GEOTECHNICAL ENGINEERING EXPLORATION AND PAVEMENT JUSTIFICATION REPORT

KAPOLEI INTERCHANGE COMPLEX

PHASE 3

KAPOLEI, OAHU, HAWAII

W.O. 5537-50 AUGUST 17, 2015

Prepared for

MITSUNAGA & ASSOCIATES, INC.

GEOLABS, INC.

Geotechnical Engineering and Drilling Services

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Prepared for

MITSUNAGA & ASSOCIATES, INC.

GERALD Y. SEKI
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HAWAII, U.S.A.

THIS WORK WAS PREPARED BY
ME OR UNDER MY SUPERVISION.

SIGNATURE 4-30-16
EXPIRATION DATE OF THE LICENSE

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Hawaii • California

GEOLABS, INC.
Geotechnical Engineering and Drilling Services

August 17, 2015
W.O. 5537-50

Mr. Chad McDonald
Mitsunaga & Associates, Inc.
747 Amana Street, Suite 216
Honolulu, HI 96814

Dear **Mr. McDonald:**

Geolabs, Inc. is pleased to submit our report entitled "Geotechnical Engineering Exploration and Pavement Justification Report, Kapolei Interchange Complex, Phase 3, Kapolei, Oahu, Hawaii" prepared in support of the project design.

Our work was performed in general accordance with the scope of services outlined in our fee proposal dated November 7, 2013.

Please note that the soil samples recovered during our field exploration (remaining after testing) will be stored for a period of two months from the date of this report. The samples will be discarded after that date unless arrangements are made for a longer sample storage period. Please contact our office for alternative sample storage requirements, if appropriate.

Detailed discussion and specific design recommendations are contained in the body of this report. If there is any point that is not clear, please contact our office.

Very truly yours,

GEOLABS, INC.

Clayton S. Mimura

Clayton S. Mimura, P.E.
President

CSM:GS:as

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Hawaii • California

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PAVEMENT JUSTIFICATION REPORT

KAPOLEI INTERCHANGE COMPLEX

PHASE 3

KAPOLEI, OAHU, HAWAII

W.O. 5537-50 AUGUST 17, 2015

SUMMARY OF FINDINGS AND RECOMMENDATIONS

Our field exploration generally encountered a surface fill layer underlain by alluvium and coralline materials extending to the maximum depth explored of about 46.5 feet below the existing ground surface. The surface fill layer consisted of loose to very dense sandy/clayey gravel and clayey sand, very dense boulders, and stiff to hard clay with some sand, gravel, and cobbles. The alluvium was composed of stiff to hard clays and silts. The coralline materials consisted of coralline detritus (loose to medium dense silty sand/gravel) and soft to medium hard sandstone and coral formation. We did not encounter groundwater in the borings at the time of the field exploration.

Based on our field observations, the surface soils at the project site are dry and friable. In addition, we observed substantial shrinkage cracks extending up to about 2 to 3 feet below the existing ground surface. Because of the observed ground conditions, we recommend implementing specific site preparation procedures during the earthwork construction for this project. After clearing and grubbing, the exposed soils should be over-excavated by at least 18 inches, moisture conditioned to at least 2 percent above the optimum moisture content, and replaced as compacted fills. After over-excavation of the soils below the existing ground surface, the over-excavated subgrade areas should be scarified to a depth of at least 12 inches, moisture conditioned to at least 2 percent above the optimum moisture content, and compacted to no less than 90 percent relative compaction. The 18 inches of over-excavated soils may be re-used as a source of general fill materials provided that the materials are free of deleterious materials and are moisture-conditioned and compacted as recommended herein.

We understand that new retaining structures up to about 28 feet in height are planned for the project. Based on the results of our field exploration and analysis, we believe that a shallow foundation system consisting of continuous strip footings bearing directly on or embedded in the stiff to hard clay may be used to support the new retaining wall structures. Conventional and segmental retaining wall systems are being considered for the project. Based on the results of our field exploration, bearing values of up to 13,500 and 6,700 pounds per square foot (psf) may be used for the extreme event and strength limit states, respectively, using Load Resistance Factor Design (LRFD) method. These bearing values assume that the wall system will bear on stiff to hard clay soils. For service limit state condition, a bearing value up to 4,500 psf may be used. Based on our analysis, a minimum geogrid length of 125 percent of the height of the wall should be used for the segmental retaining wall system.

We understand that new drainage structures including a reinforced concrete box culvert will be constructed. The bearing values for the different limit states provided for the retaining structures may be used for the drainage structures bearing on stiff to hard clay soils. The box culvert should be designed to resist an at-rest lateral pressure of 53 pounds per cubic foot (equivalent fluid pressure).

Based on the traffic data provided, the R-Value of the subgrade soils, and the economic analyses performed, we recommend using the following structural pavement section for the new pavements for the project.

12.0-Inch Portland Cement Concrete
24.0-Inch Aggregate Subbase Course
36.0-Inch Total Pavement Thickness on Compacted Subgrade

One of the primary distress mechanisms in pavement structures is pumping due to saturation of the base, subbase, and/or subgrade soils. It should be noted that the clayey subgrade soils are susceptible to softening and loss of strength upon saturation. Therefore, special attention should be given to the surface and subsurface drainage of the pavements. As a minimum, the pavement surface should be sloped and drainage gradients should be maintained to carry surface water off the pavement to appropriate drainage structures. To collect infiltrated water into the pavement structural section, we recommend providing a pavement edge drain system (hydraulically connected to the untreated permeable base) along the edges of the new pavements.

The text of this report should be referred to for detailed discussion and specific design recommendations.

END OF SUMMARY OF FINDINGS AND RECOMMENDATIONS

SECTION 1. GENERAL

1.1 Introduction

This geotechnical engineering exploration report presents the results of our analyses and geotechnical recommendations for the *Kapolei Interchange Complex, Phase 3* project in the District of Ewa on the Island of Oahu, Hawaii. The project location and general vicinity are shown on the Project Location Map, Plate 1.

This report summarizes the findings from our field exploration and presents our geotechnical recommendations derived from our analyses for site grading, retaining wall design, storm water detention basin design, and pavement design only. The findings and recommendations presented herein are subject to the limitations noted at the end of this report.

1.2 Project Considerations

The Phase 3 project is part of the Kapolei Interchange Complex project in the Kapolei area on the Island of Oahu, Hawaii. The Phase 3 project consists of new on and off-ramps between Interstate Route H-1 and the future State Harbor Access Road located west of Kapolei Commons and new retaining walls.

The new on and off-ramps between Interstate Route H-1 and the future State Harbor Access Road will be about 1,400 and 1,700 linear feet in length. We understand that new retaining walls will be required for the ramps due to the right-of-way limits. In addition, we understand that the use of a segmental retaining wall system is being considered. Since new pavements will be constructed within the State's right-of-way, a pavement justification report will be required for the project.

A new 4-foot by 10-foot reinforced concrete box culvert will be installed near the northern end of the future Harbor Access Road. The new box culvert will carry drain water from three existing 30-inch diameter pipe culverts crossing the Interstate Route H-1 freeway to beyond the new western ramp embankment. In addition, two 30-inch diameter pipe culverts will be installed on the eastern portion of the project and will connect to the new box culvert structure.

The use of storm water basins is being considered for the project. Therefore, field infiltration testing was performed.

Based on the preliminary drawings, fills on the order up to of about 28 feet relative to the existing ground surface will be required to achieve the design grades for the new roadways.

1.3 Purpose and Scope

The purpose of our exploration was to obtain an overview of the surface and subsurface conditions to develop a generalized soil/rock data set to formulate geotechnical recommendations for the design of site grading, retaining wall structures, storm water detention basins, and pavements. Our work was performed in general accordance with our fee proposal dated November 7, 2013. The scope of work for this exploration included the following tasks and work efforts:

1. Coordination of utility clearance by our field engineer.
2. Coordination of the trail clearing to access the boring locations by our field engineer.
3. Trail clearing with a loader to provide access for our truck-mounted drill rig.
4. Mobilization and demobilization of truck-mounted drill rigs and two operators to and from the project site.
5. Drilling and sampling of 10 borings extending to depths of about 14.25 to 46.5 feet below the existing ground surface.
6. Coordination of the field exploration and logging of borings and percolation tests by our field geologist.
7. Laboratory testing of selected soil samples obtained during the field exploration as an aid in classifying the materials and evaluating their engineering properties.
8. Engineering analyses of the field and laboratory data to formulate geotechnical recommendations for the design of the site grading, retaining walls, storm water detention basins, and pavements.
9. Preparation of this report summarizing our work on the project and presenting our findings and geotechnical recommendations.
10. Coordination of our overall work on the project by our engineer.

11. Quality assurance of our work and client/design team consultation by our principal engineer.

12. Miscellaneous work efforts such as drafting, word processing, and clerical support.

Detailed descriptions of our field exploration and Logs of Borings are provided in Appendix A. Results of the laboratory tests performed on selected soil samples are presented in Appendix B. Results of the percolation tests are presented in Appendix C. Pavement design calculations are presented in Appendix D.

END OF GENERAL

SECTION 2. SITE CHARACTERIZATION

2.1 Regional Geology

The Island of Oahu was built by the extrusion of basalt and basaltic lavas from the Waianae and Koolau shield volcanoes. The older Waianae Volcano is estimated to be middle to late Pliocene in age and forms the bulk of the western one-third of the island. The younger Koolau Volcano is estimated to be late Pliocene to early Pleistocene (Ice Age) in age and forms the majority of the eastern two-thirds of the island. As volcanic activity in Waianae Volcano ceased, lava flows from Koolau Volcano banked against its eroded eastern slope forming a broad plateau, known as Schofield Plateau.

Following extrusion of the lavas in the early Pleistocene Epoch, the island underwent a long cycle of erosion and weathering forming the prominent ridgelines and summits as we know today. During the erosion period, the Island of Oahu began to slowly subside by more than 1,200 feet in elevation, resulting in the drowning and sedimentation of the valleys and the formation of the steep Koolau Pali. Coral reefs continued to grow in the surrounding shallow waters of the island.

From the mid to late Pleistocene Epoch, the sea level repeatedly rose and fell in response to global glaciation and the availability of surface waters to sustain the oceans. The various sea level elevations and their representative deposits are known as “stands” and include from oldest to youngest: the Kahuku, Kahipa, Kaena, Laie, Waialae, Waipio, and Waimanalo. Geologic deposits associated with the various sea level stands, including marine sediments and coral reefs, were deposited and subsequently altered or removed by later sea level fluctuation. Therefore, depositional records reflecting the changes in sea level and the occurrence of emerged coral reef deposits are often incomplete.

The project site is situated on the Ewa Plain to the southeast of the Waianae Mountains. The Ewa Plain is a gently sloping alluvial plain formed by the deposition of alluvial clays and silts derived from weathering of the basalt rock formation further up-slope. The alluvial deposits were laid down and are inter-bedded with marine sediments and coral/algal reef formations to form a sedimentary wedge. The thickness of the sedimentary wedge ranges from zero in the area of the Interstate Route H-1 Highway to over 1,000 feet at Ewa Beach. This wedge forms the Ewa Plain and serves as the confining formation, or "caprock," over the artesian basal aquifers of Southern Oahu. Basalt rock formation resides below the marine deposits at a substantial depth.

2.2 Site Description

The project site is between the existing Harbor Access Road and Interstate Route H-1 and west of the Kapolei Commons development as shown on the Site Plans, Plates 2.1 and 2.2. The new roadway alignments traverse vacant areas and are heavily vegetated by brush and wild grasses along the proposed alignments.

Based on the topographic survey map provided, the area generally slopes down toward the south. The ground surface elevations at the boring locations range from about +76 to +95 feet Mean Sea Level (MSL). An existing drainage course was observed along the proposed Harbor Access Road extension.

2.3 Subsurface Conditions

The subsurface conditions at the project site were explored by drilling and sampling 10 borings, designated as Boring Nos. 1 through 10, extending to depths of about 5 to 46.5 feet below the existing ground surface. The approximate boring locations are shown on the Site Plans, Plates 2.1 and 2.2.

Our field exploration generally encountered a surface fill layer ranging from about 0.5 to 9.5 feet thick underlain by alluvial deposits and coralline materials extending to the maximum depth explored of about 46.5 feet below the existing ground surface. In one of the borings, weathered basalt rock formation was encountered below the alluvial deposit.

The surface fill layer consisted of loose to very dense sandy/clayey gravel and clayey sand, very dense boulders, and stiff to hard clay with some sand, gravel, and cobbles. The alluvium was composed of stiff to hard clay, clayey silt, and silty clay. The coralline materials consisted of coralline detritus (loose to medium dense silty sand/gravel) and soft to medium hard sandstone and coral formation.

We did not encounter groundwater in our borings at the time of our field exploration. It should be noted that water levels may be influenced by seasonal precipitation, storm surge conditions and other factors.

Detailed descriptions of the field exploration methodology are presented in Appendix A. Descriptions and graphic representations of the materials encountered and the water levels observed in the borings are presented on the Logs of Borings in Appendix A. Laboratory tests were performed on selected soil samples, and the test results are presented in Appendix B. Percolation test results are presented in Appendix C. Pavement design calculations are presented in Appendix D.

END OF SITE CHARACTERIZATION

SECTION 3. DISCUSSION AND RECOMMENDATIONS

Our field exploration generally encountered a surface fill layer underlain by alluvium and coralline materials extending to the maximum depth explored of about 46.5 feet below the existing ground surface. We did not encounter groundwater in the borings at the time of the field exploration.

The clayey fill and alluvial soils at or near the existing ground surface are very dry and friable with numerous relatively deep shrinkage cracks evident at the ground surface. In addition, the fill and alluvial clay soils are moderately to highly expansive. Therefore, we recommend implementing special site preparation procedures for the clayey soils below the existing ground surface during construction.

Based on our field exploration, we believe that a shallow foundation system consisting of continuous strip footings bearing directly on or embedded in the stiff to hard clay may be used to support the new retaining wall structures. In addition, we believe that segmental retaining walls bearing on the stiff to hard clay soils may be used.

Detailed discussion of these items and our geotechnical recommendations for design of site grading, retaining wall foundations, segmental retaining walls, storm water detention basins, and pavements for the new roadway project are presented in the following sections.

3.1 Site Grading

Based on the existing terrain and the roadway profile plans, we anticipate that site grading work for the proposed roadway construction will generally consist of fills up to 28 feet in vertical height in order to achieve the finished grades for the new roadway embankment. The following grading items are addressed in the succeeding subsections:

1. Site Preparation
2. Fills and Backfills
3. Fill Placement and Compaction Requirements
4. Cut and Fill Slopes

Based on field observations, we noted substantial shrinkage cracks at the existing ground surface. In general, shrinkage cracks will develop in the ground due to the lack of maintenance of the soil moisture content as the soils start to dry out under the sun and dry weather conditions. The development of shrinkage cracks in the ground was apparently intensified by the prolonged dry climatic conditions in the Ewa area resulting in large fluctuations in the soil moisture content.

It should be noted that shrinking of the near-surface soils would occur as soil moisture evaporates from the soil system under the prolonged dry climatic conditions characteristic of the Ewa area due to the medium to high plasticity of the on-site clayey soils. These shrinkage cracks may close up to a certain degree upon wetting as the soils expand during periods of wet weather or heavy rains. However, shrinkage of the on-site soils and subsequent cracking of the ground surface will recur when the dry weather returns. Therefore, the development of shrinkage cracks will recur when the on-site clayey soils are subjected to prolonged dry climatic conditions, which causes large fluctuations in the moisture content, unless some measures (such as landscape irrigation) are taken to maintain the soil moisture content at a relatively constant level.

As discussed above, the phenomenon of shrinkage crack development will recur depending on climatic conditions unless measures are taken to maintain a relatively constant soil moisture content, thereby reducing the shrink/swell behavior of the on-site clayey soils due to moisture content fluctuations. These long-term solutions may include landscaping, placement of non-expansive fills over the graded areas, and/or provision of an irrigation system to maintain the moisture contents of the near-surface soils in a relatively narrow range.

Therefore, we recommend grassing the graded surfaces (including cut and fill slopes) to reduce the potential for soil erosion after completion of the site grading. The relatively dry climatic conditions of the Ewa area will likely require controlled irrigation of the graded surfaces to maintain the moisture contents and to keep graded surfaces free of large shrinkage cracks. However, over-watering of the graded surfaces should be avoided to reduce the potential for saturation and softening of the surface soils.

Due to the nature of the on-site soils, 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 and assumed in our analyses.

3.1.1 Site Preparation

As previously mentioned, the surface soils at the project site are dry and friable. Based on our field observations, we noted substantial shrinkage cracks (on the order of about 2 to 3 feet deep) are present at the existing ground surface. Because of the conditions at the existing ground surface, we recommend implementing special site preparation procedures during the earthwork construction.

In general, the areas within the contract grading limits should be cleared and grubbed thoroughly at the on-set of earthwork. Vegetation, debris, deleterious material, and other unsuitable materials should be removed and disposed properly off-site or disposed in a designated area to reduce the potential for contaminating the excavated materials.

After clearing and grubbing, the normally dry and friable soils should be over-excavated by at least 18 inches, moisture conditioned to at least 2 percent above the optimum moisture, and replaced as compacted fills. After over excavation of the soils below the existing ground surface, the over-excavated subgrade areas should be scarified to a depth of at least 12 inches, moisture conditioned to at least 2 percent above the optimum moisture content, and compacted to no less than 90 percent relative compaction. The 18 inches of the over-excavated soils may be re-used as a source of general fill materials provided that the materials are free of deleterious materials and are moisture conditioned and compacted as recommended herein.

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 (ASTM D1557). Optimum moisture is the water content (percentage by dry weight) corresponding to the maximum dry density. The intent of the recommended site preparation procedures serves to close up some of the large shrinkage cracks observed at the site. We strongly recommend that a Geolabs respresentative be present during and after the clearing and grubbing operations to evaluate the exposed ground conditions prior to site preparation operations.

Soft and/or yielding areas encountered during clearing and grubbing below areas designated to receive fill or future improvements should be over-excavated to expose stiff and/or dense materials. The resulting excavation should be backfilled with compacted on-site soils or replaced with select granular fill materials. The excavated soft and/or organic soils should be disposed properly off-site or used in landscaped areas, if appropriate. Contract documents should include additive and deductive unit prices for over-excavation and compacted fill placement to account for variations in the over-excavation quantities.

Where shrinkage cracks are observed after compaction of the subgrade, we recommend scarifying the soils and preparing again as recommended above. Saturation and subsequent yielding of the exposed subgrade due to inclement weather and poor drainage may require over-excavation of the soft areas and replacement with well-compacted fill.

3.1.2 Fills and Backfills

In general, the excavated on-site materials may be re-used as a source of general fill materials provided that the materials are free of deleterious materials. Imported fill materials, if required for general embankment construction, should consist of non-expansive select granular materials, such as crushed coralline or basaltic materials. The materials should be well graded from coarse to fine with particles no larger than 3 inches in largest dimension and should contain between 10 and 30 percent particles passing the No. 200 sieve. The materials should have a laboratory CBR value of 20 or more and should have a maximum swell 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.

3.1.3 Fill Placement and Compaction Requirements

Fill materials (clayey on-site soils and imported fill material) should be placed in level lifts not exceeding 8 inches in loose thickness, moisture-conditioned to at least 2 percent above the optimum moisture content, and compacted to at least 90 percent relative compaction. Aggregate subbase materials should be moisture-conditioned to above the optimum moisture content, placed in 8-inch level loose lifts, and compacted to no less than 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 (ASTM D1557). Optimum moisture is the water content (percentage by dry weight) corresponding to the maximum dry density. Compaction should be accomplished by using sheepsfoot rollers, vibratory rollers, or other types of acceptable compaction equipment.

3.1.4 Cut and Fill Slopes

In general, cut slopes exposing the in-situ clayey soils should be designed with a slope inclination of two horizontal to one vertical (2H:1V) or flatter. Permanent fill slopes may be designed with a slope inclination of 2H:1V or flatter. Fills placed on slopes steeper than 5H:1V should be keyed and benched into the existing slope to provide stability of the new fill against sliding.

The fill slope face should be finished to a relatively smooth and well-compacted surface. The fill slope face should be free of voids, which would allow erosion and migration of fines to occur. In addition, water should be diverted away from the tops of slopes, and 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.

3.2 Surcharge Fill and Settlement Monitoring

Based on our engineering analyses, we estimate about 1 to 3 inches of ground settlements may occur as a result of up to 28 feet of new embankment fill placement for the new roadways. Because of the high fill placement and the anticipated settlements, we recommend implementing a settlement monitoring program for the project and delaying construction of the pavement and utility structures to allow for the majority of the settlement to occur under the new load.

The estimated time to achieve 90 percent consolidation is about 1 to 2 months. After the settlement period, additional fill may be required to bring the grade to the finished grade level.

It should be recognized that it is difficult to accurately predict the exact time required for the filled ground to settle, since the settlement rate will be affected by variations of the subsoil structure and history of the subsoil deposition. We believe that the estimated settlement period could vary as much as 50 to 100 percent from the actual settlement period required. Therefore, the actual settlement rate should be monitored, and provisions should be made for potential delays in the construction schedule if a longer settlement period is required.

To monitor the actual settlement rate, we recommend installing settlement gauges within the fill areas of the new roadways. A typical settlement gauge detail is shown on Plate 3. The settlement gauges should be spaced at about 100-foot intervals from the start to the end of the fill area. The gauges should be read optically by a licensed surveyor, and readings should be transmitted to Geolabs for review. We recommend taking two readings (one week apart) for each settlement gauge prior to any site filling to establish a baseline. Subsequent readings should be taken on a weekly basis for the entire settlement period. Special care should be taken by the contractor to avoid damaging the settlement gauges. Each damaged settlement gauge should be replaced with two settlement gauges at the contactor's expense.

3.3 Retaining Structures

Two retaining wall structures are planned on the east and west sides of Interstate Route H-1. The borings drilled on the east side of Interstate Route H-1 was used for the retaining wall recommendations for both the east and west side walls. The retaining wall structures should be designed to resist the lateral earth pressures due to the adjacent soils and surcharge effects. Design of foundations for the retaining walls should be based on the parameters presented in the following subsections.

3.3.1 Seismic Design Considerations

Based on the design criteria by the State of Hawaii – Department of Transportation, the new retaining walls for the project will need to be designed based on a horizontal peak bedrock seismic acceleration coefficient of 0.251g. Based on the subsurface materials encountered in the drilled borings and the latest AASHTO LRFD Bridge Design Specifications, the project site may be classified from a seismic analysis standpoint as a stiff soil site corresponding to a Site Class D.

3.3.2 Shallow Retaining Wall Foundations

Based on the information provided, we understand that retaining walls of up to about 28 feet high may be required for the project. In general, we anticipate that shallow foundations bearing on alluvium consisting of stiff to hard clays encountered at the project site may be utilized for support of the planned retaining walls. Based on our field exploration, we believe 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	13,500 psf	6,700 psf	4,500 psf
Coefficient of Sliding Friction	0.40	0.34	N/A
Passive Pressure Resistance	275 pcf	138 pcf	N/A

In general, foundations should be embedded a minimum of 2 feet 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 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. Wall footings oriented parallel to the direction of the slope should be constructed in stepped footings.

Based on a service limit state bearing pressure of 4,500 pounds per square foot (psf), we estimate that foundation settlements under the anticipated design loads for foundations bearing on the stiff to hard alluvium to be less than 1 inch.

Lateral loads acting on the structure may be resisted by frictional resistance 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 hard alluvium. Unless covered by pavements or slabs, the passive resistance in the upper 12 inches should be neglected.

3.3.3 Static Lateral Earth Pressures

Retaining structures, including the abutment walls and wing walls, 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

The values provided above assume that Type A Structure Backfill Material conforming to Section 703.20 of the Hawaii Standard Specifications for Road and Bridge Construction, 2005 (HSS) 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 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.

3.3.4 Dynamic Lateral Earth Pressures

Dynamic lateral earth forces due to seismic loading may be estimated by using $15.6H^2$ 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 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.

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 $27.1H^2$ pounds per lineal foot of wall (H measured in feet) for level backfill conditions.

3.3.5 Drainage

Retaining walls 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.

3.4 Segmental Retaining Walls

We understand that the use of a segmental retaining wall system is being considered for the new roadways. In general, the segmental retaining wall system is a composite wall system, which 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. The installation of utility lines and structures within the reinforced soil zone and future maintenance of the utility lines needs to be considered for this wall system.

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 must 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. Some of the geotechnical parameters necessary in evaluating the internal stability of the retaining wall are presented in the "Reinforced Fill and Backfill Materials" subsection.

3.4.1 Segmental Retaining Wall Foundations

Based on information provided, we understand that segmental retaining walls up to about 28 feet high may be required for the new roadways. In general, we anticipate that shallow foundations bearing on alluvium consisting of stiff to hard clays encountered at the project site may be utilized for support of the planned segmental retaining walls. Based on the field exploration results, we believe 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 methods.

SEGMENTAL RETAINING WALL FOUNDATIONS			
	Extreme Event Limit State	**Strength Limit State**	**Service Limit State**
Bearing Pressure	13,500 psf	6,700 psf	4,500 psf
Coefficient of Sliding Friction	0.40	0.36	N/A
Passive Pressure Resistance	275 pcf	138 pcf	N/A

Based on a service limit state bearing pressure of 4,500 pounds per square foot (psf), we estimate that foundation settlements under the anticipated design loads for foundations bearing on the stiff to hard clay soils to be less than 1 inch. In general, foundations should be embedded a minimum of 2 feet below the lowest adjacent finished grades. Foundations next to utility trenches or easements should be embedded below a 1H:1V imaginary plane extending upward from the bottom edge of the utility trench or 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 foundation settlement.

Lateral loads acting on the structure may be resisted by friction developed 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 cubic foot (pcf), may be used to evaluate the passive pressure resistance for footings embedded and bearing on the stiff to hard clay soils. Unless covered by pavements or slabs, the passive resistance in the upper 12 inches should be neglected.

3.4.2 Static Lateral Earth Pressures

The static lateral earth pressure recommendations provided in the "Retaining Structures" section may be used for the segmental retaining walls.

3.4.3 Dynamic Lateral Earth Pressures

Dynamic lateral earth forces due to seismic loading may be estimated by using $15.6H^2$ pounds per lineal foot of wall length for level backfill conditions, where H is the height of the wall in feet. The seismic loading consists of peak ground horizontal acceleration adjusted for Site Class effects of 0.326g. 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.

3.4.4 Drainage

The segmental retaining walls 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 behind the reinforced backfill with a perforated pipe (perforations facing down) at the base of the wall system discharging to an appropriate outlet or weepholes. The backfill above the drain rock should consist of relatively impervious material (compacted on-site soils) to reduce the potential for significant water infiltration behind the retaining wall. A non-woven filter fabric, such as Mirafi 180N or equivalent should be used between the natural material at the excavation face and the wall backfill materials. 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.

3.4.5 Overall Stability

We have evaluated the overall stability of the segmental wall structures planned for the project. Based on our analyses, the capacity to demand ratio for short-term and long-term stability of the segmental retaining wall is greater than 1.0, which is the minimum capacity to demand ratio normally recommended. Based on our analyses, a minimum base width to wall height ratio of at least 1.25 is recommended for the segmental retaining wall structures.

3.4.6 Reinforced Fill and Backfill Materials

We believe that the reinforced fill material for the segmental retaining wall should consist of imported select granular fill materials. In general, the imported select granular fill materials 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 California Bearing Ratio (CBR) value of 25 or higher and a swell potential of one percent or less when tested in accordance with ASTM D 1883.

In addition, the reinforced fill material (imported select granular fill materials) should have an angle of internal friction of at least 34 degrees when tested by the standard direct shear test (ASTM D3080). The sample to be tested should be compacted to 95 percent relative compaction at moisture contents above the optimum.

Reinforced fill materials should be placed in level loose lifts not exceeding 8 inches in loose thickness and be compacted to at least 95 percent of the maximum dry density established in accordance with AASHTO T-180 at moisture contents above the optimum.

3.4.7 Geogrids

Geogrids should consist of geosynthetic reinforcement material having regular and defined open areas. The geogrid structure should be select high density polyethylene or polypropylene resin. The allowable long-term design strength (LTDS) of the geogrid should be determined by Geosynthetic Research Institute (GRI) Test Method GG4. Factors of safety for installation damage, biological and chemical degradation should be incorporated in the LTDS. In addition, an overall factor of safety of 1.5 should be used in determining the allowable LTDS.

During wall construction, the geogrids should be oriented with the highest strength axis perpendicular to the wall alignment. The geogrid should lay horizontally on compacted backfill, pulled taut, and anchored before placing backfill on the geogrid.

The geogrid should be continuous throughout the geogrid embedment lengths. Splices to connect two sections of geogrids may be used provided the splice connector is capable of providing 100 percent load transfer between the two geogrid sections.

3.5 Box Culvert Structures

We understand that a 4-foot by 10-foot reinforced concrete box culvert structure will be required for drainage purposes for the proposed roadway project. Recommendations provided in the “Retaining Structures” section may be used with additional recommendations provided below.

We understand that the box culvert structure will consist of cast-in-place concrete. As a minimum, we recommend that a 12-inch cushion layer consisting of aggregate subbase materials be provided between the box culvert base slab and the underlying bearing soils to provide uniform support. The aggregate subbase cushion layer should be compacted to at least 95 percent relative compaction. Aggregate subbase materials below the drainage structures should conform to the requirements stipulated in Subsection 703.17 of the State of Hawaii, Standard Specifications for Road and Bridge Construction, 2005.

Depending on the time of the year, soft and/or wet soils may be encountered at the foundation subgrade levels at some of the box culvert locations with respect to the existing drainage way alignment. Where encountered during construction, we recommend removing these soft and/or wet soils to expose the underlying firm soils prior to backfilling with compacted general fills. Contract documents should include additive and deductive unit prices for over-excavation and compacted fill placement to account for variations in the over-excavation and backfill quantities.

3.6 Pipe Culverts and Other Utility Trenches

We understand that pipe culverts (and other new utility lines) will be installed for the project. In general, good construction practices should be utilized for the installation and backfilling of the trenches for the new utilities. The contractor should determine the method and equipment to be used for trench excavation, subject to practical limits and safety considerations. In addition, the excavations should comply with the applicable federal, state, and local safety requirements. The contractor should be responsible for trench shoring design and installation.

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), also should 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 a level of 12 inches above the pipes to the top of the subgrades or finished grade may consist of the on-site soils generally less than 6 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.

3.7 Sidewalks and Concrete Gutters

We understand that concrete sidewalks and gutters will be required. The near-surface soils encountered in the borings generally consist of clays, which exhibit moderate to high expansion potential when subjected to moisture fluctuations. Therefore, we recommend providing a minimum of 24 inches of non-expansive select granular fill, such as aggregate subbase material, below the concrete flatwork to reduce the potential of appreciable structural distress to the lightly loaded concrete flatwork.

The soil subgrades below the sidewalks and concrete gutters should be scarified to a depth of at least 8 inches, moisture-conditioned to at least 2 percent above the optimum moisture, and compacted to no less than 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 determined in accordance with ASTM D1557. Optimum moisture is the water content (percentage by dry weight) corresponding to the maximum dry density. The subgrade should be kept moist until covered by the select granular fill (aggregate subbase material).

Due to the moderately to highly expansive nature of the subgrade soils, we recommend reinforcing the concrete sidewalks and concrete gutters with No. 3 reinforcing bars spaced at 12 inches in both directions as a minimum. In addition, we recommend providing crack control joints at intervals equal to the width of the sidewalks (or slabs) with expansion joints at right-angle intersections to reduce the potential for substantial shrinkage cracks in the slabs.

3.8 Pavement Design

3.8.1 Methodology of Pavement Design

Two types of pavement structural sections (flexible and rigid pavements) were considered in the pavement analyses. The flexible pavement sections presented herein were generally determined based on the methodology described in Chapter 3 of the revised Pavement Design Manual dated March 2002 prepared by the State of Hawaii - Department of Transportation, Highways Division, Material Testing and Research Branch. The pavement design methodology is based on the Hveem Stabilometer method developed and used by the California Department of Transportation (Caltrans).

The design procedures for the rigid pavement sections are generally based on the design procedures described in the Portland Cement Association “Thickness Design for Concrete Highway and Street Pavements”.

3.8.2 Design Traffic Loading Conditions/Traffic Index

Based on the design guidelines from the revised Pavement Design Manual dated March 2002, we believe that the roadways may be classified as a “High Volume Urban” roadway. Therefore, the pavements will need to be designed for a pavement life of 50 years. Peak Hour Traffic information after construction of the State Harbor Access Road (Year 2015) and Year 2030 was obtained from the Traffic Impact Report Update for Kapolei West prepared by Wilson Okamoto Corporation. The design traffic parameters were determined from the Peak Hour Traffic information and some assumptions based on our experience in the project vicinity.

We have assumed that completion of the roadway would be in the Year 2016. Also, we have assumed the completion of Kapolei West Development would be by Year 2030. The growth rates between Years 2016 and 2030 range between 7 and 11 percent. Beyond the Year 2030, we have assumed a growth rate of about 1 percent. The 24-Hour Truck Traffic and the Truck Traffic Distribution were also assumed since data were not available. The following table summarizes the design traffic parameters used in our pavement analyses.

DESIGN TRAFFIC PARAMETERS	
Design Period	50 Years
Average Daily Traffic (ADT)	Vehicles per day per direction
Year 2016 Year 2066	3,401 27,244
24-Hour Truck Traffic	17%
Type of Axle 2-axle 3-axle 4-axle 5-axle 6-axle 7-axle	Truck Traffic Distribution 27.3% 16.7% 18.3% 30.6% 4.3% 2.8%

Based on a design period of 50 years, the traffic volume, and the truck distribution information assumed, Traffic Index (TI) of 16.0 has been determined for the project. Since the roadway is anticipated to have high truck traffic with heavy loads, the

State Department of Transportation – Highways Division provided the following Equivalent Single Axle Load Constants (ESALC).

EQUIVALENT SINGLE AXLE LOAD CONSTANTS	
Type of Axle	**ESALC**
2-axle	597
3-axle	525
4-axle	1,162
5-axle	1,462
6-axle or more	1,174

Detailed analyses on the Traffic Index Determination are presented in Appendix D.

3.8.3 <u>Design Subgrade Conditions</u>

Based on our field exploration results and the current design concept, we anticipate that the pavement subgrade soils will likely consist of the on-site very stiff clay soils.

Laboratory Resistance (R) value tests performed on near-surface soils obtained from our field exploration indicated that the materials exhibited R-values of approximately less than 5 and 16. The test results are presented in Appendix B. Based on the laboratory test results, a design R-value of 4 was adopted in our pavement analyses for the subgrade materials. If site grading exposes soils other than those assumed, additional tests should be performed to confirm and/or revise the recommended pavement sections for actual field conditions.

It should be noted that the on-site clay and silty clay soils generally exhibit a moderate to high expansion potential when subjected to moisture fluctuations. Therefore, we also recommend providing a minimum of 24 inches of subbase materials below the concrete pavements for the rigid pavement sections.

Based on a design R-value of 4 adopted for the pavement subgrade soils as described above, a corresponding K-value of 60 pounds per square inch per inch of deflection (pci) is obtained from Figure 1-1 of the Pavement Design Manual prepared by the State of Hawaii - Department of Transportation, Highways Division, Material Testing and Research Branch. Based on a subbase thickness of 24 inches, the design "Subgrade/Subbase K" value may be increased to 150 pci in accordance with Figure 1-2 of the Pavement Design Manual. The Portland cement concrete should have a minimum flexural strength of 650 pounds per square inch (psi) at 28 days when tested in accordance with ASTM C78.

3.8.4 Design Pavement Section

Based on the relatively high traffic volumes anticipated for the project roadways, we believe that asphalt concrete base (ACB) should be used as the base material for the project. Therefore, pavement sections utilizing untreated aggregate base courses were not considered in our pavement design analyses.

As previously mentioned, the on-site clay soils generally exhibit a moderate to high expansion potential when subjected to moisture fluctuations. Therefore, we recommend providing a minimum of 24 inches of subbase materials below the concrete pavements for the rigid pavement design option. Based on the assumptions presented above, we considered the following pavement structural sections for use as the pavements for the project. Detailed analyses and calculations for the pavement design options are presented in Appendix C.

Flexible Pavement

4.5-Inch Asphaltic Concrete
11.0-Inch Asphalt Concrete Base
6.0-Inch Aggregate Base Course (95 Percent Relative Compaction
24.0-Inch Aggregate Subbase (95 Percent Relative Compaction)
45.5-Inch Total Pavement Thickness on Compacted Subgrade

Rigid Pavement

12.0-Inch Portland Cement Concrete
24.0-Inch Aggregate Subbase (95 Percent Relative Compaction)
36.0-Inch Total Pavement Thickness on Compacted Subgrade

An economic analysis was performed on the pavement structural sections presented above to evaluate the initial construction cost and life cycle cost of the pavement sections. Based on our cost comparisons of the pavement design options, it appears that the rigid pavement section would result in the most economical pavement design over the design life of the pavement. The rigid pavement structural section consists of 12.0 inches of Portland cement concrete (PCC) on 24.0 inches of aggregate subbase (ASB) course. The results of our economic analysis are presented in Appendix C.

3.8.5 Subgrade Preparation & Fill Materials

For subgrade preparation and fill materials, see “Site Grading” section above.

3.8.6 Compaction Requirements

Fill and backfill materials should be moisture-conditioned to 2 percent 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. Aggregate subbase materials required for the new pavements should conform to the requirements stipulated in Subsection 703.17 of the State of Hawaii, Standard Specifications for Road and Bridge Construction, 2005 (HSS). The aggregate subbase should be moisture-conditioned to above the optimum moisture content, placed in 8-inch level loose lifts, and compacted to no less than 95 percent of the maximum density according to AASHTO T 180.

Asphalt concrete base (ACB) material should consist of asphalt-treated basaltic aggregate, placed in a layer not to exceed 6 inches in compacted thickness, and compacted to at least 92 percent of the maximum theoretical specific gravity determined in accordance with AASHTO T 209 (ASTM D2041).

Asphaltic concrete (AC) material should be constructed in general accordance with Section 401 - Hot Mix Asphalt Pavement of the State of Hawaii, Standard Specifications for Road and Bridge Construction (2005) and subsequent amendments.

Field density tests should be performed on the compacted fills and backfills in general accordance with ASTM D1556. In general, field density tests should be performed at the frequencies presented in the following table.

Material	Location of Material	Test Frequency
Aggregate Subbase	Pavements & Shoulders	One test per 100 lineal feet of roadway per lift.
Subgrade (Clays & Sandy Gravels)	Pavements & Shoulders	One test per 100 lineal feet of roadway per lift.
Trench Backfill	Utility Trenches	One test per 200 lineal feet of trench per lift of backfill.

3.8.7 Pavement Drainage

One of the primary distress mechanisms in pavement structures is pumping due to saturation of the base, subbase, and/or subgrade soils. It should be noted that the clayey subgrade soils are susceptible to softening and loss of strength upon saturation. Therefore, special attention should be given to the surface and subsurface drainage of the pavements. As a minimum, the pavement surface should be sloped and drainage gradients should be maintained to carry surface water off the pavement to appropriate drainage structures. Surface water ponding should not be allowed on-site during or after construction. To collect infiltrated water into the pavement structural section, we recommend providing a pavement edge drain system hydraulically connected to the aggregate base layer along the sides of the new roadway.

We envision that an edge drain will be needed to collect and discharge infiltrated water from the aggregate base layer. In general, the edge drain should consist of a 6-inch PVC perforated pipe (with perforations facing down) surrounded by untreated permeable base. A permeable separator (non-woven filter fabric) conforming to Section 648 of the HSS should be placed around the untreated permeable base to reduce the potential for migration of fines into the drainage material. Care should be exercised when placing the subbase, base, and/or construction of the surface course to avoid damaging the edge drain collector pipe. A typical edge drain detail is provided on Plate 4 for reference. The edge drains should be daylighted into appropriate drainage structures for proper discharge of infiltrated water.

3.9 Infiltration Testing

In order to characterize the water infiltration rates at the project site, we conducted three cased borehole infiltration tests at the site. The depth to the bottom of the cased borehole was 5 feet below the existing ground surface extending into the alluvial clay deposit near Boring Nos. 4 and 6, and into the coralline detritus deposit in Boring No. 10. Final infiltration rate of 0.1 inch per hour for the bottom of the casing into the alluvial clay was measured. For the bottom of the casing into the coralline detritus deposit, final infiltration rate of 0.6 inch per hour was measured. The results of the field infiltration testing are provided in Appendix C.

Based on the test results, it appears that the alluvial clay have relatively low infiltration rate. A higher infiltration rate was measured in the coralline detritus deposit encountered in Boring No. 10. Since the infiltration rate for the test performed at Boring No. 10 is above the minimum soil infiltration rate of 0.5 inch per hour, underground infiltration system extending into the underlying coralline detritus deposit encountered at about 0.5 feet depth may be considered near the Boring No. 10 area. For a shallower basin (less than 5-foot depth) near the Boring No. 10 area, we anticipate that minimum soil infiltration rate of less than 0.5 inch per hour may be encountered. It should be noted that in coralline materials the movement of water is dependent upon many factors, such as grain size of the clasts forming the matrix, depositional sorting and discontinuities and the degree of cementation. Low infiltration rates of less than 0.5 inch per hour were measured in the alluvial clay layer near the Boring Nos. 4 and 6 areas. Therefore, a detention system may be required in these areas.

3.10 Design Review

Preliminary and final drawings and specifications should be forwarded to Geolabs for review and written comments prior to construction bidding. This review is necessary to evaluate general conformance of the plans and specifications with the intent of the geotechnical recommendations provided herein. If this review is not made, Geolabs cannot be responsible for misinterpretation of our recommendations.

3.11 Construction Monitoring

Geolabs should be retained to provide geotechnical engineering services during construction. The critical items of construction monitoring that require "Special Inspection" include observation of the settlement monitoring program, subgrade preparation, fill and backfill placement, retaining wall construction, and pavement installation. This is to confirm compliance with 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 exposed 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

SECTION 4. LIMITATIONS

The analyses and recommendations submitted herein are based, in part, upon information obtained from in-house literature research and field exploration. Variations of subsurface conditions between and beyond the field exploration points 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 provided herein.

The field boring and bulk sample locations indicated in this report are approximate, having been taped from features shown on the General Site Plan dated January 2015 by Mitsunaga & Associates, Inc. Elevations of the borings were estimated based on interpolation between the spot elevations and contour lines shown on the same plan. The physical locations and elevations of the borings 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 soil/rock types and, as such, may denote a gradual transition. Water level data from the borings were measured at the times shown on the graphic representations and/or presented in the text of this report. These data have been reviewed and interpretations made in the formulation of this report. However, it must be noted that fluctuation may occur due to variation in rainfall, temperature, and other factors.

This report has been prepared for the exclusive use of Mitsunaga & Associates, Inc. for specific application to the proposed *Kapolei Interchange Complex Phase 3* 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 engineer in the preparation of the design drawings related to the site grading, retaining walls and pavement design for the project only. Therefore, this report may not contain sufficient data, or the proper information, to serve as a basis for construction cost estimates. A contractor wishing to bid on this project is urged to retain a competent geotechnical engineer to assist in the interpretation of this report and/or in the performance of additional site-specific exploration for bid estimating purposes.

The owner/client should be aware that unanticipated subsurface 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.

This geotechnical engineering exploration conducted at the project site was not intended to investigate the potential presence of hazardous materials existing at the site. It should be noted that the equipment, techniques, and personnel used to conduct a geo-environmental exploration differ substantially from those applied in geotechnical engineering.

END OF LIMITATIONS