SECTION 3. DISCUSSION AND RECOMMENDATIONS

Our field exploration along Kekaulike Avenue at about MP 8.2 generally encountered fill materials placed over weathered basalt rock, consisting of residual/saprolite soils and hard basalt formation at greater depths. It should be noted that cobbles/boulders and soft/loose pockets were encountered within the fill materials at Boring No. 1. We did not encounter groundwater in the drilled borings at the time of our field exploration. However, groundwater levels are subject to change due to rainfall, time of year, seasonal precipitation, surface water runoff, and other factors.

Based on the current design concept, the segmental retaining wall system consisting of concrete panel walls (a.k.a. Tee Walls) will be installed along the damaged roadway embankment. In general, the segmental retaining wall system is a composite wall system that utilizes high-density polyethylene, or other reinforcing elements. This composite system essentially forms an internally stabilized gravity wall structure with an ability to tolerate total and differential settlements. To accommodate the high rainfall environment conditions in the Kula areas, we recommend using Controlled Low-Strength Material (CLSM) to substitute the conventional select granular fill material that requires compaction under the controlled moisture conditioning. The use of the CLSM backfill will also exert very low lateral pressure onto the wall after the mix hardens.

We recommend that an ultimate bearing capacity of up to 7,500 psf may be used to evaluate the extreme limit state of the footings bearing on the medium stiff/dense silty/sandy material. To evaluate the strength limit state of the foundations, a bearing pressure of up to 3,750 psf may be used.

Consideration should also be given to densify the subgrade of the segmental retaining wall, using proof rolling by the heavy construction bulldozer. Specific proof rolling construction procedures should be developed during the construction. Geolabs should be retained during construction to assist in developing the procedure and criteria.

Detailed discussion of these items and our geotechnical recommendations for design of segmental retaining walls, retaining structures, and other geotechnical aspects of the project are further discussed in the following subsections.

3.1 Segmental Retaining Walls

Design alternatives including, soil nail shotcrete retaining walls, the lagging wall with soldier piles and tiebacks, and a concrete panel wall system, were considered. The following elements for each option were considered.

Option 1 - Soil Nail Shotcrete Retaining Wall

Advantages:

- One lane of traffic open
- Second economical solution
- Possible increasing roadway width

Disadvantages:

- Possible boulder encounter (potential construction change order)
- Heavy equipment
- Possible temporary lane closure
- Possible Electrical Line relocation on Makai

Option 2 – Lagging Wall with Soldier Piles and Tiebacks

Advantages:

- One lane of traffic open
- Most economical solution
- Adaptive to exiting grade condition with GRP bench to reduce risk of undermining lagging wall between the soldier piles
- Possible natural look to blend into surroundings using wood lagging with the concern of the life span

Disadvantages:

- Possible boulder encounter (potential construction change order)
- Heavy equipment
- Drilling into fill material requires more anchors
- Possible temporary lane closure

Option 3 – Concrete Panel Wall (Tee Wall) System

Advantages:

- Upstream and downstream embankment improvement
- Eliminate boulders encountered at the site
- Conventional excavation and replacement methodology with heavy excavation equipment readily available in Maui
- CLSM can be supplied by HC&D in Maui. The precast panel form is currently stored in Maui
- No future maintenance is required
- DOT and FHWA have no issue with successful examples like H-1 at Aiea, Kahekili Hwy, Kam Hwy in Waimea, etc.
- Maui contractor such as GBI will be capable of qualifying for construction

Disadvantages:

- Heavy equipment
- Restoration of the whole roadway for approximately 110 lineal feet
- Possible Electrical and utility Line relocation on both sides
- Removal of roadway and excavation
- Roadway closure for two months

All options will require excavation to place the proposed solution and stabilization of the downstream area with approximately the same amount of construction time. Additional improvements include redirecting the outlet of the existing culvert to flow towards the center of the gulch.

Based on the concerns of large boulders disclosed in the existing embankment fills, the concrete panel wall (a.k.a. Tee Wall) system was selected to replace the damaged roadway embankment. The concrete panel wall system will behave similarly to the segmental retaining wall system.

In general, the segmental retaining wall system is a composite wall system that utilizes high-density polyethylene, or other reinforcing elements to reinforce the backfill zone and improve the shear strength of the reinforced soil zone. This composite system essentially forms a gravity wall structure with an ability to tolerate significant total and differential settlements. In addition, segmental retaining walls are also desirable due to the flexibility of the wall, ease of construction, high load carrying capacity, and economy.

Design of the segmental retaining wall system will need to take into consideration both the external and internal stability of the structure. In evaluating external stability, the retaining wall requires to satisfy four stability conditions: (1) bearing failure, (2) translational sliding, (3) overturning stability, and (4) overall slope stability. Geotechnical design parameters to evaluate these stability conditions are presented in the following subsections.

3.1.1 Segmental Retaining Wall Foundations

Based on the generally medium stiff/dense silty and sandy subsurface conditions encountered, we recommend that the following values may be used to evaluate the bearing support, sliding resistance, and passive pressure resistance of the planned retaining walls based on LRFD design methods.

RETAINING WALL FOUNDATIONS			
Description	**Extreme Event Limit State**	**Strength Limit State**	**Service Limit State**
Bearing Pressure (min. 24” of embedment)	7,500 psf	3,750 psf	2,500 psf
Coefficient of Friction	0.46	0.39	N/A
Passive Pressure Resistance	360 pcf	180 pcf	N/A

In general, the retaining wall should be embedded a minimum of 24 inches below the lowest adjacent finished grade. In addition, the footing should be extended deeper to obtain a minimum 6-foot setback distance measured horizontally from the outside edge of the footing to the face of the slope for sloping ground conditions.

The wall subgrades should be compacted to a minimum of 90 percent relative compaction to provide a firm and unyielding base. Soft and/or loose soils encountered at the wall subgrades should be over-excavated to a minimum depth of 24 inches below the bottom of the wall elevation. The over-excavation should also extend a minimum of 24 inches laterally beyond the front face of the walls. The resulting over-excavation should be backfilled with aggregate subbase materials.

Based on a service limit state bearing pressure of 2,500 psf, we estimate that foundation settlements for foundations bearing on the recompacted subgrade to be less than 1 inch. Differential settlements between adjacent footings supported on the silty/sandy soils and/or basalt formation should be on the order of about 0.5 inches.

Lateral loads acting on the structure may be resisted by frictional resistance between the base of the foundation and the subgrade and by passive earth pressure developed against the near-vertical faces of the embedded portion of the foundation. The passive pressure resistance values presented in the table above, expressed in pounds per square foot per foot of embedment (pcf), may be used to evaluate the passive resistance for footings embedded in medium dense sandy soils. Unless covered by pavements, slabs, or grouted rubble paving, the passive resistance in the upper 12 inches should be neglected.

3.1.2 Lateral Earth Pressures

We envision the segmental retaining wall will be backfilled with CLSM which will exert relatively low lateral pressure after setting. Considering flowable fill material during the initial construction placement, hydrostatic pressure should be considered to be supported by construction shoring support as appropriate.

3.1.3 Reinforced Fill

To provide internal stability, adequate layers of geogrids (or reinforcing strips) are typically installed to strengthen the backfills and secure the concrete panel.

Geogrids are generally polymer grid structures with a tensile strength comparable to steel. It generally provides a cost-effective solution to slope stability problems, which may include the following: insufficient right-of-way, high surcharge loads, poor-quality fills, high seismic forces, steep slopes, or difficult landslide repairs. When geogrids are placed in soil, the grid geometry interlocks with the adjacent soil, creating a soil-geogrid composite with greatly enhanced engineering properties. Different grid configurations are available to provide optimum soil-grid interaction for a range of soil types and slope reinforcement applications. Reinforced slope geotextiles work in a similar manner to reinforced geogrids.

The lengths of the geogrid layers are designed to anchor potential failure zones into stable interior sections of the retaining system. As forces develop within a soil mass, the high-modulus geogrids are immediately pulled into tension. The geogrids transfer its tensile force from the unstable soil back into less-stressed portions of the slope, and stability is thus maintained.

Considering high rainfall environments in the vicinity of the project site, the conventional select granular fills may experience the difficulty of compaction, inducing the construction delay. We recommend using CLSM for the proposed new concrete panel segmental retaining wall system. Based on our field exploration, soft and/or loose material was encountered within the existing embankment fill area. We believe this modification to the backfill material will help minimize the lateral earth pressure against the retaining wall while reducing the potential for settlement in the soft and/or loose material.

Consideration should also be given to densify the subgrade of the segmental retaining wall, using proof rolling by the heavy construction bulldozer. Specific proof rolling construction procedures should be developed during the construction. Geolabs should be retained during construction to assist in developing the procedure and criteria.

3.1.4 Overall Slope Stability

We have evaluated the overall slope stability of the segmental retaining wall structure planned for the project. Based on our analyses, the factor of safety for the stability of the segmental retaining wall is at least 1.5, which is the minimum factor of safety normally recommended.

3.2 Retaining Walls

Based on the current design concept, we anticipate retaining wall systems will be installed to construct return walls and/or wingwall extensions for the underlying drainage structure. Therefore, the following recommendations may be considered for the design of retaining structures for the project.

3.3 Retaining Structure Foundations

Based on the generally medium stiff/dense silty and sandy subsurface conditions encountered, we recommend the retaining structure foundations consist of a shallow foundation system consisting of strip footings. Parameters for design of foundations for retaining structures should be designed in accordance with the "Segmental Retaining Wall Foundations" subsection herein.

3.3.1 Lateral Earth Pressures

Retaining structures should be designed to resist the lateral earth pressures due to the adjacent soils and surcharge effects. We recommend the following lateral earth pressures for design of retaining structures, expressed in equivalent fluid pressures of pounds per square foot per foot of depth (pcf), as 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	40	60
	Vertical	None	None
Maximum 3H:1V Sloping Backfill	Horizontal	45	64
	Vertical	17	23

The values provided above assume that the excavated on-site materials consisting of particles less than 6 inches in largest dimension and/or general fill materials will be used to backfill behind the concrete retaining wall. It is assumed that the backfill behind the retaining structures will be compacted to between 90 and 95 percent relative compaction per ASTM D1557. Over-compaction of the retaining structure backfill should be avoided. The lateral earth pressure values provided above do not include hydrostatic pressure that may be caused by groundwater trapped behind the retaining wall structure.

Surcharge stresses due to areal surcharges, line loads, and point loads within a horizontal distance equal to the depth of the structure 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 over the entire height of the structure 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 structure may be used for design. Additional analyses during design may be needed to evaluate the surcharge effects of point loads and line loads.

3.3.2 Dynamic Lateral Earth Pressures

Based on the anticipated stiff residual/saprolite overlying hard basalt formation subsurface condition, we believe the site classification at the cut slope site to be Site Class C in accordance with Table 3.10.3.1-1 of AASHTO LRFD (2017 edition). Dynamic lateral earth forces due to seismic loading (A_s=0.295g) may be estimated by using 6.5H^2 and 15.8H^2 pounds per lineal foot of wall length (where H is the height of the wall in feet) for flat backfill conditions and sloping backfill conditions of 3H:1V, respectively. 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. An appropriately reduced factor of safety may be used when dynamic lateral earth forces are accounted for in the design of the retaining structures.

3.3.3 Drainage

In general, a subdrain is recommended behind the retaining wall structure to collect and discharge excess water that may infiltrate behind the wall. A typical subdrainage system would consist of a perforated pipe (with perforations down) enclosed by at least 12 inches of permeable drainage material, such as AASHTO M43, No. 67 gradation. The perforated pipe should be directed to discharge into a proper drainage facility. The permeable drainage material should be wrapped in a non-woven filter fabric, such as Mirafi 180N or equivalent. Unless covered by concrete slabs, the upper 12 inches of backfill should consist of relatively impervious material (compacted on-site soils) to reduce the potential for significant water infiltration behind the retaining wall.

3.4 Site Grading

We anticipate the project will generally consist of cuts of up to about 20 feet deep and fills of less than 5 feet in thickness. In general, grading work should conform to Section 200 of the Hawaii Standard Specifications for Road and Bridge Construction (2005) and the site-specific recommendations contained in this report. Items of site grading that are addressed in the subsequent subsections include the following:

1. Cut and Fill Slope Design
2. Site Preparation
3. Fills and Backfills
4. Fill Placement and Compaction Requirements
5. Excavations

A Geolabs representative should monitor site grading operations to observe whether undesirable materials are encountered during the excavation process and to confirm whether the exposed soil conditions are similar to those encountered in our field exploration.

3.4.1 Cut and Fill Slope Design

Based on the subsurface conditions anticipated along the embankment reconstruction project, we believe that the planned cut slopes will likely expose relatively medium stiff/dense fill materials and residual soils. In general, we believe that a cut slope inclination of 3H:1V or flatter may be used for the design of the planned cut slopes for the project.

In general, permanent embankments constructed of the compacted on-site soils should also be designed with a slope inclination of 3H:1V or flatter. Fills to be placed on existing slopes with inclinations steeper than 5H:1V should be keyed and benched into the existing slope to provide stability for the new fill against sliding. The keyway at the bottom of fill slopes should be embedded at least 2 feet below the lowest adjacent grade and have a minimum base width of 10 feet.

Excessive surface water runoff over the slope face may cause erosion of the exposed soils, thus jeopardizing the long-term stability and performance of the cut and fill slopes. Therefore, it is our opinion that slopes should be protected by appropriate grouted rubble paving or by other means, such as placement of geotextile fabrics on the slope face, as soon as practical after the slope is constructed.

3.4.2 Site Preparation

At the on-set of earthwork, areas within the contract grading limits should be thoroughly cleared and grubbed. It should be noted that portions of the existing terrain are heavily vegetated. Vegetation, debris, deleterious materials, and other unsuitable materials should be removed and disposed of properly off-site to reduce the potential for contamination of the excavated materials.

Soft and yielding areas encountered during clearing and grubbing below areas designated to receive fill should be over-excavated to expose firm natural material, and the resulting excavation should be backfilled with well-compacted general fill. The excavated soft soils should be properly disposed of off-site.

In general, the over-excavated subgrades and areas designated to receive fills (exposing soils) 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.

3.4.3 Fills and Backfills

In general, backfills may consist of compacted general fills unless otherwise specified. The near-surface silty/sandy soils encountered during our field exploration should be suitable for use as general fill materials, provided that the maximum particle size is less than 6 inches in largest dimension. The on-site cut materials generated from excavations into the underlying basalt formation may be used as general fill or backfill materials, provided that they are screened of the over-sized materials and/or processed to meet the above gradation requirements (less than 6 inches in largest dimension).

Imported material to be used as select granular fill should be non-expansive granular material, such as crushed coral, basalt, or cinder sand. 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 less than 15 percent particles passing the No. 200 sieve. The material should have a laboratory CBR value of 25 or more and should have a maximum swell value of one 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.

3.4.4 Fill Placement and Compaction Requirements

In 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 90 percent relative compaction. Fills and backfills within 3 feet of the pavement grade elevation should be compacted to 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 determined in accordance with ASTM D1557. Optimum moisture is the water content (percentage by dry weight) corresponding to the maximum dry density. Compaction should be accomplished by sheepsfoot rollers, vibratory rollers, or other types of acceptable compaction equipment. Water tamping, jetting, or ponding should not be allowed to compact the fills.

It should be noted that some of the on-site soils generally exist in a relatively moist to wet condition. Therefore, some moisture reduction may be required to achieve the minimum 90 percent compaction criteria, especially for materials primarily consisting of silts and clays. Aeration to lower the soil moisture and more compaction effort to achieve the specified compaction would generally reduce the rate of fill placement for this project. In addition, adequate stockpile areas may not be readily available on-site. Contractors proposing to work on this project should be encouraged to examine the site conditions and its limitations.

3.4.5 Excavations

Our site reconnaissance and field exploration program disclosed that the near-surface soils generally consist of medium stiff/dense silts and sands with scatted large cobbles and boulders overlying weathered basalt rock at the greater depth. Basalt rock formation and boulders may be encountered in the excavations and in localized areas along the project alignment. In general, it is our opinion that conventional heavy excavation equipment, such as large bulldozer, excavator, or similar heavy construction equipment, may achieve the excavations into these materials. However, excavations into the harder areas will likely require the use of hoearms or chipping.

The method and equipment to be used for excavation should be determined by the contractor, subject to practical limits and safety considerations. The excavations should comply with all applicable local safety considerations. The excavations should comply with all applicable local safety requirements. The above discussions regarding the rippability of the surface materials are based on field data obtained from our field reconnaissance and the borings performed at the subject site. Contractors proposing to work on this project should be encouraged to examine the site conditions to make their own interpretation.

3.5 Design Review

Preliminary and final drawings and specifications for the proposed construction should be forwarded to Geolabs for review and written comments prior to bid advertisement and/or construction. This review is needed to evaluate the conformance of the plans and specifications with the intent of the earthwork and foundation recommendations provided herein. If this review is not made, Geolabs cannot assume responsibility for misinterpretation of our recommendations.

3.6 Post-Design Services/Services During Construction

Due to the variability in the subsurface conditions, it is highly recommended to retain Geolabs for geotechnical engineering support and continued services during construction of the proposed project. The following are critical items of construction monitoring that require "Special Inspection":

1. Observation of the shallow foundation excavations
2. Observation of segmental retaining wall excavations and construction
3. Observation of the subgrade soil preparation, including proof rolling operation
4. Observation of fill placement and compaction

Other aspects of the earthwork construction should also be observed by a representative from Geolabs. This is to observe compliance with the intent of the design concepts, specifications, and/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 exposed subsurface soil conditions encountered during construction differ from those assumed or considered in this report, Geolabs should be contacted to review and/or revise the geotechnical recommendations presented herein.

END OF DISCUSSION AND RECOMMENDATIONS

SECTION 4. LIMITATIONS

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

The boring locations indicated herein are approximate, having been staked out in the field using a hand-held Global Positioning System (GPS) device. Elevations of the borings were interpolated based on the spot elevations shown on the Topographic Survey Map prepared by Fukumoto Engineering, Inc. dated December 27, 2021. The field boring locations and elevations should be considered accurate only to the degree implied by the methods used.

The stratification lines shown on the graphic representations of the borings depict the approximate boundaries between the soil types and, as such, may denote a gradual transition. We did not encounter groundwater in the borings at the time of our field exploration. However, it must be noted that fluctuation may occur due to variation in seasonal rainfall, surface water runoff and other factors.

This report has been prepared for the exclusive use of AECOM and their client, State of Hawaii – Department of Transportation, Highways Division, for specific application to the design of the *Kekaulike Avenue, Emergency Repairs at MP 8.2* project 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 engineers in the preparation of the design for the emergency repairs project. Therefore, this report may not contain sufficient data, or the proper information, for use to form the basis for preparation of construction cost estimates or contract bidding. A contractor wishing to bid on this project should retain a competent geotechnical engineer to assist in the interpretation of this report and/or performance of site-specific exploration for bid estimating purposes.

The owner/client should be aware that unanticipated soil conditions are commonly encountered. Unforeseen subsurface conditions, such as perched groundwater, soft deposits, cobbles/boulders, hard layers or cavities, may occur in localized areas and may require additional probing or corrections in the field (which may result in construction delays) to attain a properly constructed project. Therefore, a sufficient contingency fund is recommended to accommodate these possible extra costs.

END OF LIMITATIONS