleration, S_{DS} =	0.806 g
Design Spectral Response Acceleration, S_{D1} =	0.378 g
Design Peak Bedrock Acceleration, PBA (Site Class B) =	0.174 g
Design Peak Ground Acceleration, PGA (Site Class E) =	0.332 g

EARTHWORK REQUIREMENTS

Based on the existing topography at the project site, we anticipate earthwork for the project generally will involve only site preparation and backfilling to achieve the finished grades. Items of earthwork that are addressed in the subsequent subsections include the following:

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

Site Preparation

At the on-set of earthwork, areas within the contract grading limits should be cleared and grubbed (where necessary). Vegetation, debris, deleterious materials, and other unsuitable materials should be removed and disposed properly to reduce the potential for contaminating the excavated materials to be used as backfill materials.

After clearing, areas designated to receive improvements 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 (per AASHTO T-180). Relative compaction, in this memorandum, 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. Optimum moisture is the water content (percentage by weight) corresponding to the maximum dry density. Compaction should be accomplished by sheepsfoot rollers, vibratory rollers, or other types of acceptable compaction equipment.

Soft and/or loose, weak, or yielding areas (or cavities) disclosed during site preparation operations should be over-excavated to expose firm ground, and the resulting excavation should be backfilled with granular fill materials (well graded 3-inch minus materials) or aggregate subbase compacted to a minimum 95 percent relative compaction. Saturation and subsequent yielding of the exposed subgrades due to inclement weather and poor drainage may require over-excavation of the soft areas and replacement with compacted fill material.

Excavation

Excavation for this project generally will consist of excavations for foundations, ITS conduits and other infrastructure. Generally, we anticipate the site grading and infrastructure work may involve excavation into the underlying volcanic tuff and basalt. It is anticipated that the surface fill materials may be excavated with normal heavy excavation equipment. However, deeper excavations and excavations into volcanic tuff and/or basalt may require the use of hoerams. We recommend encouraging contractors bidding on this project to examine the site conditions and geotechnical data to make their own prudent interpretation.

Fills and Backfills

In general, the excavated on-site granular fill material may be used as a source of backfill material provided that deleterious materials such as vegetation are removed and over-sized materials greater than 3 inches in maximum dimension are screened. It should be noted that excavated clayey silt soils (or other cohesive soils) should not be used as a source of backfill materials due to the poor pavement support characteristics of the soils.

In the event that imported fill materials are needed for the project construction, imported fill materials should be well graded from coarse to fine with no particles larger than 3 inches in largest dimension and should 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 of 1 percent or less when tested in accordance with AASHTO T-193.

Imported fill materials, if needed, should be tested for conformance with these recommendations prior to delivery to the project site for the intended use. An accredited testing laboratory should test the imported fill materials for conformance with these recommendations prior to delivery to the project site for the intended use.

Fill Placement and Compaction Requirements

Fill and backfill materials should be placed in level lifts not exceeding 8 inches in loose thickness, moisture-conditioned to above the optimum moisture content, and compacted to at least 95 percent relative compaction (AASHTO T-180).

LIGHT POLE FOUNDATIONS

We understand some of the light poles will not be attached to the new impact barriers. Based on this consideration and the anticipated subsurface conditions, the new foundations for standalone light poles generally should consist of 24-inch diameter reinforced concrete drilled shafts extending down to depths of about 10 feet below the ground surface (assuming relatively level ground).

In general, drilled shaft foundations are constructed by drilling a hole down into the bearing strata, placing reinforcing steel, and then pumping high slump concrete to fill up the hole. The result is a cast-in-place concrete drilled shaft for foundation support. Based on the subsurface conditions encountered at the project site, we envision the drilled shaft foundation would derive vertical support primarily from skin friction between the drilled shaft and the surrounding materials. For design of the drilled shaft foundations, we have assumed that a minimum concrete compressive strength of 4,000 pounds per square inch (psi) will be specified and a nominal longitudinal reinforcing steel of about 1% of the cross-sectional area of the drilled shaft will be used.

The load bearing capacities of the drilled shaft will depend largely on the relative density of the soils within the bearing strata. Because local variations in the subsurface materials likely will occur, it is imperative that our representative is present during the shaft drilling operations to confirm the subsurface conditions encountered during the drilled shaft construction and to observe the installation of drilled shaft. 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 subsurface conditions and the foundation design parameters, we anticipate the drilled shaft installation for the project will require an experienced drilled shaft subcontractor to install the drilled shaft foundations. The subsequent subsections address the design and construction of the drilled shaft foundation, which include the following:

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

Lateral Load Resistance

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

Based on the structural loads provided, the following table summarizes the anticipated lateral deflection and induced moment for the new light pole drilled shaft foundations.

SUMMARY OF LATERAL LOAD ANALYSES			
Shaft Length	Lateral Deflection (inches)	Max. Induced Moment (kip-feet)	Depth to Max. Moment (feet)
10 feet	0.24	21.2	1.6
NOTE: Analyses based on concrete compressive strength of 4,000 psi and a minimum of 1% longitudinal steel reinforcement.			

Foundation Settlements

Settlement of the drilled shaft foundation will result from elastic compression of the shaft and subgrade response of the foundation embedded in the subsurface soils. Total settlements of the drilled shafts are estimated to be less than 0.5 inches. 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 shaft 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 foundation.

The load bearing capacities of drilled shaft depend, to a significant extent, on the frictional resistance between the shaft and the surrounding soils. Therefore, proper construction techniques especially during the drilling operations are important. The contractor should exercise care in drilling the shaft hole and in placing concrete into the drilled hole.

The subsurface materials consist of soft soils, stiff soils, tuff, and basalt formation. The soil profile may contain cobbles and boulders; therefore, some difficult drilling conditions may be encountered and should be expected in these subsurface conditions. In addition, very hard drilling conditions into the tuff and basalt formation likely will be encountered at the project site and should be expected. The drilled shaft contractor will need to have the appropriate equipment and tools to drill through the cobbles, boulders, and hard basalt formation, where encountered in the subsurface.

Based on the anticipated subsurface conditions, some loose granular materials and soft soils may be present at the site. Due to the presence of loose granular soils and soft soils, caving-in and/or sloughing of these materials likely will occur during the drilling operations. To reduce the potential for caving-in of the drilled hole, temporary casing of the drilled hole should be required during the drilled shaft installation to maintain the integrity of the drilled hole.

Drilling by methods utilizing drilling fluids may have a significant effect on the supporting capacity of the drilled shaft; therefore, use of drilling fluids for the drilling operations of the drilled shaft should be specifically accepted by Geolabs upon evaluation of the type of drilling fluids proposed.

A low-shrink concrete mix with high slump (7 to 9-inch slump range) should be used to provide close contact between the drilled shaft and the surrounding soils. Due to factors such as seasonal rainfall and perched water, groundwater may be encountered in the drilled holes. Concrete for the drilled shafts 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 hole.

It is imperative for a Geolabs representative to be present at the project site to observe the drilling and installation of drilled shaft during construction. Although the drilled shaft is designed based primarily on skin friction, the bottom of the drilled hole should be relatively free of loose materials prior to placement of concrete. Therefore, it is necessary for Geolabs to observe the drilled shaft installation operations to confirm the assumed subsurface conditions.

IMPACT BARRIER FOUNDATIONS

In general, we recommend supporting the proposed impact barriers on a shallow foundation system bearing on the re-compacted on-site soils. Based on our field exploration, we recommend utilizing the following values to evaluate the bearing support, sliding resistance, and passive pressure resistance of the planned retaining walls based on LRFD design methods.

SHALLOW FOUNDATIONS			
Description	**Extreme Event Limit State**	**Strength Limit State**	**Service Limit State**
Bearing Pressure	10,500 psf	4,700 psf	3,500 psf
Coefficient of Sliding Friction	0.45	0.36	N/A
Passive Pressure Resistance	350 pcf	175 pcf	N/A

In general, foundations should be embedded a minimum of 2 feet below the lowest adjacent finished grades. 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 footings 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.

Based on a service limit state bearing pressure of 3,500 pounds per square foot (psf), we estimate that foundation settlements under the anticipated design loads for the retaining wall foundations bearing on the stiff to very hard and dense fill soils to be less than 1 inch.

Lateral loads acting on the structure 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 table 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 to very hard and dense fill soils. Unless covered by pavements or slabs, the passive pressure resistance in the upper 12 inches should be neglected.

RETAINING STRUCTURES

Where impact barriers are functioning as retaining structures, they should be designed to resist the lateral earth pressures due to the adjacent soils and surcharge effects. The recommended lateral earth pressures for design of retaining structures, expressed in equivalent fluid pressures of pounds per square foot per foot of depth (pcf), 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	34	53
	Vertical	None	None

The values provided in the table above assume that granular backfills will be used to backfill behind the retaining structures. It is assumed that the backfill behind retaining structures will be compacted to at least 95 percent relative compaction. In general, an active condition may be used for gravity retaining walls that are free to deflect by as much as 0.5 percent of the structure height. If the tops of the structures are not free to deflect beyond this degree, or are restrained, the retaining structures 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 structures.

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

Dynamic Lateral Earth Pressures

Dynamic lateral earth forces due to seismic loading should be considered for impact barriers acting as retaining structures. An appropriately reduced safety factor may be used when dynamic lateral earth forces are accounted for in the design of the retaining structure.

Forces due to dynamic lateral earth pressures may be estimated using $10H^2$ pounds per linear foot of wall length for level backfill conditions, where H is the height of the wall in feet. The dynamic lateral earth forces are in addition to the static lateral earth pressures provided above. The resultant force should be assumed to act through the mid-height of the wall. 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.

Drainage

Retaining structures should be well drained to reduce the build-up of hydrostatic pressures. A typical drainage system would consist of 1 to 2 cubic foot of permeable material, such as open-graded gravel (AASHTO M43, No. 67 gradation), wrapped with non-woven filter fabric (Mirafi 180N or equivalent) placed at each of the weep hole locations. The weep holes should be spaced not more than 6 feet apart.

Backfill behind the drainage zone should consist of the granular backfills compacted to at least 95 percent relative compaction as mentioned above. Unless covered by concrete slabs, the upper 12 inches of backfill should consist of relatively impervious materials to reduce the potential for appreciable water infiltration behind the retaining structures.

LIMITATIONS

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

This report has been prepared for the exclusive use of Nakamura, Oyama and Associates, Inc. for specific application to the proposed Highway Lighting Improvements, Moanalua Freeway from Halawa Heights Off-Ramp to Middle Street Overpass on the Island of Oahu, Hawaii in accordance with generally accepted geotechnical engineering principles and practices. No warranty is expressed or implied.

This report has been prepared solely for the purpose of assisting the engineer in the project design. The owner/client should be aware that unanticipated soil and/or rock conditions are commonly encountered. Unforeseen subsurface conditions, such as soft deposits, hard layers, cavities, or perched groundwater, 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 or need additional information, please contact our office.