GEOTECHNICAL ENGINEERING EXPLORATION
KAMEHAMEHA V HIGHWAY (ROUTE 450)
KAWELA BRIDGE REPLACEMENT
MP 5.110 TO MP 5.118
ISLAND OF MOLOKAI, HAWAII

W.O. 5909-00(A) OCTOBER 6, 2010

Prepared for

KAI HAWAII, INC.

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THIS WORK WAS PREPARED BY
ME OR UNDER MY SUPERVISION.

SIGNATURE

4-30-12
EXPIRATION DATE
OF THE LICENSE

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Hawaii • California

GEOLABS, INC.
Geotechnical Engineering and Drilling Services

October 6, 2010
W.O. 5909-00(A)

Mr. Michael Hunnemann
KAI Hawaii, Inc.
31 North Pauahi Street, 2nd Floor
Honolulu, HI 96817

Dear **Mr. Hunnemann:**

Geolabs, Inc. is pleased to submit our report entitled "Geotechnical Engineering Exploration, Kamehameha V Highway, Kawela Bridge Replacement, MP 5.110 to MP 5.118, Island of Molokai, Hawaii" prepared for the design of the replacement bridge.

Our work was performed in general accordance with the scope of services outlined in our revised fee proposal of September 13, 2005.

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 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.

Robin M. Lim, P.E.
Vice President

RML:RP:as

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SUMMARY OF FINDINGS AND RECOMMENDATIONS

Our exploratory borings drilled at the bridge abutment locations encountered surface fills consisting of very dense silty sands extending to depths of about 5 to 6 feet below the existing ground surface. The fills were underlain by lagoonal deposits consisting of very loose to loose sands and gravel extending to depths of about 52 to 55 feet below the existing ground surface. The lagoonal deposits were underlain by alluvial deposits consisting of very dense basaltic boulders and cobbles extending to the maximum depths explored of about 72 to 77 feet below the existing ground surface. We encountered groundwater at depths between 5 and 6 feet below the existing ground surface.

We understand a concrete slab will be constructed covering the bottom of the channel at the bridge location. Therefore, the effects of scour on the bridge foundations were not considered in our bridge foundation analyses. Based on the subsurface conditions encountered, the foundation loads provided, and considering the logistics of construction at the project site, we recommend utilizing a deep foundation system consisting of 7.5-inch diameter micropiles embedded into the very dense basaltic boulders and cobbles to support the replacement bridge structure. In general, we recommend extending the micropiles at least 20 to 23 feet into the cobble and boulder layer in order to provide the required support for the new bridge abutment. The corresponding tip elevations for the micropiles would be approximately -70 and -73 feet MSL in order to provide a Strength Limit State compression load capacity of 100 and 112 kips, respectively. We also recommend the top 50 feet of the micropile (extending down to Elevation -50 feet MSL) be permanently cased.

It should be noted that potentially difficult drilling conditions may be encountered during micropile installation due to the presence of very loose lagoonal soils, hard to very hard basaltic boulders and cobbles below the loose lagoonal soils, and shallow groundwater levels encountered in the borings. Therefore, we recommend providing a permanent steel casing extending from the top of the micropile (embedded into the footing) to the top of the very dense boulder and cobble bearing layer (approximately Elevation -50 feet MSL).

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

END OF SUMMARY OF FINDINGS AND RECOMMENDATIONS

SECTION 1. GENERAL

1.1 Introduction

This report presents the results of our geotechnical engineering exploration performed for the Kawela Bridge Replacement project along Kamehameha V Highway (Route 450), MP 5.110 to MP 5.118, on the Island of Molokai, Hawaii. The project location and general vicinity are shown on the Project Location Map, Plate 1.

This report summarizes our findings and presents our geotechnical engineering recommendations derived from our field exploration, laboratory testing, and engineering analyses. These recommendations are intended for the design of foundations, retaining structures, and site grading only. The findings and recommendations presented herein are subject to the limitations noted at the end of this report.

1.2 Project Considerations

The replacement bridge project site is along Kamehameha V Highway across Kawela Gulch on the Island of Molokai, Hawaii. Based on the information provided, the existing bridge (known as Kawela Bridge) was built in 1940 and measures 44 feet long by 28 feet wide. The existing bridge is supported by two abutments and one intermediate pier in relatively good condition.

Based on field observations during our field exploration, Kawela Stream was relatively active with fast flowing shallow water. The bridge appears to be low, and the opening from the bottom of the bridge to the water surface was about 2 feet at the time of our site visit. We understand the stream overflows the banks and floods the bridge and surrounding area during the rainy season. Both the upstream and downstream banks are heavily vegetated, and numerous cobbles and boulders were observed on the streambed. We understand the existing bridge is hydraulically inadequate and does not conform to current State of Hawaii, Department of Transportation (HDOT) and Federal Highway Administration (FHWA) design and seismic standards.

We understand the current design concept involves demolishing the existing bridge and replacing it with a new concrete bridge with a bikeway/pedestrian walkway that will meet current HDOT and FHWA standards. In addition, a detour road using reinforced concrete pipe (RCP) drainage culverts at the downstream side of the existing bridge will be built to allow traffic to traverse around the bridge construction area.

Based on the new bridge plans, we understand the new bridge structure will be a 60-foot long by 44-foot wide single-span concrete girder bridge. The planned finished deck elevation of the new bridge structure is set at about +6.9 feet Mean Sea Level (MSL). The bottom of footing elevation for the abutments is set at approximately -4.9 feet MSL. The bottom of the stream channel at the new bridge location is set at about Elevation –2.4 feet MSL.

In order to construct the new longer bridge structure, the existing stream channel opening will need to be widened. Therefore, excavation into the existing stream banks on the order of about 8 feet at both sides of the bridge abutment will be required. In general, the widened stream channel will be lined with Cement Rubble Masonry (CRM) walls immediately upstream and downstream of the new bridge structure. In addition, a 12-inch thick concrete bed will be constructed at the bottom of the stream channel to reduce the effect of stream scour on the new bridge foundation. It should be noted that the design of pavement structural sections is presented in the pavement justification report prepared by our office (transmitted separately).

1.3 Purpose and Scope

The purpose of our geotechnical engineering exploration is to obtain an overview of the subsurface conditions to develop a soil/rock data set to formulate geotechnical recommendations for the design of the bridge replacement project. Our work was performed in general accordance with our fee proposal dated September 13, 2005. Our scope of work generally consisted of the following tasks and work efforts:

1. Reconnaissance of the project site by our engineers to observe the existing field conditions.
2. Review of available in-house soil and geologic information around the replacement bridge project location.
3. Application of the necessary excavation permits from the State of Hawaii – Department of Transportation, Highways Division, Maui District, prior to drill crew mobilization (including preparation of a traffic control plan).

4. Coordination of the utility toning with the various utility companies and clearance of the proposed boring locations by our field geologist.

5. Provision of traffic control at the proposed boring locations during our field exploration program.

6. Mobilization and demobilization of truck-mounted drill equipment, water truck, and operators to the project site and back.

7. Drilling and sampling of four borings extending to depths of about 72 to 77 feet below the existing ground for a total of about 293 lineal feet of field exploration.

8. Coordination of the field exploration and logging of the borings by our field geologist.

9. Laboratory testing of selected soil and/or rock samples obtained during the field exploration as an aid in classifying the materials and evaluating their engineering properties.

10. Analyses of the field and laboratory data to develop geotechnical recommendations for design of the bridge replacement project, such as seismic design considerations, scour analyses, bridge foundations, earthwork, pavements, and construction considerations.

11. Preparation of a formal report and a pavement justification report summarizing our work on the project and presenting our findings and geotechnical recommendations.

12. Coordination of our work on the project by our engineer.

13. Quality assurance of our overall work on the project and client/design team consultation by our principal engineer.

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

Detailed descriptions of our field exploration methodology and the Logs of Borings are presented in Appendix A. Results of the laboratory tests performed on selected soil samples obtained from our field exploration are presented in Appendix B.

END OF GENERAL

SECTION 2. SITE CHARACTERIZATION

2.1 Regional Geology

The Island of Molokai was built by the extrusion of basaltic lava flows from two shield volcanoes during the early to middle Pleistocene Epoch. The two shield volcanoes comprising the Island of Molokai are known as East Molokai Mountain and West Molokai Mountain. The new bridge project site is on the southeastern flank of the East Molokai Mountain.

The East Molokai Mountain was originally a typical elongated basaltic/andesitic shield-shaped dome. It was built over the northwest and east-trending rifts, with a steep slope on the north side where the lava flows plunged into deep water, and a gentle slope on the west side where the lava flows banked against the West Molokai dome.

During the Pleistocene Epoch (Ice Age), many sea level changes occurred as a result of widespread glaciation in the continental areas of the world. As the great continental glaciers advanced and accumulated, the level of the ocean fell due to a lower quantity of water available to fill the oceanic basins. Conversely, as the glaciers receded, or melted, global sea levels rose because of the increase in available water. The land mass of the Island of Molokai remained essentially stable during these changes, and the fluctuations were eustatic in nature. These glacio-eustatic fluctuations resulted in stands of the sea that were both higher and lower relative to the present sea level on Molokai.

The processes of erosion and deposition were affected by these glacio-eustatic sea level fluctuations. When the sea level was low, the erosional base level was correspondingly lower, and valleys were carved to depths below the present sea level. When the sea level was high, the erosional base level was raised such that sediments accumulated at higher elevations.

In the mountainous regions of the Island of Molokai, the erosional processes are dominated by detachment of soil and rock masses from the valley walls that are transported down slope toward the axis of a valley primarily by gravity as colluvium. Once these materials reach the stream in the central portion of a valley, alluvial processes become dominant, and the sediments are transported and deposited as alluvium.

In general, stream flows are intermittent and flashy, such that the stream flows transmit large volumes of water for very short duration. Because of this, transport of sediments is intermittent, and the bulk of the stream's hydraulic load consists of a poorly sorted mixture of boulders, cobbles, gravel, sands, and fines. When the erosional base levels change, these sediment loads are left as deposits.

When deposits are left in place for long periods of time, chemical processes begin to alter the materials simultaneously causing a breakdown or weathering of the material. Chemical processes also cause induration, or cementation, of the coarse-grained portion of the sediment resulting in a poorly consolidated sedimentary rock, or conglomerate. Simultaneously, erosion continues in the areas above the valley floors and upstream in headwaters. This continued erosion generates materials, which are transported down slope, covering the older alluvial deposits.

2.2 Site Description

The project site is along Kamehameha V Highway (Route 450) between MP 5.110 to MP 5.118 on the Island of Molokai, Hawaii. The existing bridge, which spans across Kawela Stream, is a two-lane, two-span concrete structure supported by two abutments and one center pier. The bridge measures 44 feet long and about 28 feet wide. The bridge center pier, abutments and wing walls are of cement rubble masonry (CRM) and concrete construction.

Based on the topographic survey, the existing bridge deck elevations is between about +6 and +7 feet MSL. At the time of our field exploration, we observed relatively fast flowing shallow water in the stream. The opening between the bottom of the bridge and the water surface is approximately 2 feet.

2.3 Subsurface Conditions

Our field exploration program consisted of drilling and sampling four borings, designated as Boring Nos. 1 through 4, near the proposed bridge abutment locations extending to depths of about 72 to 77 feet below the existing ground surface. The approximate boring locations are shown on the Site Plan, Plate 2.

In general, the borings encountered a layer of surface fill that primarily consisted of very dense silty sands extending to depths of about 5 to 6 feet below the existing grade. The surface fill was underlain by lagoonal deposits consisting of very loose to loose sands and gravel extending to about 52 to 55 feet below the existing ground surface. Below the lagoonal deposits, our borings encountered very dense alluvial deposits consisting of cobbles and boulders. The alluvial deposits extended to the maximum depths drilled of approximately 77 feet below the existing ground surface.

We encountered groundwater in our borings at depths of about 5 feet below the existing ground surface at the time of our field exploration. It should be noted that groundwater levels are expected to fluctuate depending on tides, seasonal rainfall, time of year, surface runoff, and other factors. Considering that the bridge is adjacent to a stream, the groundwater level also will vary in response to the water level in the stream.

Detailed descriptions of the field exploration methodology are presented in Appendix A. Descriptions and graphic representations of the materials encountered in the borings drilled are provided on the Logs of Borings in Appendix A. Results of the laboratory tests performed on selected soil samples retrieved from our field exploration are presented in Appendix B.

END OF SITE CHARACTERIZATION

SECTION 3. DISCUSSION AND RECOMMENDATIONS

As previously mentioned, the project site is underlain by fills extending to depths of about 5 to 6 feet below the existing ground surface. The fills were underlain by very loose to loose lagoonal deposits extending to depths of about 52 to 55 feet below the existing ground surface. The very loose to loose lagoonal deposits were underlain by alluvial deposits consisting of very dense basaltic boulders and cobbles extending to the maximum depths explored of about 72 to 77 feet below the existing ground surface. Groundwater was encountered in our borings at depths of about 5 feet below the existing ground surface.

Based on the subsurface conditions encountered at the project site, we recommend utilizing deep foundations consisting of micropiles for foundation support of the proposed replacement bridge structure. The micropile foundations would derive support principally from adhesion between the micropiles and the very dense basaltic boulders and cobbles encountered in our borings at depths starting at about 55 feet. Based on the foundation loads, we recommend using micropiles with a minimum grout bulb diameter of 7.5 inches with a tip elevation of approximately -70 and -73 feet MSL for the bridge foundations. In addition, we understand a concrete slab is planned at the bottom of the stream channel elevation as a counter-measure to reduce the potential for scour at the bridge location. Therefore, the effects of scour at the bridge location were not considered in our bridge foundation analyses.

It should be noted that potentially difficult drilling conditions will likely be encountered during installation of the micropiles due to the presence of the very loose lagoonal deposits and the presence of very dense basaltic boulders and cobbles. Therefore, a permanent steel casing will be required to reduce the potential for caving-in of the drilled holes during the drilling operation. Special drilling tools also will be required in order to advance the drilled holes considering the presence of basaltic boulders and cobbles at the site. Detailed discussions and recommendations for the bridge structure design are presented in the following sections.

3.1 General Information

Based on the information provided, the new replacement bridge structure will be a single-span concrete bridge spanning approximately 60 feet from abutment to abutment. The subsurface conditions at the bridge abutment locations were explored by drilling four borings extending to depths ranging from about 72 to 77 feet below the existing ground surface. Descriptions and graphic representations of the materials encountered in the borings are provided on the Logs of Borings in Appendix A.

3.2 Seismic Design Considerations

Based on the design criteria provided by the State of Hawaii - Department of Transportation, the Kawela Bridge Replacement project will need to be designed based on a peak horizontal bedrock acceleration (PBA) coefficient of 0.25g. Based on the average penetration resistance of the subsurface materials encountered in the borings, the project site may be classified as a "Soft Soil" profile for seismic design considerations. Therefore, the project site may be designed based on a Site Class E soil profile type based on AASHTO Guide Specifications for LRFD Seismic Bridge Design (May 2007).

3.3 Stream/Channel Material for Scour Analysis

One of the most common causes of bridge failure stems from scouring of bridge foundations from flood or other water flow damage. Therefore, the foundation design of the bridge abutments will need to take into consideration the potential for stream/channel scour. Scour is the result of erosive action of flowing water, excavating and carrying material away from the bed and banks of streams/channels. Total scour over a period of time generally consists of three components: 1) Aggregation and Degradation; 2) Contraction Scour; and 3) Local Scour. The rates of scour depend on a number of factors such as the shape and dimensions of a pier or abutment, depth of flow, velocity of approach flow, size and gradation of stream/channel bed material, and bed configuration.

One of the factors affecting the scour depth is the grain size characteristics of the stream bed material. The median diameter of the stream bed material (D_{50}), in conjunction with the depth of flow and flow velocity, is used to calculate flow velocity of stream bed materials in scour depth analysis. Based on our field exploration, both upstream and downstream banks are heavily vegetated and numerous cobbles and boulders were observed on the streambed. In addition, Kawela Stream was quite active with fast flowing shallow waters.

We believe that the stream bed materials encountered at the project site are susceptible to erosion. Therefore, we understand counter-measures to reduce the potential for scour at the bridge foundation will be implemented at the replacement bridge location. Based on the available information, a 12-inch concrete slab with cutoff walls will be constructed at the bottom of the stream channel elevation as a counter-measure to reduce the potential effects of scour for the bridge foundation. Therefore, the effects of scour on the bridge foundation will not be considered in our bridge foundation analyses.

3.4 Micropile Foundations

Based on the anticipated structural loads acting on the abutment structure and the subsurface conditions (loose lagoonal deposits and cobbles and boulders) encountered at the project site, we recommend utilizing a deep foundation system to support the new bridge structure. Considering the remote location of the project and the lack of readily available equipment for drilled shaft concrete, we recommend utilizing partially-cased micropile foundations to support the new bridge structure.

In general, the cased micropile foundation system consists of a small diameter (usually less than 12 inches), drilled and grouted, foundation element with steel reinforcing. The micropile foundation typically is constructed by drilling a hole (with or without casing), placing reinforcing steel in the hole, and grouting the hole. Micropiles are desirable because they can be installed readily in access restrictive environments and in numerous soil types and ground conditions. In addition, the micropile installation generally causes minimal disturbance to adjacent structures, the adjacent soils, and the environment.

Based on the subsurface conditions encountered at the project site, we recommend utilizing a micropile system with a grout bulb diameter of at least 7.5 inches to support the new bridge structure. Based on the information provided by the project structural engineer, we understand that the front row of micropiles will be subjected to a slightly higher load demand. Therefore, we recommend designing each micropile based on a compressive load capacity of 100 and 112 kips under the strength limit state for the rear and front rows of the micropile foundation, respectively. The ultimate single micropile load capacity will need to be on the order of about 180 and 205 kips per micropile for the rear and front rows, respectively, utilizing a resistance factor of 0.55. To achieve an ultimate single micropile compression load capacity of 180 and 205 kips per micropile for the extreme event limit state, we believe the micropiles should be embedded a minimum of 20 and 23 feet into the bearing layer (basaltic cobbles and boulders) encountered at about 55 feet below the existing ground (at about Elevation -50 feet MSL).

We expect that the micropiles would derive its vertical support primarily from skin friction between the grout and the surrounding dense basaltic cobbles and boulders. Based on the subsurface conditions at the project site, we recommend casing the micropiles (permanent casing) at the top. The permanent casing should have an outside diameter (OD) of at least 7.5 inches (same as the grout bulb size), and the permanent casing should extend about 55 feet (to about Elevation -50 feet MSL) below the existing ground surface. Due to the significant depth of very loose lagoonal soil deposits at the project site, we recommend conducting a micropile static load test to further evaluate and validate our assumptions in providing the above micropile recommendations for support of the new structural elements at the project site.

3.4.1 Micropile Load Test Program

It should be noted that the compressive load capacity of the micropiles is highly dependent on the drilling procedures and the grouting methods employed by the contractor to install the micropile. Therefore, the compressive load capacity of the micropile may vary considerably between different contractors and micropile foundation systems. In order to determine whether the contractor’s methods of micropile installation are adequate and to determine the ultimate compressive load capacity, we recommend performing a pre-production compressive load test on a sacrificial micropile. In general, the purpose of the pre-production load test on a micropile is to fulfill the following objectives:

- To examine the adequacy of the methods and equipment proposed by the contractor to install the micropiles to the depths required
- To assess the contractor's method of drilling and grout injection
- To evaluate the degree of long-term settlements of the micropile bearing in the soft to very loose to loose lagoonal deposits and dense alluvial deposits

In general, the pre-production load tests should be performed in accordance with ASTM D 1143 (Standard Loading Procedure). Based on experience, we believe the load test should be conducted no earlier than 7 days after completion of the micropile installation to allow the grout adequate time to cure. Two (or four) additional micropiles may be used for reaction during the compressive load testing of the pre-production load test micropile. The reaction micropiles may be installed to depths as deep as the load test micropile to provide adequate reaction in uplift (to be determined by the contractor).

The load test micropile should be loaded gradually to at least 200 percent of the strength limit state load capacity in compression. We recommend holding the maximum test load (200 percent of the design load) for a minimum of 4 to 8 hours depending on the recorded movements of the load test micropile. The pre-production load tests are an integral part of the micropile foundation design. Therefore, we recommend a Geolabs representative observe the pre-production load test program (sacrificial micropile installation and load test).

In addition to the pre-production load test, we also recommend performing pullout tests (proof tests) on selected micropiles during construction to confirm the load carrying capacity of the installed micropiles. We recommend testing a minimum 10 percent of the total number of micropiles (or minimum of four micropiles) for pullout. The pullout tests should consist of subjecting the micropile to at least 150 percent of the design loads, and the maximum test load should be held for at least 10 or 60 minutes. Pullout tests on the micropiles also are integral parts of the design of the micropile foundation system.

3.4.2 Lateral Load Resistance

In general, lateral load resistance of micropile is a function of the stiffness of the surrounding soil, the stiffness of the micropile, allowable deflection at the top of micropile, and induced moment in the micropile. Our lateral load analyses utilized the computer program GROUP 3D to evaluate the lateral deflection and reactions of the micropile. The program was developed to compute the distribution of loads (vertical, lateral, and overturning moment in up to three orthogonal axes) from the pile cap to the micropiles arranged in a group. The program solves the non-linear response of each micropile under combined loadings and assures compatibility of geometry and equilibrium of forces between the applied external loads and the reactions of each micropile head.

The group geometry, micropile properties, soil properties, and loading conditions were input into the GROUP 3D program based on available information and structural loads provided by the project structural engineer. Based on the current design concept, we understand two rows of ten micropiles for a total of 20 micropiles would be used to support each abutment. It should be noted that the micropiles were modeled based on a free-head (pinned) connection at the pile cap as requested by project structural engineer.

The following two micropile section properties were used in our geotechnical design analyses to provide some level of conservatism in the design and for comparison purposes.

- Upper Micropile Section (Cased Section): 7.5-inch diameter micropile using 60 percent of the pile stiffness (0.6 EI) based on f'_c = 4,000 psi (reducing the micropile stiffness due to the potential for cracking of the grout and corrosion of the steel casing).

- Lower Micropile Section (Uncased Section): 7.5-inch diameter micropile using 60 percent of the pile stiffness (0.6 EI) based on f'_c = 4,000 psi (reducing the micropile stiffness due to the potential for cracking of the grout).

The stiffness of the micropile cased section was calculated by discounting the steel casing thickness by 0.125 inches due to the loss of steel from potential corrosion and the potential for cracking within the cement grout in the micropile to resist lateral loads. The lateral deflections and maximum induced moments of the micropiles, based on a free-head condition at the top of the micropiles, are presented in the following table.

LATERAL DEFLECTION AND MAXIMUM INDUCED MOMENT IN THE MICROPILE FOUNDATIONS (Extreme Event Limit State)						
Location	Loading Condition	Vertical Load (kips)	Lateral Load (kips)	Lateral Deflection (inches)	Maximum Moment Induced (kip-feet)	*Depth to Maximum Moment (feet)
Abutment Nos. 1 & 2	Extreme Event Limit State	1,000	40	0.5	7	7
			55	1.0	11	8
			71	2.0	18	9
			35**	0.5	5	6
			47**	1.0	10	7
			60**	2.0	14	8

Notes:
* The depth to maximum moment is measured from the top of the micropile.
** Total stiffness was reduced to 0.6EI to model the loss of stiffness due to steel casing corrosion and cracking of the grout.

The above maximum moment induced is the moment induced in each micropile as a result of the total lateral load applied to the abutment structure. The center-to-center spacing of the micropiles in the pile cap is about six times the nominal micropile diameter.

3.4.3 Micropile Foundation Settlements

Settlements of the micropile foundations will result primarily from elastic compression of the micropile member and subgrade response. We estimate the total settlement of the micropile-supported foundations to be 0.5 inches or less with differential settlements between footings supported on micropiles not exceeding about one-half of the total settlement. We believe these settlements are essentially elastic and should occur as the loads are applied.

3.4.4 Construction Considerations

A specialty subcontractor, experienced in the construction of a cased micropile foundation system of similar size and subsurface conditions, should perform the micropile installation. Based on the subsurface conditions at the project site, it should be noted that hard drilling conditions (such as cobbles and boulders) will be encountered at the project site.

Due to the specialized nature of the micropile foundation construction, observation of the micropile foundation installation system and testing of the micropiles should be designated a "Special Inspection" item. Therefore, observation of the micropile installation operations by a Geolabs representative is necessary to confirm our design assumptions.

3.5 Retaining Structures

Based on the information provided, we understand retaining structures, such as the abutment walls and wing walls, on the order of about 10 feet in height will be required for the replacement bridge project. In general, foundations for the abutment structure and wing walls (structural elements attached to the bridge structure) should be designed based on the recommendations for support of the bridge structure (micropile foundations). The following guidelines may be used in designing the retaining structures for this project.

3.5.1 Static Lateral Earth Pressures

The abutment walls and wing walls for the replacement bridge should be designed to resist the lateral earth pressures due to the adjacent soils and surcharge effects caused by loads adjacent to the abutment walls. The recommended lateral earth pressures for design of the retaining structures, expressed in equivalent fluid pressures of pounds per square foot per foot of depth (pcf), are presented in the following table.

LATERAL EARTH PRESSURES FOR DESIGN OF RETAINING STRUCTURES			
Backfill Condition	Earth Pressure Component	Active (pcf)	At-Rest (pcf)
Level Backfill	Above Groundwater	40	58
	Below Groundwater	80	88

Backfill behind the retaining structures (above the groundwater level) may consist of the on-site soils or select granular fills (Type A Structure Backfill) compacted to at least 95 percent relative compaction. Because shallow groundwater conditions are anticipated, backfill materials below the groundwater level should consist of free-draining granular materials, such as AASHTO M43, No. 67 gradation (ASTM C 33, No. 67 gradation), wrapped on all sides with non-woven filter fabric (Mirafi 180N or equivalent). The free-draining granular materials should be used up to a level of about 12 inches above the groundwater level to facilitate compaction of the backfill materials.

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.

3.5.2 Dynamic Lateral Earth Forces

We understand dynamic lateral earth forces will need to be considered in the design of the retaining structures based on LRFD methods. An appropriately reduced factor of safety may be used when dynamic lateral earth forces are accounted for in the design of retaining structures. Based on a peak bedrock acceleration of 0.25g and modified to account for the soil effects based on Site Class E, we calculated a peak ground acceleration (PGA or a_{max}) of 0.3625g. Based on this level of peak ground acceleration or horizontal seismic acceleration, the dynamic lateral earth forces acting on the abutment in the event of an earthquake are presented below based on the degree of wall displacements. In general, a force due to dynamic lateral earth pressures acting on the retaining structure will increase with decreased lateral movement of the structure.

DYNAMIC LATERAL EARTH FORCES FOR RETAINING STRUCTURES	
Lateral Movement (inches)	Dynamic Lateral Earth Forces (H^2 pounds per linear foot)
0.5	36.8
1.0	26.6
1.5	21.5
2.5	13.5
3.5	7.8

It should be noted that the above table only applies to level backfill conditions, where H is the height of the wall in feet. The resultant force should be assumed to act through the mid-height of the wall. The dynamic lateral earth forces presented above are in addition to the static lateral earth pressures.

3.5.3 Drainage

Retaining structures should be well drained to reduce the build-up of hydrostatic pressures. A typical drainage system should consist of permeable material, such as AASHTO M 43, No. 67 gradation material, placed near the bottom and along the length of the wall discharging to an appropriate outlet or weepholes. As an alternative, the drainage system may consist of about 1 cubic foot of permeable material, such as AASHTO M 43, No. 67 gradation material, wrapped with non-woven, filter fabric at each of the weephole locations. The weepholes should be spaced not more than 8 feet apart.

Backfill behind the permeable drainage zone should consist of Type A Structure Backfill Material conforming to Section 703.20 of the HSS. Unless covered by concrete slabs or pavements, the upper 12 inches of backfill should consist of relatively impervious material to reduce the potential for significant water infiltration behind the walls. In addition, the backfill from the bottom of the wall to about the elevation of the weepholes should consist of relatively impervious soil backfill, such as the on-site soils, to reduce the potential for excess water infiltration into the foundation materials.

3.6 <u>Stream Channel Invert Slab</u>

As mentioned previously, the bottom of the stream channel will be lined with a 12-inch thick concrete slab as a counter-measure to reduce the potential for scour at the bridge location. To provide uniform support, we recommend providing a cushion layer consisting of 8 inches of open-graded gravel, such as AASHTO M43 Size No. 67 gradation, below the concrete slab. Soft and/or loose materials encountered at the bottom of the cushion layer should be over-excavated to expose firm material. The resulting over-excavation should be lined with a non-woven filter fabric (Mirafi 180N or equivalent) and backfilled with open-graded gravel, such as AASHTO M43 Size No. 67 gradation.

3.7 <u>Site Grading</u>

Based on the existing topography and the design finished grades of the new bridge approaches, the extent of grading required to construct the proposed replacement bridge will consist of cuts up to about 7 feet deep and fills up to about 2 feet thick. In general, grading operation should conform to Section 200 of the Hawaii Standard Specifications for Road and Bridge Construction, 2005 (HSS), and the site-specific recommendations contained in this report.

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

In general, areas to receive fills should be cleared of vegetation and deleterious materials. The resulting grub/spoil material should be disposed of properly off-site. Soft, weak, or yielding areas disclosed during clearing operations should be over-excavated to expose firm or dense ground, and the resulting excavation should be backfilled with general fill materials moisture-conditioned to above the optimum moisture content and compacted to a minimum of 90 percent relative compaction. After clearing and grubbing, the exposed subgrades should be scarified to a minimum depth of 8 inches, moisture-conditioned to above the optimum moisture content, and compacted to a minimum of 90 percent relative compaction. Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same soil established in accordance with AASHTO T-180 (or ASTM D 1557). Optimum moisture is the water content (percentage by dry weight) corresponding to the maximum dry density.

In general, the excavated on-site materials (existing fills) may be re-used as a source of general fill and backfill materials provided that they are free of organic materials and contain no lumps or rock fragments greater than 3 inches in largest dimension. It should be noted that the excavated fine-grain soils consisting of the soft clays and silts should not be used as a source of general fill and backfill. Where used as general fill and backfill materials, the on-site materials should be moisture-conditioned to above the optimum moisture content, placed in level lifts not exceeding 8 inches in loose thickness, and compacted to a minimum of 90 percent relative compaction.

Imported material should consist of "select granular fill," such as crushed basalt or cinder sand. Select granular fill materials should be well graded from coarse to fine with particles no larger than 3 inches in largest dimension. Select granular fill should have a laboratory CBR value of 20 or more with a maximum swell value of 1 percent or less when tested in accordance with AASHTO T-193 (or ASTM D 1883). In addition, select granular fill material should contain between 10 and 30 percent particles passing the No. 200 sieve. Imported materials should be brought to above the optimum moisture content and compacted to a minimum of 95 percent relative compaction. Surfaces should be finished to create smooth, unyielding subgrades and should be kept moist until covered by concrete slabs or pavements. Imported materials should be tested and approved by Geolabs prior to delivery to the project site for its intended use.

For backfill behind the abutment structure, the backfill material should consist of well-graded, granular fill material conforming to Type A Structure Backfill Material of Section 703.20 of the HSS. The material should be moisture-conditioned to above the optimum moisture content, placed in level loose lifts not exceeding 8 inches, and compacted to a minimum of 95 percent relative compaction. For backfill below the ground water level, free draining granular material such as AASHTO M43, No. 67 gradation, wrapped on all sides with non-woven filter fabric shall be used. This free draining granular material should be used up to a level of about 12 inches above ground water level to facilitate compaction of the backfill materials.

3.8 Underground Water Line

Based on the plans provided, we understand an 8-inch diameter water line will be installed on the upstream side of the bridge and will be connected to the existing water line system. It is anticipated that the trench for the underground water line generally will be excavated in the near-surface fills and the very soft or loose soils. We also understand that approximately 120 feet concrete jacket will be provided for the portion of water line crossing the stream. In general, we recommend providing granular bedding consisting of 6 inches of open-graded gravel (AASHTO M43 Size No. 67 gradation) under the pipes. Where soft and/or loose soils are exposed at or near the invert of the pipes or the concrete jacket, an additional 18 to 24 inches of open-graded gravel wrapped in a non-woven filter fabric (Mirafi 180N or equivalent) should be provided below the bedding layer for uniform support.

Free-draining granular materials, such as open-graded gravel (AASHTO M43 No. 67 gradation), also should be used for the initial trench backfill up to about 12 inches above the pipes or about 12 inches above the groundwater level to provide for adequate support around the pipes. It is critical that the free-draining materials be used to reduce the potential for formation of voids below the haunches of pipes and to provide adequate support around the sides of the pipes. Improper trench backfill could result in backfill settlement and pipe damage.

The upper portion of the trench backfill from the level 12 inches above the pipes (or groundwater level) to the top of the subgrade may consist of the excavated on-site soils with a maximum particle size of 6 inches (excluding the very soft or loose soils). The backfill material should be moisture-conditioned to above the optimum moisture content, placed in maximum 8-inch level loose lifts, and mechanically compacted to a minimum of 90 percent relative compaction to reduce the potential for significant future ground subsidence. Where trenches are below pavement areas, the upper 3 feet of the trench backfill below the pavement grade should be compacted to a minimum of 95 percent relative compaction.

3.9 Design Review

Drawings and specifications for the proposed construction should be forwarded to Geolabs for review and written comments prior to bid advertisement. This review is necessary to evaluate adherence of the plans and specifications to the intent of the foundation and earthwork recommendations provided herein. If this review is not made, Geolabs cannot assume responsibility for misinterpretation of the recommendations presented.

3.10 Post Design / Construction Observation Services

It is recommended to retain Geolabs for geotechnical engineering services during construction. The critical items of construction monitoring that require "Special Inspection" include observation of the micropile foundation installation and testing and subgrade proof-rolling. This is to observe compliance with the intent of the design concepts, specifications, or recommendations and to expedite suggestions for design changes that may be required in the event that subsurface conditions differ from those anticipated at the time this report was prepared.

The recommendations provided herein are contingent upon such observations. If the actual soil conditions encountered during construction are different from those assumed or considered herein, 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 the field borings and bulk samples. Variations of conditions between and beyond the field borings and bulk samples 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 in this report.

The field boring and bulk sample locations are approximate, having been estimated from the Site Plan transmitted by Austin, Tsutsumi & Associates, Inc. on June 25, 2008. Elevations of the field borings were interpolated from the contour lines shown on the same plan. The boring locations and elevations should be considered accurate only to the degree implied by the methods used.

The stratification breaks shown on the Logs of 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 herein. 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 seasonal rainfall, and other factors.

This report has been prepared for the exclusive use of KAI Hawaii, Inc. and their project consultants for specific application to the *Kawela Bridge Replacement* 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 design engineers and architect in the preparation of the bridge design for the proposed 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 soil conditions, such as perched groundwater, soft deposits, 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.

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

CLOSURE

The following plates and appendices are attached and complete this report:

-ΩΩΩΩΩΩΩΩΩ-

Respectfully submitted,

GEOLABS, INC.

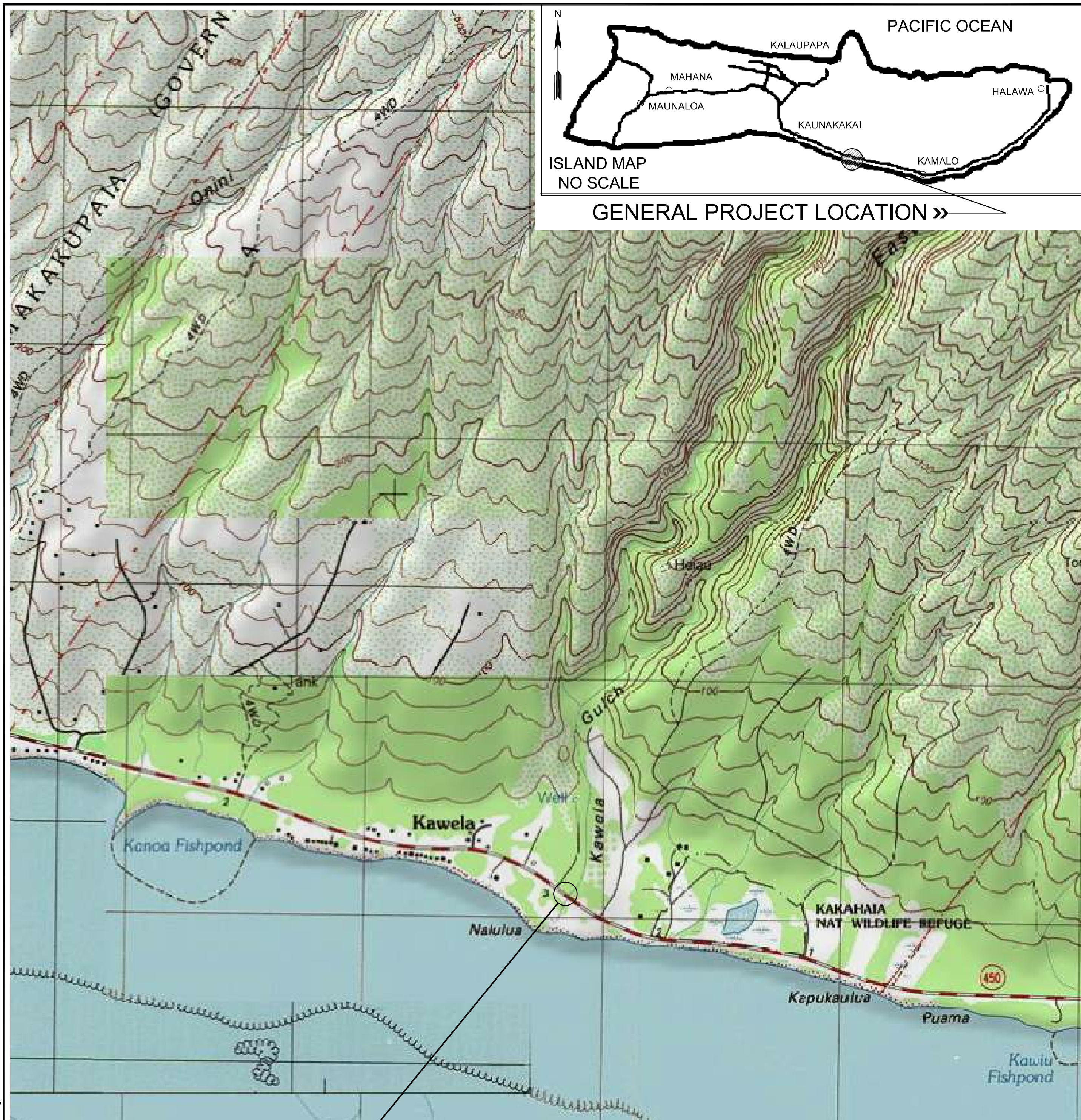
By ______________________

Robin M. Lim, P.E.
Vice President

RML:RP:as

h:\5900Series\5909-00.rp1

PLATES



PROJECT LOCATION »

PROJECT LOCATION MAP

KAMEHAMEHA V HIGHWAY
KAWELA BRIDGE REPLACEMENT
MP 5.110 TO MP 5.118
ISLAND OF MOLOKAI, HAWAII

N

.5 0 1 km

GRAPHIC SCALE

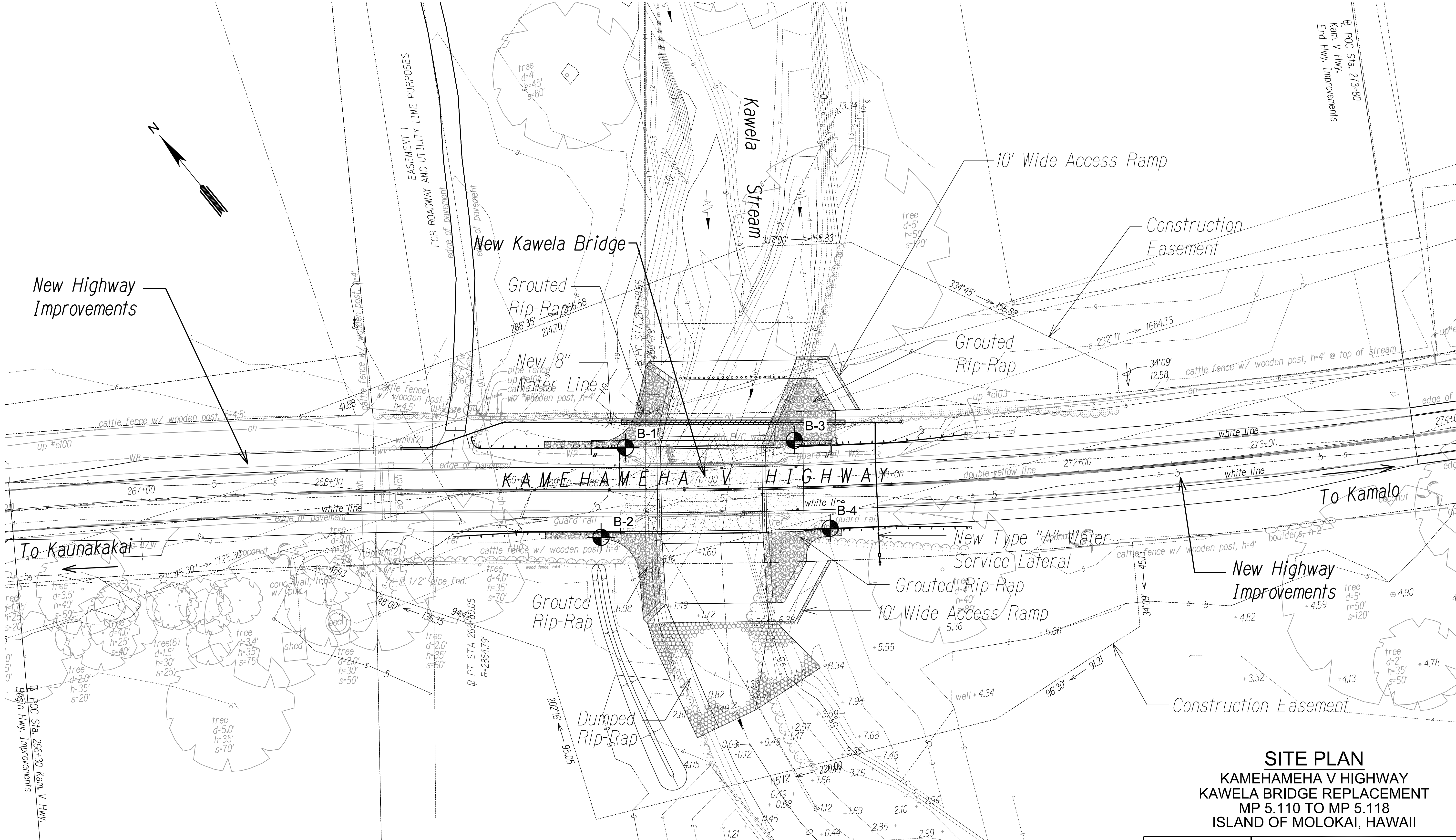
REFERENCE: MAP CREATED WITH TOPO!® ©2001 NATIONAL GEOGRAPHIC (WWW.NATIONALGEOGRAPHIC.COM/TOPO).

GEOLABS, INC.

Geotechnical Engineering

DATE	DRAWN BY	PLATE
OCTOBER 2008	HYC	
SCALE	W.O.	1
1: 24,000	5909-00(A)	

User: JLP File Created: August 26, 2008 File Last Updated: October 16, 2008 11:06:17am
File: T:\Drafting-9904\Working\5909-00(A)KawelaBridgeReplacement\5909-00(A)PLM.dwg\Molakai



SITE PLAN
KAMEHAMEHA V HIGHWAY
KAWELA BRIDGE REPLACEMENT
MP 5.110 TO MP 5.118
ISLAND OF MOLOKAI, HAWAII

LEGEND:

APPROXIMATE BORING LOCATION

REFERENCE: GENERAL PLAN TRANSMITTED BY AUSTIN, TSUTSUMI & ASSOCIATES, INC. ON JUNE 25, 2008.

0 10 20 30 40 50 100 150 FT.

GRAPHIC SCALE

GEOLABS, INC. Geotechnical Engineering		
DATE: OCTOBER 2008	DRAWN BY: JRP	PLATE
SCALE: 1" = 50'	W.O.: 5909-00(A)	2

CAD User: JJP File Last Updated: October 16, 2008 11:09:34am Plot Date: October 17, 2008 - 9:20:34am
File: T:\Drafting-9904\Working\5909-00(A)KawelaBridgeReplacement\5909-00(A)SitePlan.dwg\SitePlan

APPENDIX A

APPENDIX A

Field Exploration

We explored the subsurface conditions by drilling and sampling four borings to depths of approximately 72 to 77 feet below the existing ground surface. The borings were drilled with a truck-mounted drill rig equipped with solid stem augers and rotary coring tools at the approximate locations shown on the Site Plan, Plate 2.

Our geologist observed the field exploration operations on a near-continuous basis. Generally, he classified the materials encountered in the borings by visual and textural examination in the field. These classifications were further reviewed by visual observation and testing in the laboratory. Soils were classified in general conformance with the Unified Soil Classification System, as shown on Plate A. Graphic representations of the materials encountered are presented on the Logs of Borings, Plates A-1 through A-4.

Relatively “undisturbed” soil samples were obtained from the drilled borings in general accordance with ASTM D 3550, Ring-Lined Barrel Sampling of Soils, by driving a 3-inch OD Modified California sampler with a 140-pound hammer falling 30 inches. In addition, some samples were obtained from the drilled borings in general accordance with ASTM D 1586, Penetration Test and Split-Barrel Sampling of Soils, by driving a 2-inch OD standard penetration sampler using the same hammer and drop. The blow counts needed to drive the sampler the second and third 6 inches of an 18-inch or 24-inch drive are shown as the “Penetration Resistance” on the Logs of Borings at the appropriate sample depths.

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Geotechnical Engineering

Soil Log Legend

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)

MAJOR DIVISIONS			USCS	TYPICAL DESCRIPTIONS
COARSE-GRAINED SOILS MORE THAN 50% OF MATERIAL RETAINED ON NO. 200 SIEVE	GRAVELS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS LESS THAN 5% FINES	GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
			GP	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES MORE THAN 12% FINES	GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES
			GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
	SANDS 50% OR MORE OF COARSE FRACTION PASSING THROUGH NO. 4 SIEVE	CLEAN SANDS LESS THAN 5% FINES	SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
			SP	POORLY-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
		SANDS WITH FINES MORE THAN 12% FINES	SM	SILTY SANDS, SAND-SILT MIXTURES
			SC	CLAYEY SANDS, SAND-CLAY MIXTURES
FINE-GRAINED SOILS 50% OR MORE OF MATERIAL PASSING THROUGH NO. 200 SIEVE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50	ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT 50 OR MORE	MH	INORGANIC SILT, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY
			OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS			PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

LEGEND

(50-mm) O.D. STANDARD PENETRATION TEST

(75-mm) O.D. MODIFIED CALIFORNIA SAMPLE

SHELBY TUBE SAMPLE

GRAB SAMPLE

CORE SAMPLE

WATER LEVEL OBSERVED IN BORING

LL	LIQUID LIMIT (NP=NON-PLASTIC)
PI	PLASTICITY INDEX (NP=NON-PLASTIC)
TV	TORVANE SHEAR (kPa)
PEN	POCKET PENETROMETER (kPa)
UC	UNCONFINED COMPRESSION (kPa)
UU	UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION (kPa)

Plate

A

GEOLABS, INC. Geotechnical Engineering	KAMEHAMEHA V HIGHWAY KAWELA BRIDGE REPLACEMENT MP 5.110 TO MP 5.118 ISLAND OF MOLOKAI, HAWAII	Log of Boring B-1

Approximate Ground Surface Elevation (feet): 6 *

Laboratory: Other Tests	Laboratory: Moisture Content (%)	Laboratory: Dry Density (pcf)	Field: Core Recovery (%)	Field: RQD (%)	Field: Penetration Resistance (blows/foot)	Field: Pocket Pen. (tsf)	Depth (feet)	Sample	Graphic	USCS	Description
	13	79			37/4" Ref.					ML	Grayish brown **SANDY SILT** with some gravel and cobbles, very hard, dry (fill)
	5				22		∇ 5				grades to very stiff
	22	101			26					SW-SM	Dark grayish black **SILTY SAND (BASALTIC)** with cobbles, medium dense to dense (river deposit)
			0								
							10				
	1				2					GW	Gray **GRAVEL (BASALTIC)** with traces of sand, very loose (lagoonal deposit)
			0								
							15				
					2						
			0								
							20				
	37				2					GM	Light gray **SILTY GRAVEL** with some sand, very loose (lagoonal deposit)
			0								
							25				
	42				2						
			0								
							30				
	14				6					GW-GM	Gray **GRAVEL (CORALLINE)** with sand and little silt, loose (lagoonal deposit)
			0								
							35				
	22				6						
			0								
							40				

Date Started:	April 16, 2008	Water Level: ∇	4.7 ft. 4/16/08 1541 HRS	Plate
Date Completed:	April 16, 2008			
Logged By:	D. Gremminger	Drill Rig:	CME-55	A - 1.1
Total Depth:	72 feet	Drilling Method:	4" Auger & HQ Coring	
Work Order:	5909-00(A)	Driving Energy:	140 lb. wt., 30 in. drop	

GEOLABS, INC. Geotechnical Engineering	KAMEHAMEHA V HIGHWAY KAWELA BRIDGE REPLACEMENT MP 5.110 TO MP 5.118 ISLAND OF MOLOKAI, HAWAII	Log of Boring	**B-1**

(Continued from previous plate)

Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)	Depth (feet)	USCS	Description
								GW-GM	
	21		0		5			GM	Gray **SILTY GRAVEL (CORALLINE)** with sand, loose (lagoonal deposit)
							45		
	25		0		4				grades with traces of clay
							50		
	27		29		7			SM	Dark gray **SILTY SAND (BASALTIC)** with some gravel (coralline), loose (lagoonal deposit)
							55		Dark gray **COBBLES AND BOULDERS (BASALTIC)**, dense (alluvium)
	24		60		29/4" Ref.	4.0		ML	Dark brown **SANDY SILT** with some gravel, very dense (alluvium)
							60		
			0		10/1" Ref.				Dark gray **COBBLES AND BOULDERS (BASALTIC)**, very dense (alluvium)
							65		
			47		13/2" Ref.				
							70		
									Boring terminated at 72 feet
							75		* Elevations estimated from General Plan transmitted by Austin, Tsutsumi & Associates, Inc. on June 25, 2008.
							80		

Date Started:	April 16, 2008	Water Level: ▽	4.7 ft. 4/16/08 1541 HRS	Plate
Date Completed:	April 16, 2008			
Logged By:	D. Gremminger	Drill Rig:	CME-55	A - 1.2
Total Depth:	72 feet	Drilling Method:	4" Auger & HQ Coring	
Work Order:	5909-00(A)	Driving Energy:	140 lb. wt., 30 in. drop	

GEOLABS, INC. Geotechnical Engineering	KAMEHAMEHA V HIGHWAY KAWELA BRIDGE REPLACEMENT MP 5.110 TO MP 5.118 ISLAND OF MOLOKAI, HAWAII	Log of Boring B-2

Approximate Ground Surface Elevation (feet): 6 *

Laboratory: Other Tests	Laboratory: Moisture Content (%)	Laboratory: Dry Density (pcf)	Field: Core Recovery (%)	Field: RQD (%)	Field: Penetration Resistance (blows/foot)	Field: Pocket Pen. (tsf)	Depth (feet)	Sample	Graphic	USCS	Description
										SM	Grayish brown **SILTY SAND**, very dense, dry (fill)
	15	68			12/1" Ref.						
	26				5						grades to loose, moist
	45	67			2		5			SP	Dark gray **SAND** with some gravel, very loose (lagoonal deposit)
			0								
							10				
					5						
			0								
							15				
	35				1					ML	Gray **SILT** with sand and gravel, very soft (lagoonal deposit)
			0								
							20				
	53				2						
			0								
							25				
	35				2					GM	Gray **SILTY GRAVEL (CORALLINE)** with some sand and traces of clay, very loose (lagoonal deposit)
			0								
							30				
	25				4						
			0								
							35				
	7				8					GW-GM	grades with less silt, loose
			0								
							40				

Date Started:	April 17, 2008	Water Level: ∇	4.8 ft. 4/17/08 1544 HRS
Date Completed:	April 17, 2008		
Logged By:	D. Gremminger	Drill Rig:	CME-55
Total Depth:	72.1 feet	Drilling Method:	4" Auger & HQ Coring
Work Order:	5909-00(A)	Driving Energy:	140 lb. wt., 30 in. drop

Plate A - 2.1

BORING_LOG 5909-00.GPJ GEOLABS.GDT 10/17/08

GEOLABS, INC. Geotechnical Engineering	KAMEHAMEHA V HIGHWAY KAWELA BRIDGE REPLACEMENT MP 5.110 TO MP 5.118 ISLAND OF MOLOKAI, HAWAII	Log of Boring B-2

(Continued from previous plate)

Laboratory			Field								
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)	Depth (feet)	Sample	Graphic	USCS	Description
										GW-GM	
	15				10						
			0								
							45				
	31				11						
			0								
							50				
	35				6						
			33								
							55				Gray **COBBLES AND BOULDERS (BASALTIC)**, very dense (alluvium)
	10				50/6" Ref.						
			67								
							60				
			69		13/1" Ref.						
							65				
			45								
							70				
											Boring terminated at 72.1 feet
					15/1" Ref.						
							75				
							80				

Date Started:	April 17, 2008	Water Level: ⊽	4.8 ft. 4/17/08 1544 HRS	Plate
Date Completed:	April 17, 2008			
Logged By:	D. Gremminger	Drill Rig:	CME-55	
Total Depth:	72.1 feet	Drilling Method:	4" Auger & HQ Coring	A - 2.2
Work Order:	5909-00(A)	Driving Energy:	140 lb. wt., 30 in. drop	

GEOLABS, INC. Geotechnical Engineering	KAMEHAMEHA V HIGHWAY KAWELA BRIDGE REPLACEMENT MP 5.110 TO MP 5.118 ISLAND OF MOLOKAI, HAWAII	Log of Boring B-3

Laboratory			Field								Approximate Ground Surface Elevation (feet): 6 *
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)	Depth (feet)	Sample	Graphic	USCS	Description
										SM	Light gray **SILTY SAND** with gravel and some cobbles, very dense, dry (fill)
	23	92			31/2" Ref.						
	7				40						grades to dense
	45	72			9	1.3	5			ML	Dark gray **SILT** with fine sand, medium stiff to stiff (river deposit)
			0								
							10				
	21				2					GW	Grayish tan **GRAVEL (CORALLINE)** with sand, very loose (lagoonal deposit)
			0								
							15				
	34				2					GW-GM	grades with silt and traces of clay
			0								
							20				
	16				2					GW	grades without silt and clay
			0								
							25				
	57				2					SM	Gray **SILTY SAND** with gravel, very loose (lagoonal deposit)
			0								
							30				
	11				6						grades to loose
			0								
							35				
	24				6						
			0								
							40				

Date Started:	April 15, 2008	Water Level: ▽	4.1 ft. 4/16/08 0928 HRS	Plate
Date Completed:	April 15, 2008			
Logged By:	D. Gremminger	Drill Rig:	CME-55	
Total Depth:	72 feet	Drilling Method:	4" Auger & HQ Coring	A - 3.1
Work Order:	5909-00(A)	Driving Energy:	140 lb. wt., 30 in. drop	

GEOLABS, INC. Geotechnical Engineering	KAMEHAMEHA V HIGHWAY KAWELA BRIDGE REPLACEMENT MP 5.110 TO MP 5.118 ISLAND OF MOLOKAI, HAWAII	Log of Boring B-3

(Continued from previous plate)

Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)	USCS	Description
							SM	
	38		0		3		ML	Gray **GRAVELLY SILT** with sand, very soft (lagoonal deposit)
	24		0		6		GM	Gray **SILTY GRAVEL (CORALLINE)** with sand, loose (lagoonal deposit)
	1		77		16/6" +15/2' Ref.			Gray **COBBLES AND BOULDERS** with gravel, very dense (alluvium)
			41		12/1" Ref.			
			47					
			51		15/1" Ref.			
								Boring terminated at 72 feet

Date Started:	April 15, 2008	Water Level: ∇	4.1 ft. 4/16/08 0928 HRS
Date Completed:	April 15, 2008		
Logged By:	D. Gremminger	Drill Rig:	CME-55
Total Depth:	72 feet	Drilling Method:	4" Auger & HQ Coring
Work Order:	5909-00(A)	Driving Energy:	140 lb. wt., 30 in. drop

GEOLABS, INC. Geotechnical Engineering	KAMEHAMEHA V HIGHWAY KAWELA BRIDGE REPLACEMENT MP 5.110 TO MP 5.118 ISLAND OF MOLOKAI, HAWAII	Log of Boring **B-4**

Approximate Ground Surface Elevation (feet): 6 *

Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)	Depth (feet)	USCS	Description
							0	SM	Dark brownish gray **SILTY SAND** with gravel and cobbles, medium dense, dry (fill)
	14	85			39				
	2				46				grades to dense
	20				13		5	SP	Dark gray **SAND**, medium dense (lagoonal deposit)
			0						
							10		
					10				grades to loose
			0						
							15		
	41				2			SW-SM	grades with little silt, very loose
			0						
							20		
	6				3			GW	Grayish tan **GRAVEL (CORALLINE)** with sand, very loose (lagoonal deposit)
			0						
							25		
	1				2				
			0						
							30		
	20				5				grades to loose
			0						
							35		
					3				grades to very loose
			0						
							40		

Date Started:	April 14, 2008	Water Level: ▽	5.3 ft. 4/15/08 0929 HRS
Date Completed:	April 14, 2008		
Logged By:	D. Gremminger	Drill Rig:	CME-55
Total Depth:	77 feet	Drilling Method:	4" Auger & HQ Coring
Work Order:	5909-00(A)	Driving Energy:	140 lb. wt., 30 in. drop

Plate A - 4.1

GEOLABS, INC. Geotechnical Engineering	KAMEHAMEHA V HIGHWAY KAWELA BRIDGE REPLACEMENT MP 5.110 TO MP 5.118 ISLAND OF MOLOKAI, HAWAII	Log of Boring B-4

(Continued from previous plate)

Laboratory			Field								
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)	Depth (feet)	Sample	Graphic	USCS	Description
										GW	
	0				3						
			0								
							45				
	7				5						grades to loose
			0								
							50				
	15				7						
			0								
							55				Dark gray **GRAVEL AND COBBLES (BASALTIC)**, dense (alluvium)
	1				35						
			43								grades to very dense
							60				
			81		15/1" Ref.						
							65				
			0								
							70				
			47		25/2" Ref.						
							75				
											Boring terminated at 77 feet
							80				

Date Started:	April 14, 2008	Water Level: ▽	5.3 ft. 4/15/08 0929 HRS
Date Completed:	April 14, 2008		
Logged By:	D. Gremminger	Drill Rig:	CME-55
Total Depth:	77 feet	Drilling Method:	4" Auger & HQ Coring
Work Order:	5909-00(A)	Driving Energy:	140 lb. wt., 30 in. drop

APPENDIX B

A P P E N D I X B

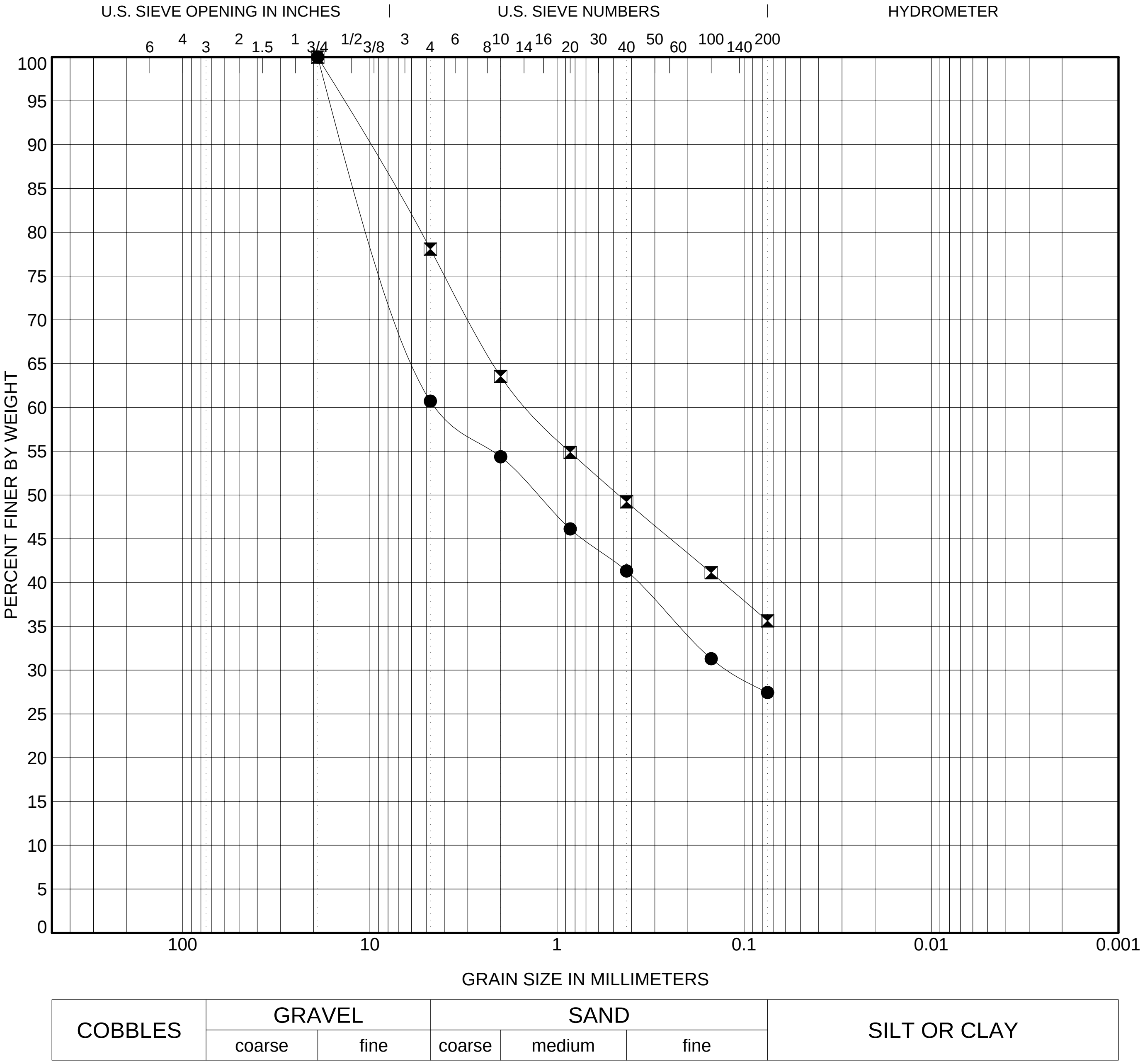
Laboratory Tests

Moisture Content (ASTM D 2216) and Unit Weight (ASTM D 7263) determinations were performed on selected soil samples as an aid in the classification and evaluation of soil properties. The test results are presented on the Logs of Borings at the appropriate sample depths.

Two Sieve Analyses tests (ASTM C 117 & C 136) were performed on selected soil samples to evaluate the gradation characteristics of the soils and to aid in soil classification. Graphic presentation of the grain size distribution is provided on Plate B-1.

Two California Bearing Ratio (CBR) tests (ASTM D 1883) were performed on bulk samples of the near-surface soils to evaluate the strength characteristics for pavement subgrade support. CBR test results are presented on Plate B-2 and B-3.

One laboratory Resistance (R) Value test (ASTM D 2844) was performed by Signet Testing Labs on a selected bulk sample of the near-surface soils to evaluate the pavement support characteristics of the soils. The test results are presented on Plate B-4.



	Sample	Depth (ft)	Description	LL	PL	PI	Cc	Cu
●	B-1	22.0-23.5	Gray silty gravel (GM) with some sand					
⧗	B-3	27.0-28.5	Gray silty sand (SM) with gravel					

	Sample	Depth (ft)	D100 (mm)	D60 (mm)	D30 (mm)	D10 (mm)	%Gravel	%Sand	%Fine
●	B-1	22.0-23.5	19	4.315	0.119		39.3	33.3	27.4
⧗	B-3	27.0-28.5	19	1.412			21.9	42.5	35.6

GEOLABS, INC.

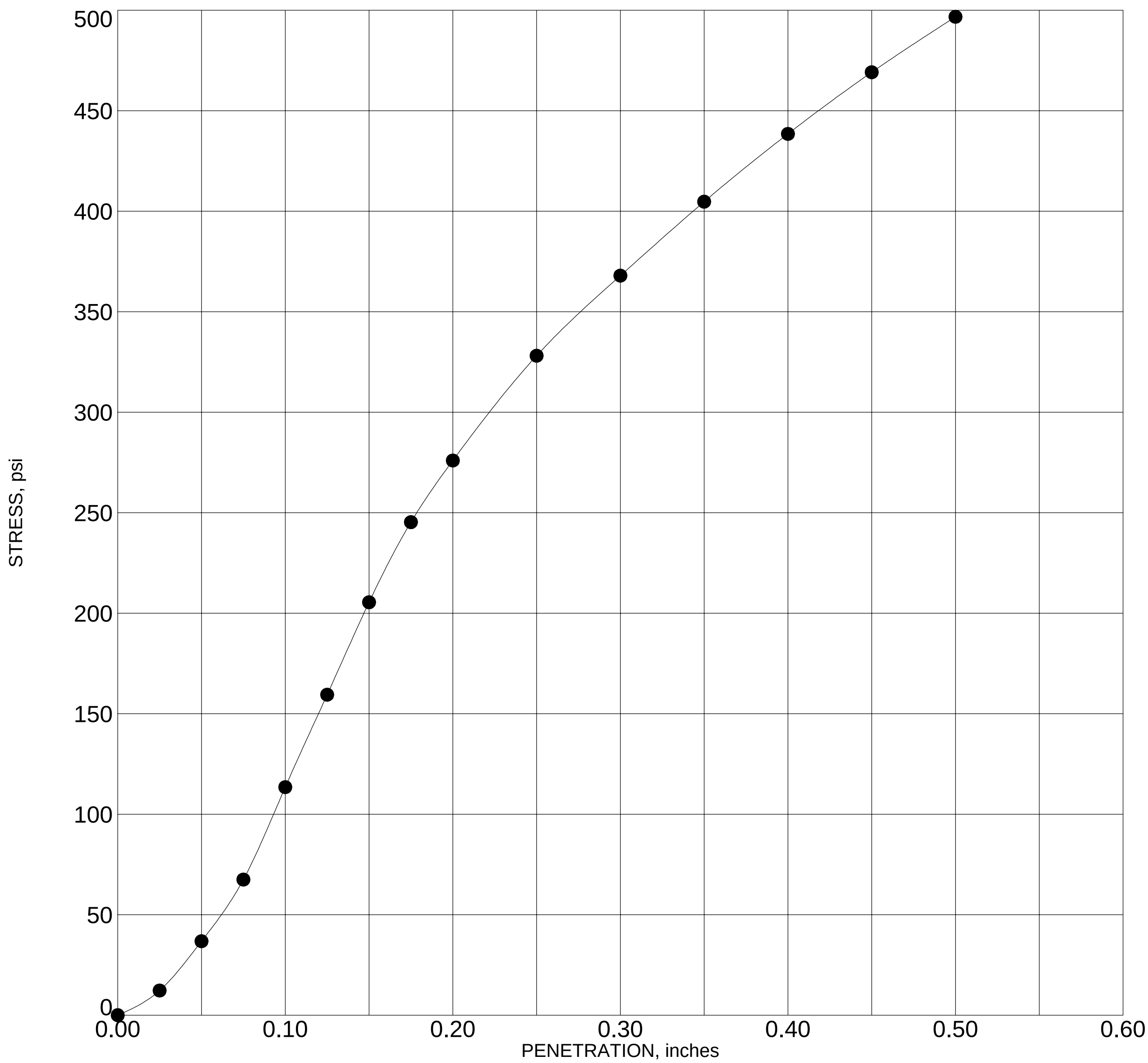
GEOTECHNICAL ENGINEERING

W.O. 5909-00(A)

GRAIN SIZE DISTRIBUTION - ASTM C 136

KAMEHAMEHA V HIGHWAY
KAWELA BRIDGE REPLACEMENT
MP 5.110 TO MP 5.118
ISLAND OF MOLOKAI, HAWAII

Plate
B - 1



Corr. CBR @ 0.1"	18.4
Swell (%)	1.13

Sample: Bulk-1
Depth: N/A
Description: Dark brownish gray sandy silt

Molding Dry Density (pcf)	101.6	Hammer Wt. (lbs)	10
Molding Moisture (%)	22.8	Hammer Drop (inches)	18
Days Soaked	5	No. of Blows	56
Aggregate	3/4 inch minus	No. of Layers	5

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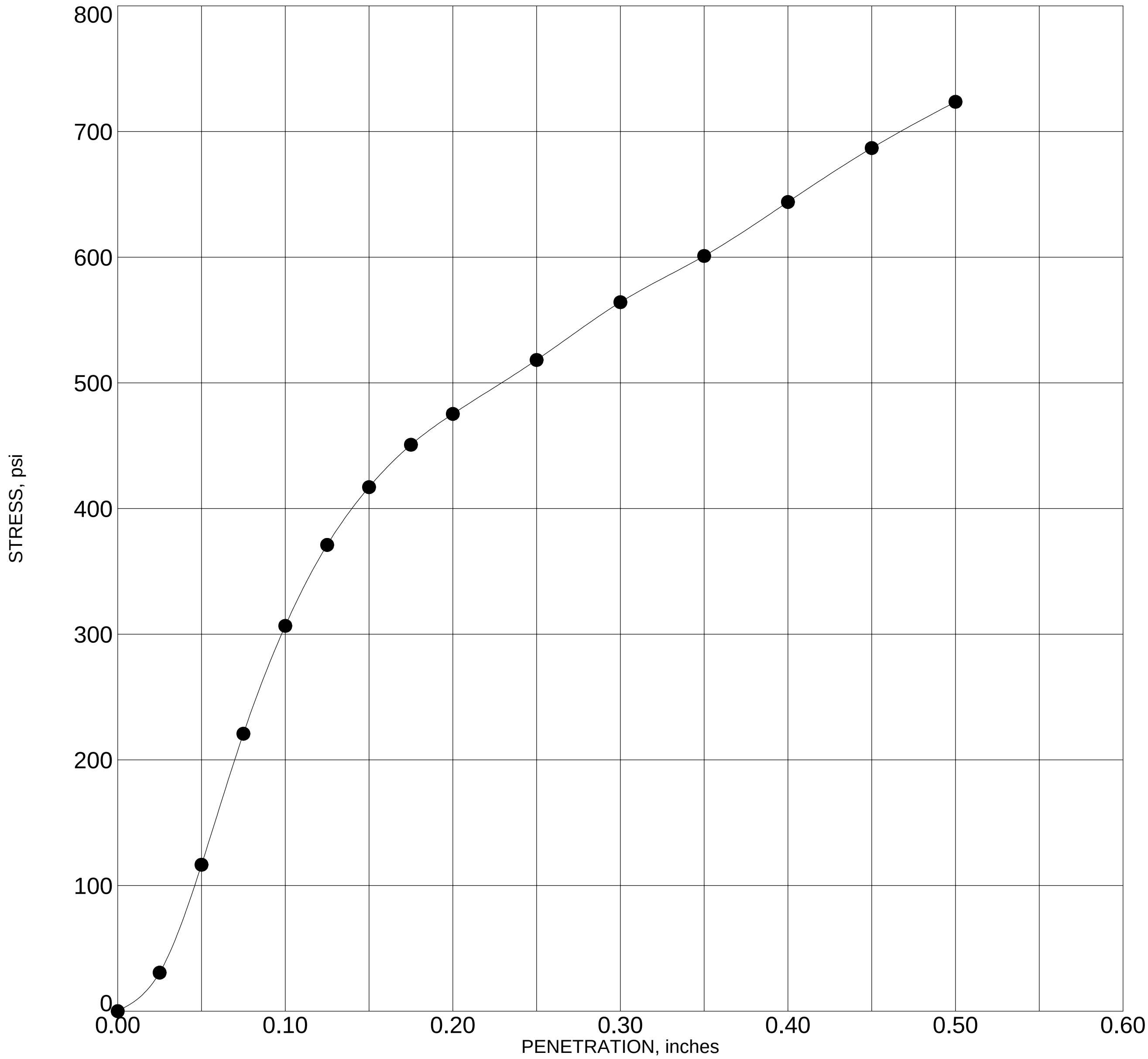
W.O. 5909-00(A)

CALIFORNIA BEARING RATIO - ASTM D 1883

KAMEHAMEHA V HIGHWAY
KAWELA BRIDGE REPLACEMENT
MP 5.110 TO MP 5.118
ISLAND OF MOLOKAI, HAWAII

Plate
B - 2



Corr. CBR @ 0.1"	34.8
Swell (%)	0.39

Sample: Bulk-2
Depth: N/A
Description: Dark brownish gray sandy silt

Molding Dry Density (pcf)	108.5	Hammer Wt. (lbs)	10
Molding Moisture (%)	19.7	Hammer Drop (inches)	18
Days Soaked	5	No. of Blows	56
Aggregate	3/4 inch minus	No. of Layers	5

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W.O. 5909-00(A)

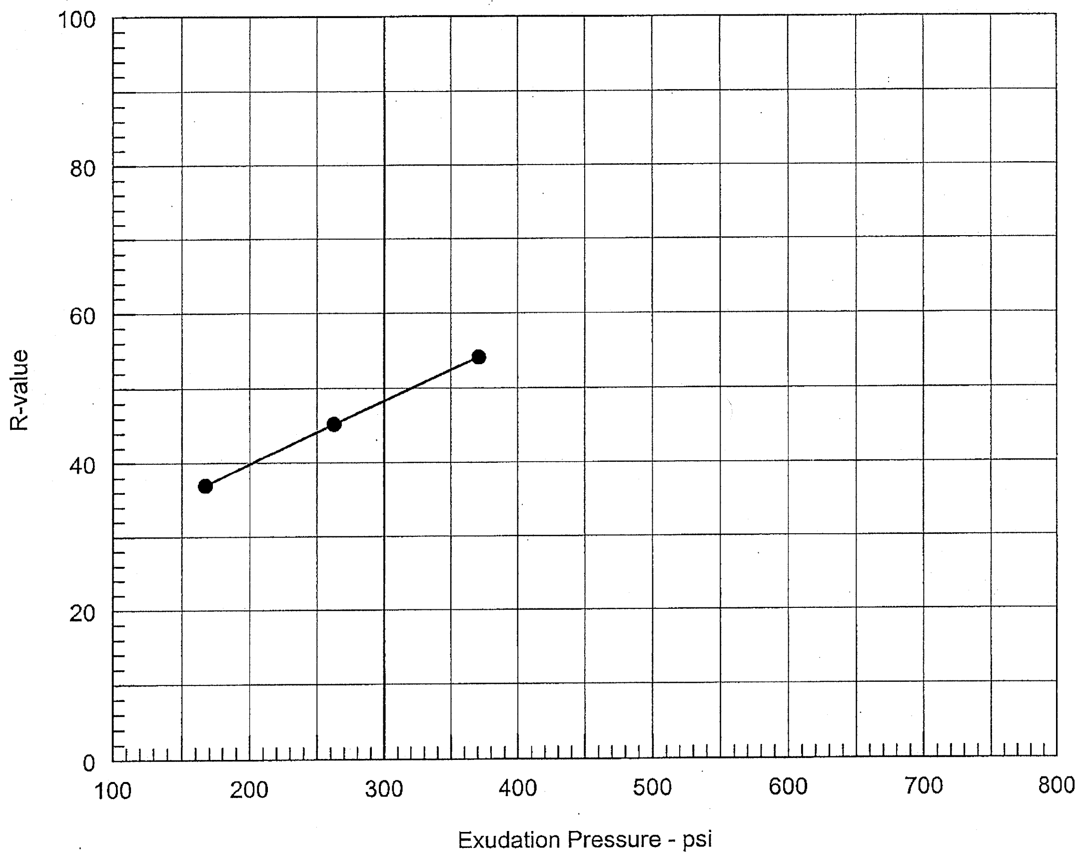
CALIFORNIA BEARING RATIO - ASTM D 1883

KAMEHAMEHA V HIGHWAY
KAWELA BRIDGE REPLACEMENT
MP 5.110 TO MP 5.118
ISLAND OF MOLOKAI, HAWAII

Plate
B - 3

G_CBR 5909-00.GPJ GEOLABS.GDT 10/17/08

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - ASTM D 2844

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psi	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	215	100.9	22.1	0.85	48	2.53	371	54	54
2	105	99.4	23.9	0.39	70	2.47	167	37	37
3	160	100.0	23.0	0.61	60	2.51	263	45	45

Test Results	Material Description
R-value at 300 psi exudation pressure = 48	Reddish brown silt, Sample-1, sample received 5/28/2008

Project No.: 0007653
Project:
Location: Kamehameha V Hwy Kawela Bridge Replacement MP5.110 to MP5.118
Sample Number: L11916
Date: 6/4/2008

R-VALUE TEST REPORT
SIGNET TESTING LABS, INC.

Tested by: DTN
Checked by: LKL
Remarks:
Sample-1
#5909-00