

GEOTECHNICAL ENGINEERING EXPLORATION
KAUMUALII HIGHWAY DRAINAGE IMPROVEMENTS
VICINITY OF OMAO STREAM BRIDGE
OMAO, KAUAI, HAWAII
W.O. 8920-10 MARCH 11, 2026

Prepared for

PAREN, INC. DBA PARK ENGINEERING



GEOLABS, INC.
Geotechnical Engineering and Drilling Services

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THIS WORK WAS PREPARED BY
ME OR UNDER MY SUPERVISION.


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EXPIRATION DATE
OF THE LICENSE



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Geotechnical Engineering and Drilling Services

March 11, 2026
W.O. 8920-10

Mr. Russel Arakaki
ParEn, Inc. dba Park Engineering
711 Kapiolani Boulevard, Suite 1500
Honolulu, HI 96813

Dear **Mr. Arakaki:**

Geolabs, Inc. is pleased to submit this report entitled "Geotechnical Engineering Exploration, Kaumualii Highway Drainage Improvements, Vicinity of Omao Stream Bridge, Omao, Kauai, Hawaii," prepared for the design of the project.

Our work was performed in general accordance with the scope of services outlined in our revised fee proposal dated July 23, 2025.

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 special 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 are any questions regarding this report, please contact our office.

Very truly yours,

GEOLABS, INC.

Gerald Y. Seki, P.E.
Vice President

GS:GB:lf

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**GEOTECHNICAL ENGINEERING EXPLORATION
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VICINITY OF OMAO STREAM BRIDGE
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SUMMARY OF FINDINGS AND RECOMMENDATIONS
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The borings drilled within the pavement limits of Kaumualii Highway encountered 9 inches of asphaltic concrete underlain by fills extending to depths of about 1.5 to 2 feet below the existing ground surface. Below the fills and near Omao Stream, the subsurface conditions generally consisted of residual soils and saprolite extending to the maximum depths explored of 31.5 to 41.5 feet below the existing ground surface. We did not encounter groundwater in the drilled borings at the time of our field exploration.

Based on our evaluation and engineering analyses, we believe that microtunneling/pipe jacking is a feasible trenchless method to install the new drain line for the drainage improvements project. We believe that the key geotechnical engineering considerations for the microtunneling/pipe jacking construction methods would be the potential for the presence of relict boulders and/or a cluster of cobbles or hard basalt ledges encountered in the residual and saprolitic soils. A boulder was possibly encountered below the drain line invert in Boring No. 102; therefore, the potential for the presence of relict boulders and/or a cluster of cobbles or hard basalt ledges does exist at the site.

For open-trench installation of the new drain line, we believe that the medium stiff silts generally anticipated along the drain line invert elevations would provide adequate support for the new pipes without the need for special foundation support. Where soft/loose soils are encountered at or near the invert elevations along the new drain line planned, we recommend providing a subgrade stabilization layer consisting of 24 inches of No. 2 Coarse Aggregate (ASTM C33, No. 4 gradation) wrapped in a non-woven filter fabric (Mirafi 180N or equivalent) below the bedding layer for uniform support.

Based on the subsurface conditions encountered, an allowable bearing pressure of up to 2,500 psf may be utilized for the design of the manhole structures bearing on the medium stiff residual soils and saprolite encountered at the project site. To provide uniform bearing support for the manhole structure, we recommend providing a 6-inch thick layer of No. 3B Fine Gravel (ASTM C33, No. 67 gradation) over the medium stiff residual soils and saprolite. 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

This report presents the results of our geotechnical engineering exploration performed for the Kaumualii Highway Drainage Improvements, Vicinity of Omao Stream Bridge project in the Omao area on the Island of Kauai, Hawaii. The project location and general site vicinity are shown on the Project Location Map, Plate 1.

This report summarizes the findings and geotechnical recommendations resulting from our field exploration, laboratory testing, and engineering analyses for the project. These findings and geotechnical recommendations are intended for the design of the drain line (DL) (trenchless and open-trench installation methods), manhole structures, corrosion potential, and pavement restoration only. The findings and recommendations presented herein are subject to the limitations noted at the end of this report.

1.1 **Project Considerations**

The project site is located along Kaumualii Highway in the Omao area on the Island of Kauai, Hawaii. We understand that the project consists of improving drainage conditions along Kaumualii Highway between Maha Road and Omao Stream Bridge.

To improve drainage conditions at the site, we understand that the installation of a new DL along the west side of Kaumualii Highway is being considered. The new DL will begin north of Maha Road and extend north to daylight at Omao Stream. We understand that the new DL may have inverted depths of about 7 feet below the existing ground surface near the vicinity of Maha Road to about 22 feet below the existing ground surface near Omao Stream.

The DL would consist of a 42-inch diameter reinforced concrete pipe. We understand that the DL between Stations 0+00 and 7+82.80 within Kaumualii Highway pavement will be installed using a trenchless method consisting of microtunneling/pipe jacking. The remaining portion of the DL located along the unpaved shoulder between Stations 7+82.80 and 11+61.88 will be installed using the open-trench method.

1.2 Purpose and Scope

The purpose of our geotechnical engineering exploration was to obtain an overview of the surface and subsurface conditions to develop an idealized soil data set to formulate geotechnical engineering recommendations for the design of the project. The work was performed in general accordance with the scope of services outlined in our revised fee proposal dated July 23, 2025. The scope of work for this exploration included the following tasks and work efforts:

1. Research and review of available subsurface information in the project vicinity and existing as-built drawings.
2. Application for the State of Hawaii, Department of Transportation roadway permit.
3. Application and coordination of underground utility toning.
4. Mobilization and demobilization of a truck-mounted drill rig currently on the Island of Kauai and operators from Oahu to the project site and back.
5. Drilling and sampling of two borings along the proposed DL alignment extending to a depth of about 31.5 feet below the existing ground surface.
6. Coordination of the field exploration and logging of the borings by our geologist.
7. Geotechnical laboratory testing of selected soil samples obtained during the field exploration as an aid in classifying the materials and evaluating their engineering properties.
8. Analyses of the field and laboratory data to formulate geotechnical recommendations for the proposed drainage improvements project.
9. Preparation of this report summarizing our work on the project and presenting our findings and recommendations for the design of the project.
10. Coordination of our overall work on the project by our project engineer.
11. Quality assurance of our work and client/design team consultation by our principal engineer.

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 are presented in Appendix B.

END OF GENERAL

SECTION 2. SITE CHARACTERIZATION

2.1 Regional Geology

The Island of Kauai is composed of a single dissected basaltic shield volcano built by the extrusion of lavas of the Waimea Canyon Volcanic Series beginning about 5 to 6 million years ago. The eruption of the Waimea Volcanic Series ended about 2½ million years ago and was followed by a long period of erosion. Following the cessation of this main volcano shield-building phase, about 1½ million years ago, there was a renewed volcanic activity with the extrusion of basaltic lavas of the post-erosional Koloa Volcanic Series and the concurrent deposition of the thick alluvial sediments of the Palikea Formation.

Rock formations of the Koloa Volcanic Series are generally characterized as thick lava flows composed of dense basalt extruded from groups of vents aligned in north-south trends at various locales. Associated with the Koloa Volcanic Series lava flows are some deposits of pyroclastic materials (volcanic ash and cinders), which usually form cinder cones surrounding the vent. Rock formations of the Koloa Volcanic Series cover most of the eastern half of the Island of Kauai, including the project site.

During the Pleistocene Epoch, many sea level changes occurred as a result of widespread glaciation in the continental areas of the world. As the great continental glaciers accumulated, the level of the ocean fell since less water was available to fill the oceanic basins. Conversely, as the glaciers receded or melted, global sea levels rose because more water was available. The landmass of Kauai remained essentially stable during these changes, and the fluctuations were eustatic in nature. These glacio-eustatic fluctuations resulted in stands of the sea, which were both higher and lower relative to the present sea level of the Island of Kauai.

The project area is generally composed of basaltic rock built by the extrusion of lavas of the Koloa Volcanic Series. The basalt rock formation observed in the Koloa area appears to represent the pahoehoe lava flow type, which spread and ponded as it approached the ocean from inland areas. Pahoehoe lavas are typically characterized by

a smoother, billowy surface and internal structure of vesicular (porous) rock with limited clinker materials.

In some places, in-situ chemical weathering of the Koloa lava flows has formed a mantle of residual and saprolitic soils. In general, saprolite is composed mainly of silty material, while residual soils are more clayey. Both residual and saprolitic soils are typical of the tropical weathering of volcanic rocks. The residual and saprolitic soils grade to basaltic rock formation with increased depth. Grading operations from the previous development brought the site to its present form.

Based on a review of available geologic information, the project site is generally underlain by a thick sequence of extremely weathered basaltic rock (saprolite) belonging to the Koloa Volcanic Series. The soil materials generally consist of in-situ basalt rock that has been deeply weathered to form silty and clayey saprolitic soils with embedded decomposed rock material. These saprolitic deposits extend to a considerable depth below the upland ground surface.

2.2 Existing Site Conditions

The project site is located along Kaumualii Highway in the Omao area on the Island of Kauai, Hawaii. The project site begins near the vicinity of Maha Road and extends to Omao Stream. The project site is shown on the Plans and Profiles, Plates 2.1 through 2.3.

Kaumualii Highway is a two-lane, undivided asphaltic concrete (AC) roadway with one lane of traffic in each direction. The shoulder areas beyond the edge of the pavement generally consist of cut grass, traffic signs, and utility posts. The elevation along Kaumualii Highway ranges from +556 to +564 feet above Mean Sea Level.

2.3 Subsurface Conditions

We explored the subsurface conditions at the project site by drilling and sampling two borings, designated as Boring Nos. 101 and 102, extending to a depth of about 31.5 feet below the existing ground surface. In addition, two bulk samples of the near-surface soils, designated as Bulk-101 and Bulk-102, were collected. The boring and bulk sample locations are presented on our Plans and Profiles, Plates 2.1 and 2.2.

The boring information from our report entitled “Geotechnical Engineering Exploration, Kaumualii Highway Drainage Improvements, Maha Road to Omao Stream, Island of Kauai, Hawaii,” dated July 9, 2025, was also used to develop a generalized subsurface profile along the DL alignment.

The borings drilled within the pavement limits of Kaumualii Highway encountered 9 inches of AC underlain by fills extending to depths of about 1.5 to 2 feet below the existing ground surface. The fills consisted of very stiff to hard clayey silt.

Below the fills and near Omao Stream, the subsurface conditions generally consisted of residual soils and saprolite extending to the maximum depths explored of 31.5 to 41.5 feet below the existing ground surface. The residual soils consisted of medium stiff to very stiff clayey silt with varying amounts of gravel and sand. The saprolitic soils were composed of medium stiff to very stiff sandy/clayey silt and medium dense silty sand. A soft zone of saprolitic soil was encountered in Boring No. 101 between 6 and 15 feet below the existing ground surface, which corresponds to about Elevations +553 and +544 feet MSL.

We did not encounter groundwater in the borings at the time of our field exploration. However, it should be noted that groundwater levels are subject to change due to rainfall, time of year, seasonal precipitation, surface water runoff, and other factors.

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 are presented in Appendix B.

2.4 Geologic Units

Considering the extent of the project site, the various geologic deposits encountered along the DL alignment are relatively uniform and may be grouped into three generalized types of geologic materials. The generalized types of geologic materials encountered at the project site include the following:

1. Fills
2. Residual Soils
3. Saprolitic Soils

Further discussions on the general characteristics of the geologic units encountered in the borings are presented in the following subsections.

2.4.1 Fills

Although fills were not encountered in the borings, fills will likely be encountered at the site. The fills may consist of variable soils, which were placed by man during the development of the area. The fills would generally consist of silts and clays that likely may have derived from the excavation of the nearby areas during the construction of Kaumualii Highway. The fill materials may be encountered as both unconsolidated and semi-consolidated deposits, having variable density resulting from man-induced compaction of the fill materials.

2.4.2 Residual Soils

Residual soils are completely weathered volcanic rocks that have lost all visible rock texture characteristics. The residual soils at the project site are typically reddish brown in color. The residual soils may be mottled with various colors. Residual soils encountered at the site are comprised mainly of clayey and silty materials with medium stiff to stiff consistency. Typically, residual soils occur overlying saprolitic soils or weathered basalt formation. It should be noted that relict boulders of hard rock may be encountered in residual soils. In addition, there is potential for encountering hard basalt rock ledges in residual soils because the volcanic rock may not have weathered uniformly throughout the rock formation. Some relict boulders of hard rock may occasionally be encountered in residual soils.

2.4.3 Saprolitic Soils

Extremely weathered volcanic rock is referred to as saprolitic soils (or saprolites). Saprolitic soils represent rock that has been reduced by weathering to soil-like components with weathered rock fragments. The soil material retains the remnant rock texture, such as layering, vesicularity, and some fractured patterns. Saprolitic soils are typical of the tropical weathering of volcanic rocks and are composed mainly of silty and sandy materials. The coloring scheme of saprolitic soils depends on the type of parent volcanic rock from which the soils are derived and the degree of chemical weathering that the material has experienced. The consistency of the

saprolitic soils encountered at the site range from soft to very stiff. Because the parent material does not weather uniformly throughout the formation, relict boulders of hard rock and/or hard rock ledges occasionally may be encountered within saprolitic soils.

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 are presented in Appendix B.

END OF SITE CHARACTERIZATION

SECTION 3. DISCUSSION AND RECOMMENDATIONS

Based on the subsurface conditions encountered at the site, we believe that microtunneling/pipe jacking is a feasible trenchless method to install the new DL for the drainage improvements project. We believe that the key geotechnical engineering considerations for the microtunneling/pipe jacking construction methods would be the potential for the presence of relict boulders and/or a cluster of cobbles or hard basalt ledges encountered in the residual and saprolitic soils.

For open-trench installation of the new DL, we believe that the medium stiff silts generally anticipated along the DL invert elevations would provide adequate support for the new pipes without the need for special foundation support. Where soft/loose soils are encountered at or near the invert elevations along the new DL planned, we recommend providing a subgrade stabilization layer consisting of 24 inches of No. 2 Coarse Aggregate (ASTM C33, No. 4 gradation) wrapped in a non-woven filter fabric (Mirafi 180N or equivalent) below the bedding layer for uniform support.

Based on the subsurface conditions encountered, an allowable bearing pressure of up to 2,500 psf may be utilized for the design of the manhole structures bearing on the medium stiff residual soils and saprolite encountered at the project site. To provide uniform bearing support for the manhole structure, we recommend providing a 6-inch thick layer of No. 3B Fine Gravel (ASTM C33, No. 67 gradation) over the medium stiff residual soils and saprolite. Detailed discussions of these items and our geotechnical recommendations for the design of the project are further discussed in the following sections.

3.1 Trenchless Installation

We understand that it is desired to construct the new DL along Kaumualii Highway using a trenchless method. Based on the current size of the proposed DL (nominal 42-inch diameter) and the line and grade requirements, we believe microtunneling/pipe jacking may be used to install the DL between Stations 0+00 and 7+82.80.

The technique involves the use of powerful hydraulic jacks that push or jack successive pipe sections through the ground behind a remotely controlled excavating unit or cutting disk head at the same time as the excavation is taking place within the excavating unit. The guidance system for microtunneling/pipe jacking consists of a passive pipe laser in the jacking pit. The laser projects a point on the cutter face for the operator to monitor and make steering corrections. Since the pipe will be installed at the same time the excavation is taking place, only one step is required for microtunneling/pipe jacking. Drive lengths of up to 1,000 feet can be achieved with microtunneling/pipe jacking.

The thrust block at the jacking pit provides the reaction for the main jacks when pushing the pipe forward. The allowable passive pressure resistance of the soil, which is a function of the strength of the soil, will control the thrust resistance that can be developed. It seems reasonable to allow the thrust block to develop the full passive pressure resistance because this is a temporary construction loading condition. However, it must be recognized that relatively large strains will be required in order to develop the maximum passive pressure resistance. At the jacking loads that impose stresses approaching the maximum passive pressure resistance, there may be large movements of the thrust block in response to the strains required to develop full passive pressure resistance.

In addition, large movements of the thrust block are not desirable for pipe jacking because the movements are not uniform, and they become difficult to maintain the thrust block perpendicular to the pipe alignment. If the thrust block does not remain square, the force applied to the pipe may not be uniform, resulting in eccentric loading that could damage the pipe.

Movements of the thrust block can also be controlled by limiting the allowable passive pressure resistance to about one-half of the maximum passive pressure resistance. This corresponds to a much lower level of strain and smaller soil movements (strain compatible design). If the project requires higher jacking forces that can be safely transferred to the soil mass behind the thrust block, then the soil mass should be improved or stabilized.

An assessment of the ground conditions likely to be encountered along the tunneling excavations is important for evaluating the face stability of the excavations and to provide information on the selection of appropriate equipment and methods for the tunnel and pit construction. The ground conditions should be assessed to reduce the potential for loss of ground, surface settlement, and damage to existing utilities, buildings, roadways, and other improvements as a result of the planned tunneling operations.

3.1.1 Anticipated Tunneling Conditions

For the design pertaining to tunneling, special attention should be given to the characteristics of the soils that are likely to be encountered. Some of the important factors relating to the subsurface soil conditions that may significantly influence construction costs include face or heading stability, settlement potential and magnitudes, friction along the pipeline, allowable passive pressures for providing thrust block resistance, groundwater control, muck characteristics, and/or special disposal requirements.

Based on the subsurface conditions encountered in the borings, it appears that the new DL along Kaumualii Highway will likely encounter medium stiff sandy/clayey silt and medium dense silty sand, as shown on the Plans and Profiles, Plates 2.1 through 2.3. Groundwater was not encountered in the borings at the time of our field exploration. Based on the anticipated subsurface conditions, the contractor should be able to maintain the required lines and grades of the new DL when installed using the microtunneling/pipe jacking method at this location.

3.1.2 Potential Tunneling Difficulties

An assessment of the ground conditions likely to be encountered along the microtunneling/pipe jacking excavations is important for evaluating the face stability of the excavations and to provide information on the selection of appropriate equipment and methods for the tunnel and pit construction. The ground conditions should be assessed to reduce the potential for loss of ground, surface settlement, damage to existing utilities, buildings, roadways, and other improvements as a result of the planned tunneling operations. Based on our review of the subsurface conditions encountered in the proximity of the highway, we

believe that the key geotechnical engineering considerations for the microtunneling/pipe jacking construction methods would be the potential for the presence of relict boulders and/or a cluster of cobbles or hard basalt ledges encountered in the residual and saprolitic soils.

As mentioned previously, this is due to non-uniform weathering of the parent rock material. A boulder was possibly encountered below the DL invert in Boring No. 102; therefore, the potential for the presence of relict boulders and/or a cluster of cobbles or hard basalt ledges does exist. Ground that contains cobble clusters, boulders, or hard basalt ledges can be problematic for microtunneling/pipe jacking equipment. A boulder, cluster of cobbles, or hard basalt ledges encountered in the heading of the tunnel boring machine may severely hinder or even halt the progress of the excavation operations. Therefore, the contractor should develop a contingency plan if hard zones are encountered during the tunneling operations.

3.2 Open-Trench Installation

We anticipate that the remaining DL alignment between Stations 7+82.80 and 11+61.88, and the manhole/inlet structures will be installed by the open-trench method. Detailed discussions and recommendations pertaining to the open-trench installation are discussed in the following subsections.

3.2.1 Earth Pressure Loads on Pipes

Loads on buried pipes are influenced by the width of the trench, the size of the pipes, the unit weight of backfill material, and the friction resistance between the backfill material and the trench walls. To calculate the vertical loads on the buried sewer pipes, we recommend using an average unit weight of 130 pounds per cubic foot (pcf) for the compacted backfill material and a coefficient of friction of 0.25 may be used to calculate the frictional resistance between the compacted backfill and the sidewalls of the excavated trench. Earth forces upon the pipe generally increase rapidly with the width of the trench. Therefore, the width of the trench should be kept to a minimum.

It should be noted that the DL alignment is relatively close to the highway; therefore, the trench excavations may need to be shored using trench shields, trench boxes, or other similar shoring methods. Therefore, the width of the trench excavations will be wider than normal and should be taken into consideration when designating the type and class of pipes. In addition, traffic loads on the buried pipes should also be considered.

3.2.2 Pipe Bedding

The stress distribution against the bottom of a pipe has a great effect upon the load supporting capacity of the pipe. Therefore, pipe bedding is an important design consideration. In general, granular bedding consisting of 6 inches of No. 3B Fine gravel (ASTM C 33, No. 67 gradation) is recommended under the DL pipes. The granular bedding should extend beyond the sides of the pipe a minimum width of one-fourth the outside diameter of the pipe. The Geolabs representative will confirm whether firm materials are exposed at the bottom of the trench or if a stabilization layer should be provided as discussed below.

Should soft/loose soils be encountered at or near the bottom of the bedding layer, the soft/loose soils should be over-excavated to expose firm materials or a maximum of 24 inches below the trench bottom. The over-excavation should be backfilled with open-graded gravel (ASTM C33, No. 4 gradation) wrapped in a non-woven filter fabric (Mirafi 180N or equivalent) and placed below the bedding layer for more uniform support.

3.2.3 Backfill

In general, trench backfill should be performed in accordance with the Hawaii Standard Specifications for Road and Bridge Construction, 2005 (HSS). Free-draining granular materials, such as No. 3B Fine gravel (ASTM C33, No. 67 gradation), should be used for the trench backfill up to about 12 inches above the pipes to provide adequate support around the pipes and to reduce compaction of the backfill, thus reducing the possibility of damaging the pipes.

The upper portion of the trench backfill from the level 12 inches above the pipes to the top of the subgrade or finished grade may consist of on-site soils with particles less than 6 inches in maximum dimension or select fill material. If required, imported select granular fill should consist of crushed coral or basaltic gravel. Imported 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 imported fill material should have a laboratory California Bearing Ratio (CBR) value of 20 or more and should have a maximum swell of 1 percent or less when tested in accordance with ASTM D1883. Imported materials should be tested for conformance with these recommendations prior to delivery to the project site for the intended use.

The backfill should be moisture-conditioned to above the optimum moisture content, placed in maximum 8-inch horizontal loose lifts, and mechanically compacted to a minimum of 90 percent relative compaction to reduce the potential for appreciable future ground subsidence.

Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same soil established in accordance with ASTM D1557. Optimum moisture is the water content (percentage by dry weight) corresponding to the maximum dry density. The upper 3 feet of the trench backfill below the pavement grade should be compacted to a minimum of 95 percent relative compaction. In addition, compaction by water ponding or jetting should not be allowed for this project.

As mentioned previously, cobbles and boulders may be encountered in localized areas along the new DL alignment. Therefore, boulder-sized materials may be generated during trench excavation. It should be noted that boulder-size materials generated during the trench excavations should not be used for trench backfill.

3.2.4 Pipe Settlement

Primary settlement of the new pipe installation is generally caused by the difference in the unit weight of the excavated original earth and the compacted

backfill material placed over the pipeline. The net increase in loading may cause settlement of the underlying subsoils below the trench invert. Based on our exploration and analyses, primary settlements on the order of about less than 1 inch may be anticipated for the new DL alignment.

3.3 Drain Manhole and Inlet Structures

Based on the information provided, we understand that drain manhole and inlet structures are planned along the new DL alignment. Based on the subsurface conditions encountered, the following general guidelines may be used for the design of the manhole and inlet structures at the project site.

3.3.1 Drain Manhole and Inlet Structure Foundations

Based on the subsurface conditions encountered, an allowable bearing pressure of up to 2,500 pounds per square foot (psf) may be utilized for the design of the manhole and inlet structures bearing on the medium stiff clayey silts and saprolite encountered at the project site. To provide uniform bearing support for the manhole structure, we recommend providing a 6-inch thick bedding layer consisting of No. 3B Fine Gravel (ASTM C33, No. 67 gradation) over the medium stiff soils.

The manhole and inlet subgrades should be recompacted to a firm and unyielding surface prior to the placement of the bedding layer. Soft and/or loose materials encountered at the manhole and inlet subgrades should be over-excavated to expose the underlying firm materials. The over excavation should be backfilled with fill materials compacted to a minimum of 90 percent relative compaction.

Based on our engineering analyses, we believe that the net increase in bearing pressure on the underlying soil deposits below the manhole and inlet structures would be minimal. Settlements on the order of less than 1 inch are anticipated for the manhole and inlet structures.

A Geolabs representative should observe foundation excavations prior to placement of reinforcing steel and concrete to confirm the foundation bearing conditions and the required over-excavation depths, if applicable.

3.3.2 Lateral Earth Pressures

The lateral earth pressures acting on the proposed underground drain manhole and inlet structures will depend on the type of backfill used, the extent of backfill, and the compactive effort on the backfill material around the structures. The recommended lateral earth pressures for design of manhole and inlet 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		
<u>Subsoil Conditions</u>	<u>At-Rest</u> (pcf)	<u>Passive</u> (pcf)
Above Groundwater	60	350

The values provided above assume that on-site soils or aggregate subbase will be used to backfill around the manhole structures. It is assumed that the backfill around the manhole structures will be compacted to between 90 and 95 percent relative compaction per ASTM D1557. Over-compaction of the backfill around the manhole and inlet structures should be avoided.

Surcharge stresses due to areal surcharges, traffic loads, line loads, and point loads within a horizontal distance equal to the depth of the structure should be considered in the design. For uniform surcharge stresses imposed on the loaded side of the manhole, a rectangular distribution with uniform pressure equal to 50 percent of the vertical surcharge pressure acting over the entire depth of the structure may be used in design. Additional analyses during design may be needed to evaluate the surcharge effects of point loads and line loads.

Lateral loads acting on the structures may be resisted by friction developed between the bottom of the foundation and the supporting subgrade soils and passive earth pressure developed against the embedded near-vertical faces of the foundation system. A coefficient of friction of 0.35 may be used between the base of the structure and the granular bedding material to resist lateral loads. Based on our field

exploration data and laboratory test results, the recommended passive earth pressures shown in the above table may be used in the design.

3.3.3 Backfilling of Manhole and Inlet Excavations

The manhole and inlet excavations will need to be properly backfilled. The excavated on-site soils that are free of vegetation, deleterious materials, and clay lumps and rock fragments greater than 3 inches in maximum dimension or aggregate subbase may be used as backfill around the manhole and inlet structures up to the finish grades.

Manhole backfills 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 at least 90 percent relative compaction.

Where manhole and inlet structures are located below pavement areas, the upper 3 feet of the manhole and inlet backfill below the pavement grade should be compacted to at least 95 percent relative compaction. Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density as determined by ASTM D1557. Optimum moisture is the water content (percentage by dry weight) corresponding to the maximum dry density.

Should the manhole excavation to be backfilled be too small for a compactor, the excavation should be backfilled with Controlled Low Strength Material (CLSM).

3.4 Retaining Structures

The concrete outlet structure at Omao Stream should be designed as a retaining structure. Based on the anticipated subsurface conditions, the following general guidelines may be used for the design of the retaining structure planned at the site.

3.4.1 Retaining Structure Foundations

An allowable bearing pressure of up to 2,500 psf may be used for the design of the retaining structure foundation bearing on the recompacted on-site soils. This bearing value is for dead-plus-live loads and may be increased by one-third ($\frac{1}{3}$) for transient loads such as those caused by wind or seismic forces.

In general, the footings should have a minimum width of 18 inches, and the bottom should be embedded a minimum of 24 inches below the lowest adjacent grade. 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.

If foundations are located next to utility trenches or easements, the footing should be embedded below a 45-degree imaginary plane extending upward from the bottom edge of the utility trench, or the footing should be extended 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.

Lateral loads acting on the structures may be resisted by friction developed between the bottom of the foundation and the bearing soil and by passive earth pressure acting against the near-vertical faces of the foundation system. A coefficient of friction of 0.30 may be used for footings bearing on the recompacted on-site soils. Resistance due to passive earth pressure may be estimated using an equivalent fluid pressure of 300 pcf. This assumes that the soils around the footings are well compacted (minimum of 90 percent relative compaction). Unless covered by pavements or slabs, the passive pressure resistance in the upper 12 inches of soil should be neglected.

It is important to compact the bottom of foundation excavations to a firm and unyielding condition prior to placement of reinforcing steel and concrete. A Geolabs representative should observe foundation excavations prior to placement of reinforcing steel and concrete to confirm the foundation bearing conditions and the required over-excavation depths, if applicable.

3.4.2 Static Lateral Earth Pressures

Retaining structures should be designed to resist 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			
<u>Backfill Condition</u>	<u>Earth Pressure Component</u>	<u>Active</u> (pcf)	<u>At-Rest</u> (pcf)
Level Backfill	Horizontal	40	60
	Vertical	None	None
2H:1V	Horizontal	58	76
	Vertical	29	38

Type A Structure Backfill Material conforming to Section 703.20 of the Hawaii Standard Specifications for Road and Bridge Construction, 2005 (HSS) should be used to backfill behind the retaining structures. Backfill behind retaining structures should be compacted to between 90 and 95 percent relative compaction. Over-compaction of the retaining structure backfill should be avoided. These lateral earth pressures do not include hydrostatic pressures that might be caused by groundwater trapped behind the walls.

In general, an active condition may be used for gravity 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 a uniform pressure equal to 33 percent of the vertical surcharge

pressure acting over the entire height of the wall, which is free to deflect (cantilever), may be used in the design. For walls that are restrained, a rectangular distribution equal to 50 percent of the vertical surcharge pressure acting over the entire height of the wall may be used for the design. Additional analyses during design may be needed to evaluate the surcharge effects of point loads and line loads.

3.4.3 Drainage

For head walls, a typical drainage system may consist of a sack of permeable material placed at each weep hole location. The sack of permeable material should consist of about 1 to 2 cubic feet of No. 3B Fine gravel (ASTM C33, No. 67 gradation) material wrapped with non-woven filter fabric (Mirafi 180N or equivalent). The weep holes should consist of a minimum 4-inch diameter Polyvinyl Chloride (PVC) pipe spaced no more than 6 feet apart.

Backfill placed immediately behind permeable drainage materials should consist of select granular fill material with a maximum particle size of 3 inches. Unless covered by concrete slabs, the upper 12 inches of backfill should consist of relatively impervious material to reduce the potential for significant water infiltration behind the walls.

3.5 Excavation

Based on the planned DL inverts, we envision that temporary shoring of the excavations will be required for the utility line installation.

3.5.1 Excavation Method

In general, the contractor should determine the method and equipment to be used for excavation, subject to practical limits and safety considerations. Based on our field exploration and the available information, we envision conventional excavation techniques using a backhoe excavator may be used for the DL excavations. However, it is anticipated that hoe-ramming and/or chipping will be required if relict boulders and/or a cluster of cobbles or hard basalt ledges are encountered during the DL and manhole installation. The excavated soils should be stockpiled no closer than a horizontal distance equal to the depth of the

excavation measured from the outside edge of the excavation in order to reduce the potential for appreciable ground movement.

3.5.2 Excavation Support

Based on the generally stiff subsoil conditions encountered and the anticipated depths of excavation, we believe trench shields/boxes may be considered for temporary shoring of the new DL planned for the project. For deeper excavations, we recommend considering soldier piles and lagging or jet grout column walls with horizontal bracing for excavation support in order to reduce the potential for appreciable adjacent ground movement.

The excavation support and shoring system used must comply with all applicable safety requirements. The contractor should retain a qualified geotechnical engineer to design and evaluate the shoring system used. The contractor should be made solely responsible for the adequacy and safety of the shoring installation. The contractors' representative, who should be required to be continuously present on site during excavation and construction work, will have the best opportunity to promptly observe changing conditions during construction, such as unforeseen subsurface soil conditions, groundwater table, inappropriate construction sequence or techniques, etc., which may adversely affect shoring stability.

Even if good construction procedures are followed, some movement of the shoring system and the adjacent ground may still occur due to changes in earth stresses during excavation. Due to the complexity of the stress changes, it is difficult to accurately estimate the magnitude of ground movement. The magnitude of ground movement also depends greatly on workmanship, such as how quickly and tightly the shoring supports are installed, the subsoil conditions, the size of the excavation, and the rate of excavation. Therefore, it is important to realize that the shoring should be installed properly and as early as practical, and if necessary, the adjacent ground should be monitored continuously for cracks, dips and/or other indications of movements.

3.6 Corrosion Potential

Laboratory corrosion tests, including pH, minimum resistivity, chloride content, and sulfate content, were performed on a selected sample obtained from our field exploration to evaluate the corrosivity of the near-surface soils at the project site. The test results are summarized and presented in Appendix B, Plate B-9. In addition, laboratory corrosion data was also used in our previous report prepared for the project.

Based on the updated version of the Board of Water Supply, City and County of Honolulu Water System External Corrosion Control Standards dated July 28, 2021, there are two soil corrosivity categories. Category A (Moderately to Severely Corrosive) applies to soil meeting any one of the following conditions. Corrosion Category B (Negligibly to Mildly Corrosive) is applicable when none of the following conditions are met.

- Resistivity $\leq 5,000$ ohm-cm
- pH ≤ 6.5
- Water-soluble chloride concentration ≥ 500 ppm
- Water-soluble sulfate concentration $\geq 1,000$ ppm
- Within a fluctuating saltwater table

Resistivity is generally recognized as one of the most significant soil characteristics regarding the corrosivity of the soil to buried metallic objects. In general, the lower the resistivity, the greater the potential for corrosion of the buried metallic structure. Conversely, the higher the resistivity, the less likely the soil will contribute to the corrosion of metallic objects. Results of the laboratory testing indicate that the on-site soils have minimum resistivity values of about 1,200 and 7,500 ohm-cm with pH values of about 5.1 and 5.4.

In addition, chloride content and sulfate content testing were performed by CERCO Analytical to evaluate the corrosivity of the on-site soils encountered. Based on the chloride and sulfate content tests performed on the on-site soils, the test values showed 16 and 26 parts per million (ppm) for chloride content and 0 and 47 ppm for sulfate content. The test results indicate that the soils at the site may be considered in Corrosion Category A (Moderately to Severely Corrosive). It may be appropriate to consult with a professional corrosion engineer to review the test results and provide detailed recommendations for corrosion protection.

3.7 Pavement Restoration

It is anticipated that AC pavement will be used to repave the top of the trenches along the DL alignment crossing private driveways. We have assumed that the traffic on these driveways will be low volume consisting of passenger vehicles. In addition, we have assumed that the pavement subgrade will consist of the excavated on-site soils or select granular fill. In general, the new pavements for the driveways along the DL alignment should match the existing pavement section or the following flexible section may be used, whichever is greater:

Flexible Pavements (Private Driveways)

2.0-Inch Asphaltic Concrete

6.0-Inch Aggregate Base Course (95 Percent Relative Compaction)

8.0-Inch Minimum Total Pavement Thickness Over Compacted Subgrade

The above section is based on the assumptions presented above and that the subgrade soils can be dried to near optimum moisture content and be compacted to a minimum of 95 percent relative compaction.

Aggregate base course required for the project should consist of crushed basaltic aggregates and should conform to Subsection 703.06 of the State of Hawaii, Standard Specifications for Road and Bridge Construction (2005). Aggregate base course should be moisture-conditioned to above the optimum moisture content and compacted to no less than 95 percent relative compaction. California Bearing Ratio and field density tests should be performed on the actual subgrade soils encountered during construction to confirm the adequacy of the above section.

3.8 Design Review

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

3.9 Services During Construction

We recommend retaining Geolabs to provide geotechnical services during construction of the proposed project. The critical items of construction monitoring that require "Special Inspection" include the following:

- Review of microtunneling/pipe jacking submittals
- Observation of drain line installation using microtunneling/pipe jacking
- Observation of backfill placement and compaction for the drain line installed using the open-trench method

A Geolabs representative should observe other aspects of the earthwork construction 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 in this report 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 our test borings and bulk samples. Variations of the subsurface conditions beyond the test boring and bulk sample locations may occur and the nature and extent of these variations may not become evident until construction is underway. If variations then appear evident, it will be necessary to re-evaluate the recommendations presented herein.

The field boring locations are approximate, having been estimated by hand-held GPS device. The boring elevations were estimated from contour lines shown on the Plans and Profiles prepared by ParEn, Inc. dba Park Engineering dated September 2025. The physical locations and elevations of the borings should be considered accurate only to the degree implied by the methods used.

The stratification breaks represented on the Logs of Borings show the approximate boundaries between soil 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. The data has been reviewed, and interpretation made in the formulation of this report. However, it must be noted that fluctuation may occur due to variations in seasonal rainfall and other factors.

This report has been prepared for the exclusive use of ParEn, Inc. dba Park Engineering and their consultants for specific application to the Kaumualii Highway Drainage Improvements, Vicinity of Omao Stream Bridge 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 in the design of the proposed project. Therefore, this report may not contain sufficient data or the proper information to serve as a basis for detailed construction cost estimates.

The owner/client should be aware that unanticipated soil conditions are commonly encountered. Unforeseen subsurface conditions, such as perched groundwater, soft deposits, and 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 project 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:

Project Location Map..... Plate 1
Plans and Profiles Plates 2.1 thru 2.3
Field Exploration Appendix A
Laboratory Tests Appendix B

-ΩΩΩΩΩΩΩΩΩΩ-

Respectfully submitted,

GEOLABS, INC.

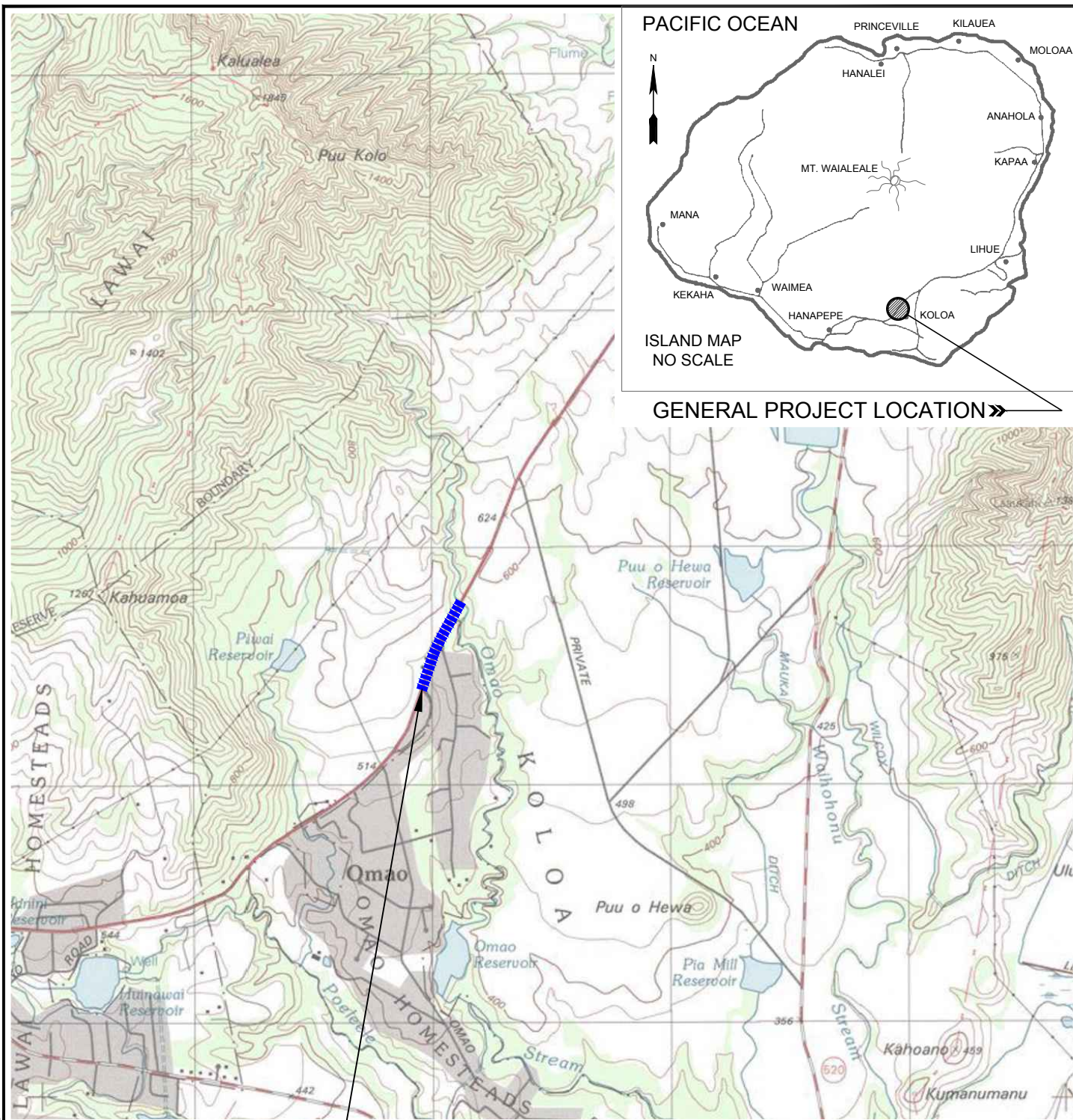
By 
Glenn Barut, P.E.
Senior Project Engineer

GS:GB:lf

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PLATES

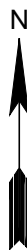
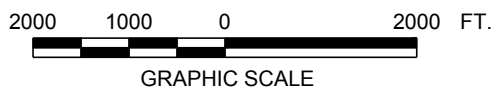
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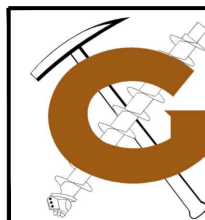
PROJECT LOCATION»

PROJECT LOCATION MAP

KAUMUALII HIGHWAY DRAINAGE IMPROVEMENTS
VICINITY OF OMAO STREAM BRIDGE
OMAO, KAUAI, HAWAII



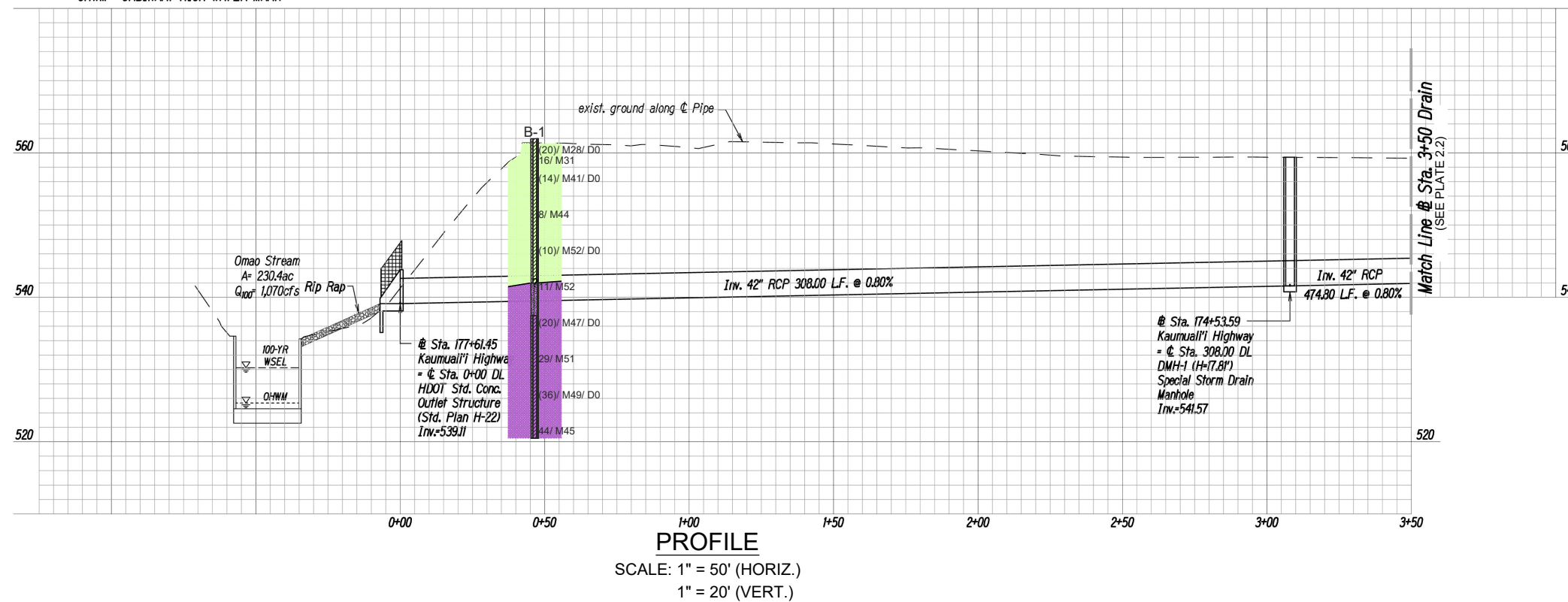
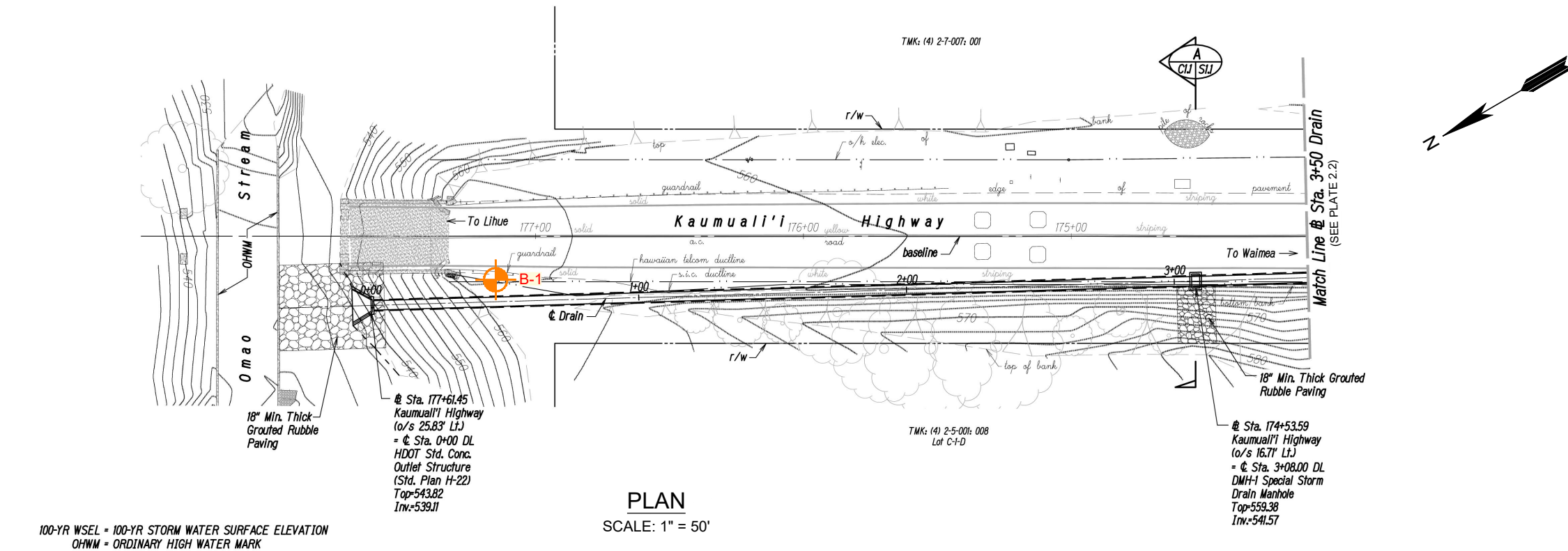
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GEOLABS, INC.

Geotechnical Engineering

DATE	DRAWN BY	PLATE
FEBRUARY 2026	HYC	1
SCALE	W.O.	
1" = 2,000'	8920-10	



LEGEND:



APPROXIMATE BORING LOCATION (FROM REPORT BY
GEOLABS, INC., W.O. 8920-00 DATED JULY 9, 2025)



RESIDUAL SOIL



SAPROLITE

REFERENCE: PLAN AND PROFILE PREPARED BY PAREN, INC. DBA
PARK ENGINEERING DATED SEPTEMBER 2025.

PLAN AND PROFILE - 1

KAUMUALII HIGHWAY DRAINAGE IMPROVEMENTS
VICINITY OF OMAO STREAM BRIDGE
OMAO, KAUAI, HAWAII



GEOLABS, INC.

Geotechnical Engineering

DATE
FEBRUARY 2026

DRAWN BY
HYC

PLATE

SCALE
HORIZ: 1" = 50'
VERT: 1" = 20'

W.O.	8920-10
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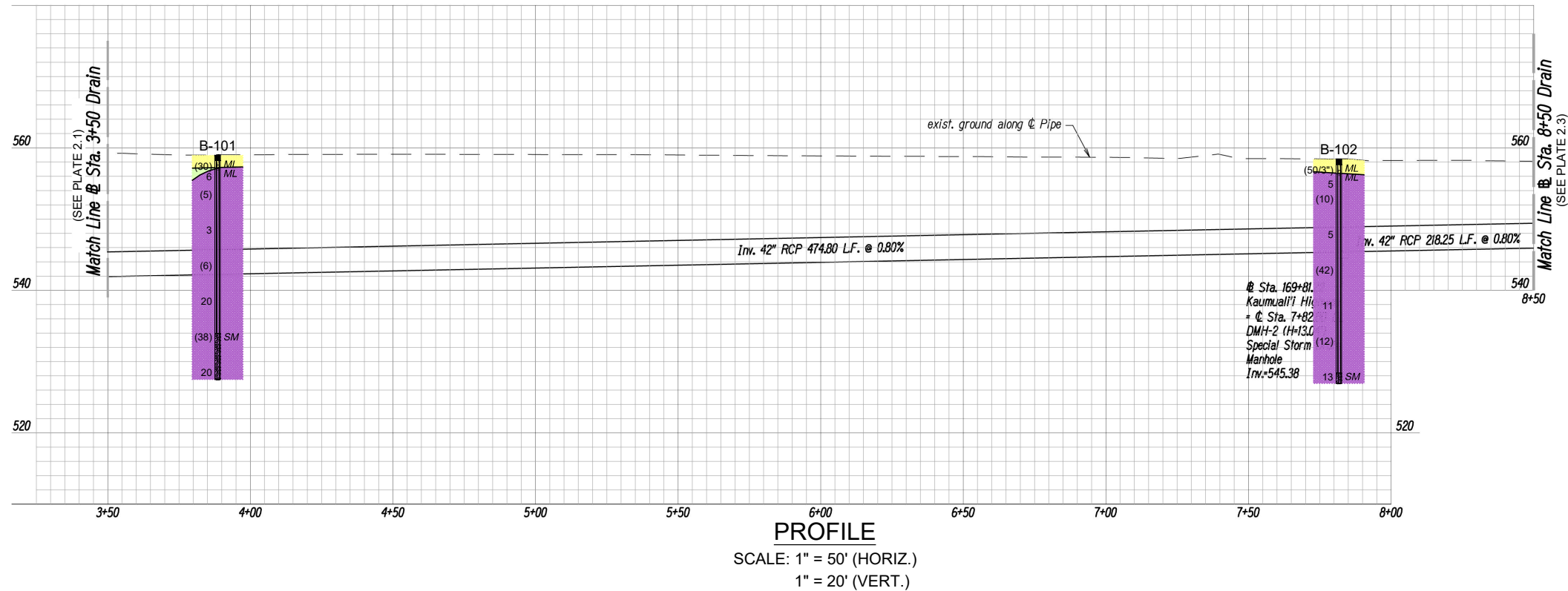
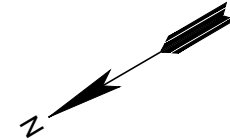
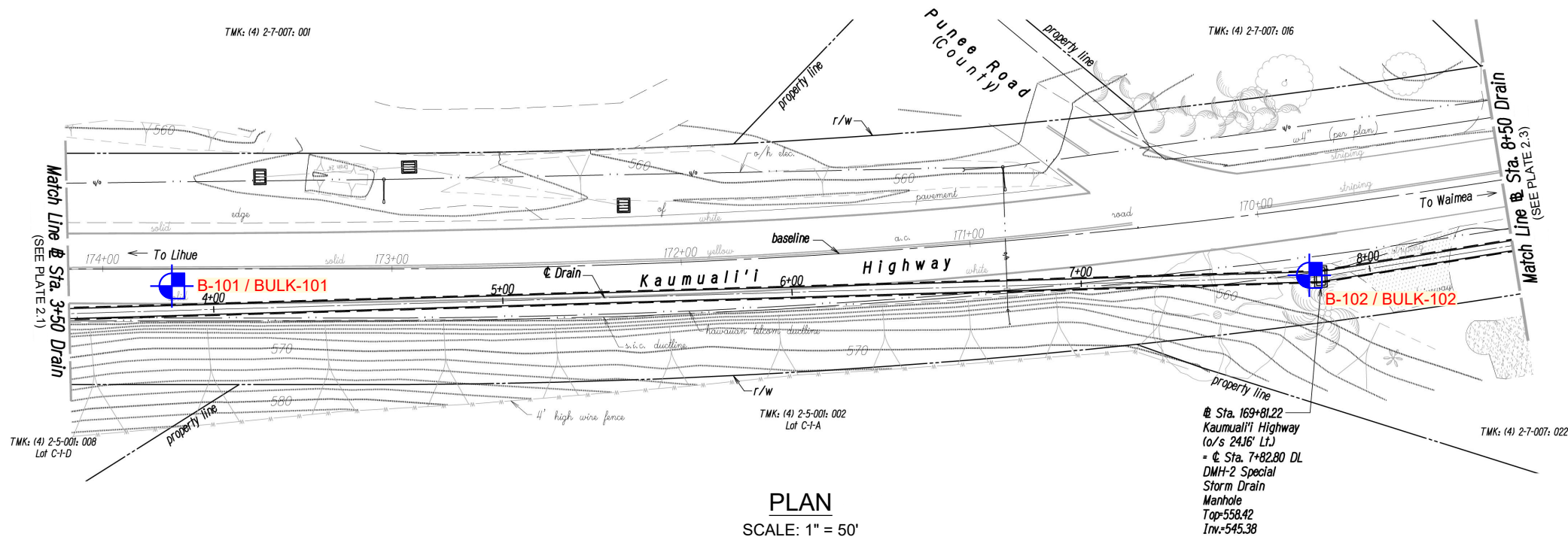
2.1

HORIZONTAL 0 25 50 100 150 FT.
VERTICAL 0 10 20 40 60 FT.



GRAPHIC SCALE

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LEGEND:



APPROXIMATE BORING LOCATION

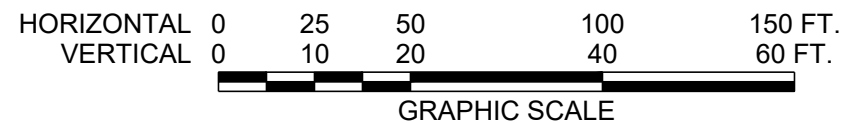
APPROXIMATE BULK SAMPLE LOCATION



FILL

RESIDUAL SOIL

SAPROLITE



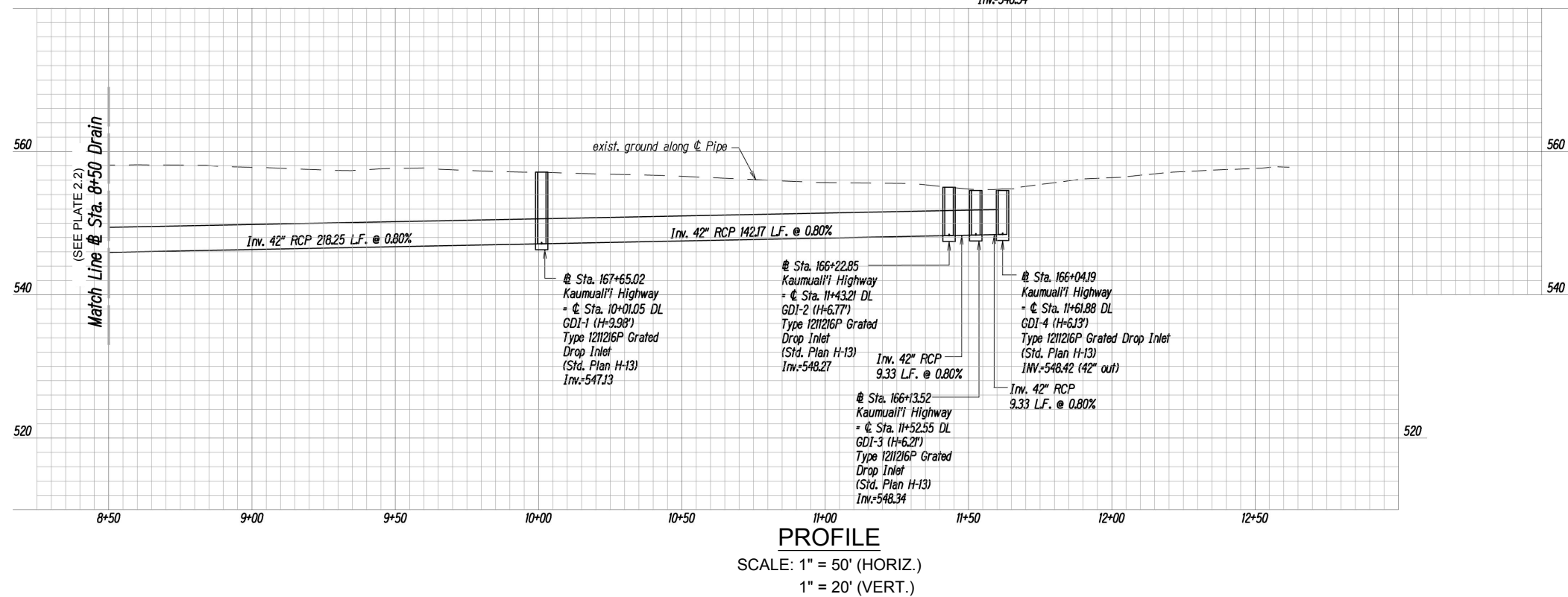
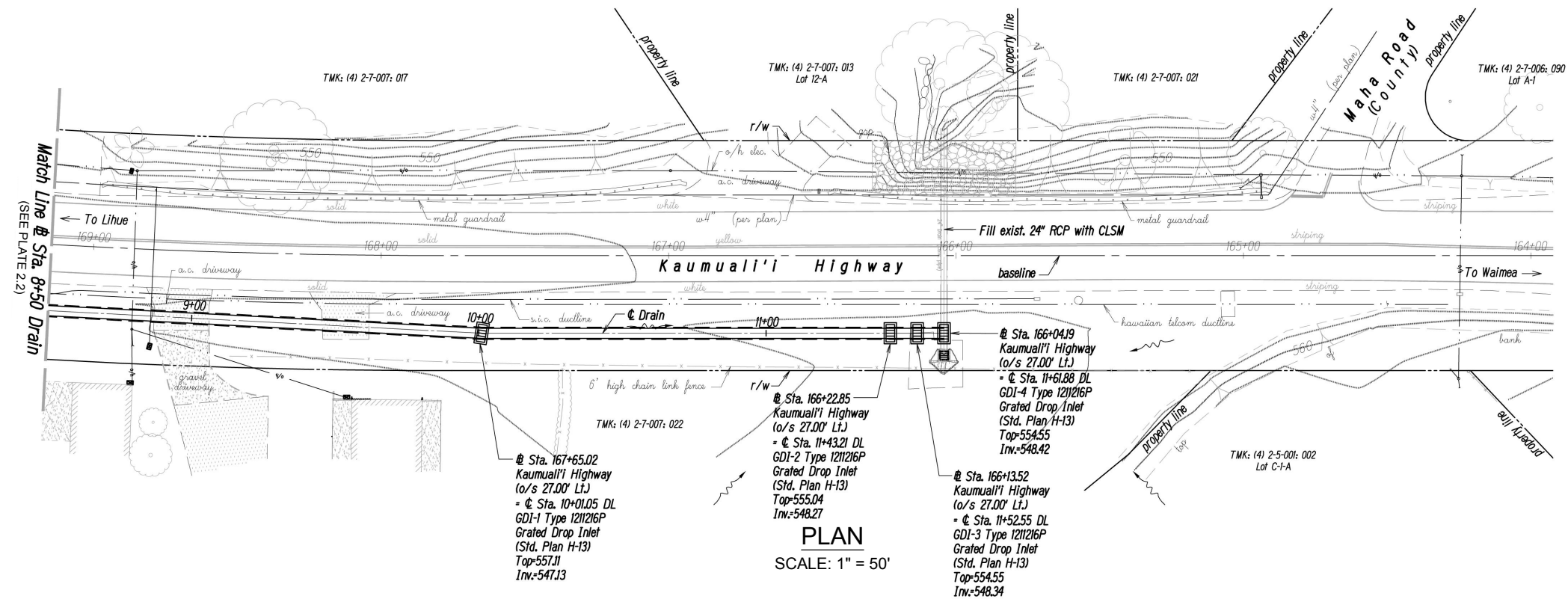
REFERENCE: PLAN AND PROFILE PREPARED BY PAREN, INC. DBA
PARK ENGINEERING DATED SEPTEMBER 2025.



GEOLABS, INC.
Geotechnical Engineering

DATE FEBRUARY 2026	DRAWN BY HYC	PLATE 2.2
SCALE HORIZ: 1" = 50' VERT: 1" = 20'	W.O. 8920-10	

NOTE: NO BORING ON THIS PLATE.

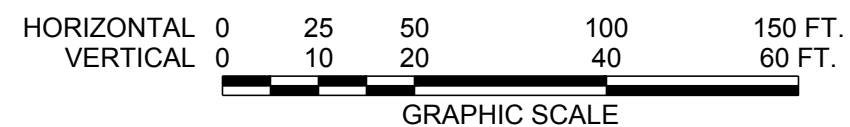


PLAN AND PROFILE - 3

KAUMUALII HIGHWAY DRAINAGE IMPROVEMENTS VICINITY OF OMAO STREAM BRIDGE OMAO, KAUAI, HAWAII



GEOLABS, INC. <i>Geotechnical Engineering</i>		
DATE FEBRUARY 2026	DRAWN BY HYC	PLATE 2.3
SCALE HORIZ: 1" = 50' VERT: 1" = 20'	W.O. 8920-10	



REFERENCE: PLAN AND PROFILE PREPARED BY PAREN, INC. DBA
PARK ENGINEERING DATED SEPTEMBER 2025.

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APPENDIX A

APPENDIX A

Field Exploration

We explored the subsurface conditions at the project site by drilling and sampling two borings, designated as Boring Nos. 101 and 102, extending to a depth of about 31.5 feet below the existing ground surface. Two bulk samples of near-surface soils were obtained to evaluate the pavement support characteristics of near-surface soils. The approximate boring and bulk sample locations are shown on the Plan and Profile, Plates 2.1 and 2.2. The borings were drilled using a truck-mounted drill rig equipped with continuous flight augers.

Our geologist classified the materials encountered in the borings by visual and textural examination in the field in general accordance with ASTM D2488, Standard Practice for Description and Identification of Soils, and monitored the drilling operations on a near-continuous (full-time) basis. These classifications were further reviewed visually and by testing in the laboratory. Soils were classified in general accordance with ASTM D2487, Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System), as shown on the Soil Log Legend, Plate A-0.1. Deviations made to the soil classification in accordance with ASTM D2487 are described on the Soil Classification Log Key, Plate A-0.2. A graphic representation of the materials encountered is presented on the Logs of Borings, Plates A-1.1 and A-2.

Relatively “undisturbed” soil samples were obtained in general accordance with ASTM D3550, 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 D1586, 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 drive are shown as the “Penetration Resistance” on the Logs of Borings at the appropriate sample depths. The penetration resistance shown on the Logs of Borings indicates the number of blows required for the specific sampler type used. The blow counts may need to be factored to obtain the Standard Penetration Test (SPT) blow counts.

Pocket penetrometer tests were performed on selected cohesive soil samples retrieved in the field. The pocket penetrometer test provides an indication of the unconfined compressive strength of the sample. Pocket penetrometer test results are summarized on the Logs of Borings at the appropriate sample depths.



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Soil Log Legend

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)

MAJOR DIVISIONS			USCS		TYPICAL DESCRIPTIONS		
COARSE-GRAINED SOILS	GRAVELS	CLEAN GRAVELS		GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES		
		LESS THAN 5% FINES		GP	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES		
		GRAVELS WITH FINES		GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES		
		MORE THAN 12% FINES		GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES		
	SANDS	CLEAN SANDS		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES		
		LESS THAN 5% FINES		SP	POORLY-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES		
		SANDS WITH FINES		SM	SILTY SANDS, SAND-SILT MIXTURES		
		MORE THAN 12% FINES		SC	CLAYEY SANDS, SAND-CLAY MIXTURES		
FINE-GRAINED SOILS	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



(2-INCH) O.D. STANDARD PENETRATION TEST

(3-INCH) O.D. MODIFIED CALIFORNIA SAMPLE

SHELBY TUBE SAMPLE

GRAB SAMPLE

CORE SAMPLE



WATER LEVEL OBSERVED IN BORING AT TIME OF DRILLING



WATER LEVEL OBSERVED IN BORING AFTER DRILLING



WATER LEVEL OBSERVED IN BORING OVERNIGHT

LL LIQUID LIMIT (NP=NON-PLASTIC)

PI PLASTICITY INDEX (NP=NON-PLASTIC)

TV TORVANE SHEAR (tsf)

UC UNCONFINED COMPRESSION OR UNIAXIAL COMPRESSIVE STRENGTH

TXUU UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION (ksf)

Plate

A-0.1



GEOLABS, INC.

Geotechnical Engineering

Soil Classification Log Key

(with deviations from ASTM D2488)

GEOLABS, INC. CLASSIFICATION*

GRANULAR SOIL (- #200 <50%)

- **PRIMARY** constituents are composed of the largest percent of the soil mass. Primary constituents are capitalized and bold (i.e., **GRAVEL, SAND**)
- **SECONDARY** constituents are composed of a percentage less than the primary constituent. If the soil mass consists of 12 percent or more fines content, a cohesive constituent is used (**SILTY** or **CLAYEY**); otherwise, a granular constituent is used (**GRAVELLY** or **SANDY**) provided that the secondary constituent consists of 20 percent or more of the soil mass. Secondary constituents are capitalized and bold (i.e., **SANDY GRAVEL, CLAYEY SAND**) and precede the primary constituent.
- **accessory descriptions** compose of the following:
 - with some: >12%
 - with a little: 5 - 12%
 - with traces of: <5%accessory descriptions are lower cased and follow the Primary and Secondary Constituents (i.e., **SILTY GRAVEL with a little sand**)

COHESIVE SOIL (- #200 ≥50%)

- **PRIMARY** constituents are based on plasticity. Primary constituents are capitalized and bold (i.e., **CLAY, SILT**)
- **SECONDARY** constituents are composed of a percentage less than the primary constituent, but more than 20 percent of the soil mass. Secondary constituents are capitalized and bold (i.e., **SANDY CLAY, SILTY CLAY, CLAYEY SILT**) and precede the primary constituent.
- **accessory descriptions** compose of the following:
 - with some: >12%
 - with a little: 5 - 12%
 - with traces of: <5%accessory descriptions are lower cased and follow the Primary and Secondary Constituents (i.e., **SILTY CLAY with some sand**)

EXAMPLE: Soil Containing 60% Gravel, 25% Sand, 15% Fines. Described as: **SILTY GRAVEL** with some sand

RELATIVE DENSITY / CONSISTENCY

Granular Soils			Cohesive Soils			
N-Value (Blows/Foot)		Relative Density	N-Value (Blows/Foot)		PP Readings (tsf)	Consistency
SPT	MCS		SPT	MCS		
0 - 4	0 - 7	Very Loose	0 - 2	0 - 4		Very Soft
4 - 10	7 - 18	Loose	2 - 4	4 - 7	< 0.5	Soft
10 - 30	18 - 55	Medium Dense	4 - 8	7 - 15	0.5 - 1.0	Medium Stiff
30 - 50	55 - 91	Dense	8 - 15	15 - 27	1.0 - 2.0	Stiff
> 50	> 91	Very Dense	15 - 30	27 - 55	2.0 - 4.0	Very Stiff
			> 30	> 55	> 4.0	Hard

MOISTURE CONTENT DEFINITIONS

Dry: Absence of moisture, dry to the touch

Moist: Damp but no visible water

Wet: Visible free water

ABBREVIATIONS

WOH: Weight of Hammer

WOR: Weight of Drill Rods

SPT: Standard Penetration Test Split-Spoon Sampler

MCS: Modified California Sampler

PP: Pocket Penetrometer

GRAIN SIZE DEFINITION

Description	Sieve Number and / or Size
Boulders	> 12 inches (305-mm)
Cobbles	3 to 12 inches (75-mm to 305-mm)
Gravel	3-inch to #4 (75-mm to 4.75-mm)
Coarse Gravel	3-inch to 3/4-inch (75-mm to 19-mm)
Fine Gravel	3/4-inch to #4 (19-mm to 4.75-mm)
Sand	#4 to #200 (4.75-mm to 0.075-mm)
Coarse Sand	#4 to #10 (4.75-mm to 2-mm)
Medium Sand	#10 to #40 (2-mm to 0.425-mm)
Fine Sand	#40 to #200 (0.425-mm to 0.075-mm)

Plate

A-0.2

*Soil descriptions are based on ASTM D2488-09a, Visual-Manual Procedure, with the above modifications by Geolabs, Inc. to the Unified Soil Classification System (USCS).



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Geotechnical Engineering

KAUMUALII HIGHWAY DRAINAGE IMPROVEMENTS
VICINITY OF OMAO STREAM BRIDGE
OMAO, KAUAI, HAWAII

Log of
Boring

101

Laboratory			Field				Depth (feet)	Sample	Graphic	USCS	Approximate Ground Surface Elevation (feet): 559 *
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)					Description
Sieve - #200 = 96.8%	58	72			30	4.5				ML	9-inch ASPHALTIC CONCRETE
	49				6					ML	Dark brown CLAYEY SILT with some sand and a little gravel, very stiff to hard, moist (fill)
	76	46			5	1.0	5				Brown with multi-color mottling CLAYEY SILT with traces of sand and gravel, medium stiff to very stiff, moist (saprolite) grades to soft
	49				3		10				
LL=44 PI=5											
TXUU S _u =1.7 ksf	41	60			6	0.8	15				grades to stiff
	44				20		20				grades to very stiff
UC=4.4 ksf Sieve - #200 = 39.6%	40	82			38	4.5	25			SM	Brown SILTY SAND with traces of gravel, medium dense, moist to very moist (saprolite)
LL=NP PI=NP	52				20		30				
											Boring terminated at 31.5 feet
											* Elevations estimated from Plan and Profile prepared by ParEn, Inc. dba Park Engineering dated September 2025.

Date Started: November 24, 2025

Date Completed: November 24, 2025

Logged By: Peter Czaplicki

Total Depth: 31.5 feet

Work Order: 8920-10

Water Level: ▼ Not Encountered

Drill Rig: CME-55D

Drilling Method: 4" Solid-Stem Auger

Driving Energy: 140 lb. wt., 30 in. drop

Plate

A - 1



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Geotechnical Engineering

KAUMUALII HIGHWAY DRAINAGE IMPROVEMENTS
VICINITY OF OMAO STREAM BRIDGE
OMAO, KAUAI, HAWAII

Log of
Boring

102

Laboratory			Field				Depth (feet)	Sample	Graphic	USCS	Approximate Ground Surface Elevation (feet): 558 *
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)					Description
Sieve - #200 = 61.8%	48	74			50/3"	1.3					9-inch ASPHALTIC CONCRETE
	46				5					ML	Dark brown CLAYEY SILT with some sand and a little gravel, hard, moist (fill)
	56				10					ML	Brown with multi-color mottling SANDY SILT with a little gravel, medium stiff, moist (saprolite)
LL=49 PI=7 UC=3.4 ksf	38	83			42	4.0					grades to very stiff
	40				11						grades with boulders
	45				12						grades to stiff
TXUU S _u =2.6 ksf	75				13	3.0					grades to very stiff
	50										
LL=NP PI=NP										SM	Brown SILTY SAND , medium dense, wet (saprolite)
											Boring terminated at 31.5 feet

Date Started: November 24, 2025

Date Completed: November 24, 2025

Logged By: Peter Czaplicki

Total Depth: 31.5 feet

Work Order: 8920-10

Water Level: ▼ Not Encountered

Drill Rig: CME-55D

Drilling Method: 4" Solid-Stem Auger

Driving Energy: 140 lb. wt., 30 in. drop

Plate

A - 2

APPENDIX B

APPENDIX B

Laboratory Tests

Moisture Content (ASTM D2216) and Unit Weight (ASTM D2937) determinations were performed on selected 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.

Four Atterberg Limits tests (ASTM D4318) were performed on selected soil samples to evaluate the liquid and plastic limits and to aid in soil classification. The test results are summarized on the Logs of Borings at the appropriate sample depths. A graphic presentation of the test results is provided on Plate B-1.

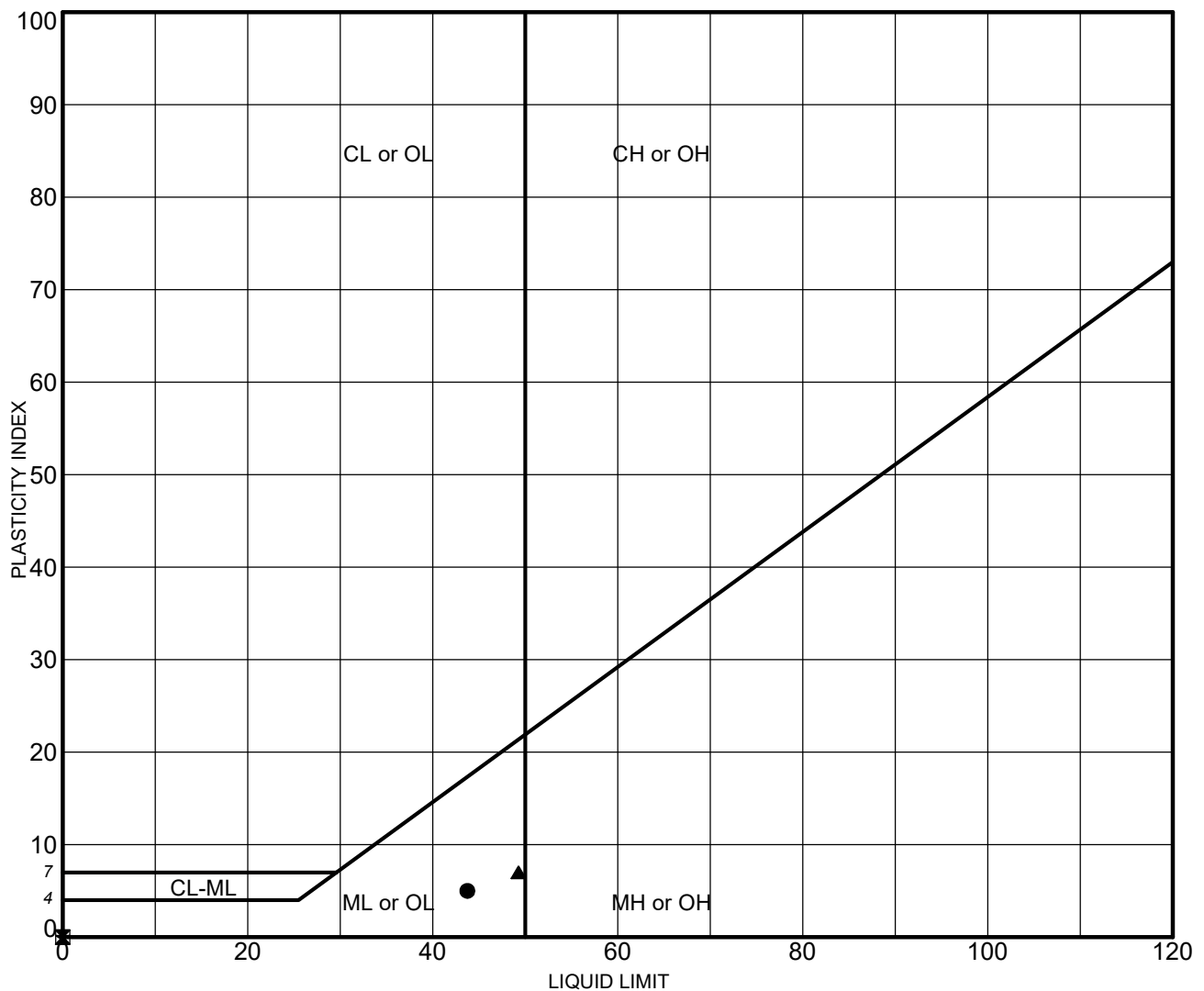
Three Sieve Analysis tests (ASTM D6913) were performed on selected soil samples to evaluate the gradation characteristics of the soils and to aid in soil classification. Graphic presentations of the grain size distributions are provided on Plate B-2.

Two Unconsolidated Undrained Triaxial Compression (TXUU) tests (ASTM D2850) were performed on selected soil samples to evaluate the undrained shear strength of the on-site soils. The approximate in-situ effective overburden pressure was used as the applied confining pressure for the relatively “undisturbed” soil samples. The test results and the stress-strain curves are presented on Plates B-3 and B-4.

Two Unconfined Compression tests (ASTM D2166) were performed on selected soil samples to evaluate the unconfined compressive strength of the on-site soils. The test results and stress-strain curves are plotted on Plates B-5 and B-6.

Two California Bearing Ratio (CBR) tests (ASTM D1883) were performed on bulk samples of the near-surface soils to evaluate the strength characteristics for pavement subgrade support. The test results are presented on Plates B-7 and B-8.

One set of corrosivity test, including pH (ASTM G51), minimum resistivity (ASTM G57), Chloride Content (ASTM D4327), and Sulfate Content (ASTM D4327), was performed by our office and CERCO Analytical, Inc. on a selected soil sample obtained from our field exploration. The test results are presented on Plate B-9.



	Sample	Depth (ft)	LL	PL	PI	Description
●	B-101	10.0-11.5	44	39	5	Brown with multi-color mottling clayey (ML) silt with traces of sand and gravel
☒	B-101	30.0-31.5	NP	NP	NP	Brown silty sand with traces of gravel
▲	B-102	15.0-16.5	49	42	7	Brown with multi-color mottling sandy silt (ML) with a little gravel
★	B-102	30.0-31.5	NP	NP	NP	Brown silty sand

NP = NON-PLASTIC

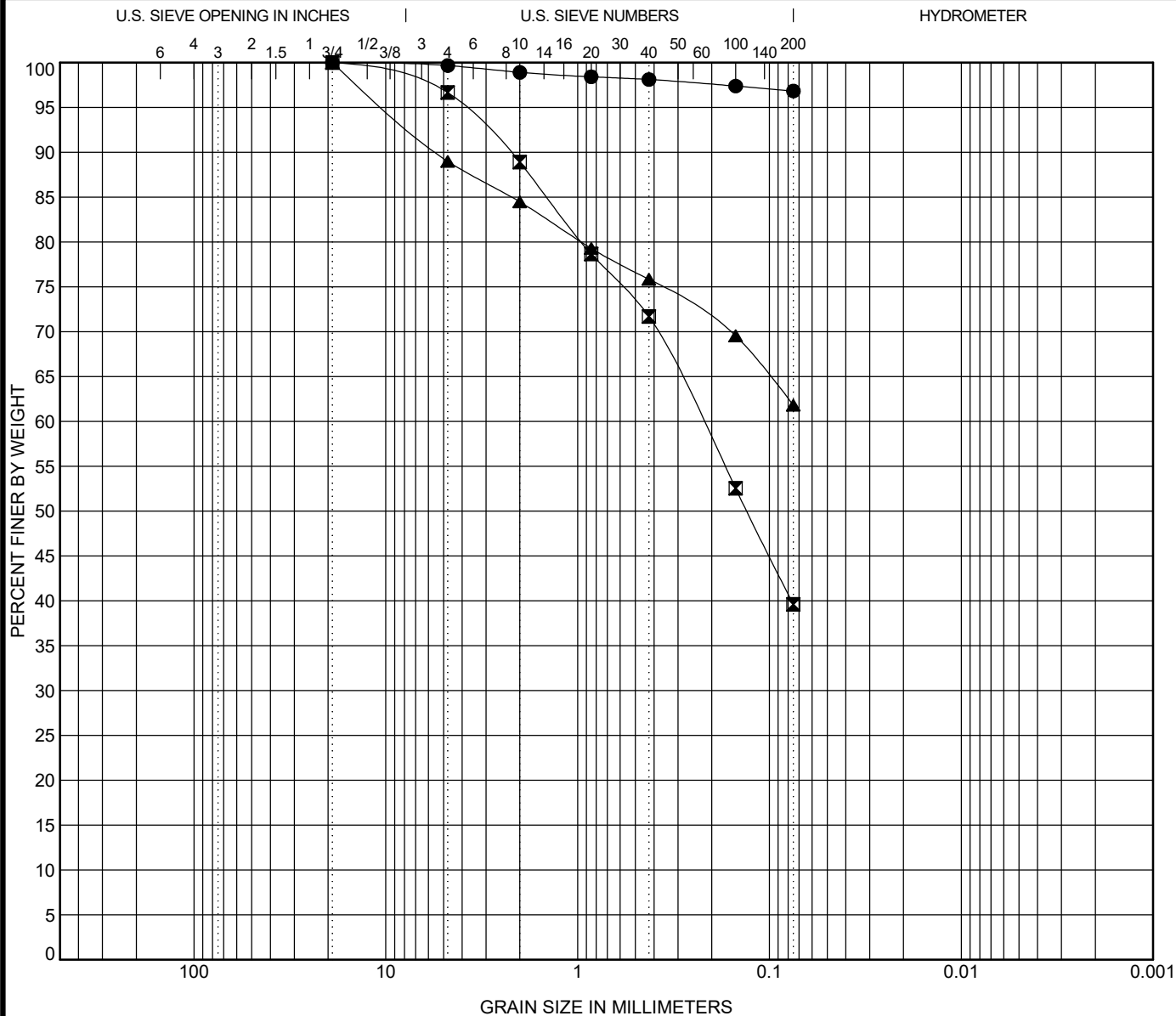


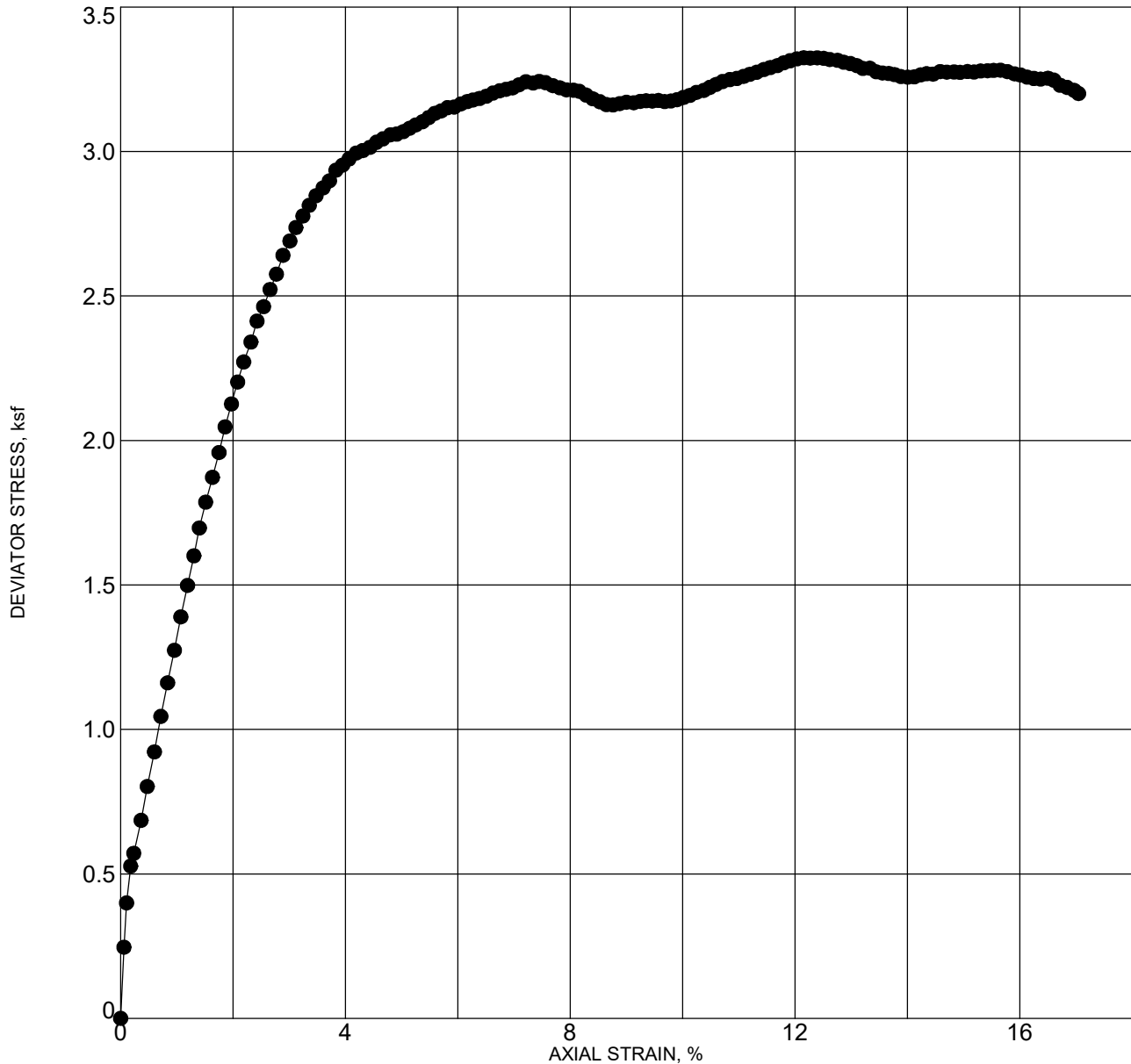
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ATTERBERG LIMITS TEST RESULTS - ASTM D4318

KAUMUALII HIGHWAY DRAINAGE IMPROVEMENTS
 VICINITY OF OMAO STREAM BRIDGE
 OMAO, KAUAI, HAWAII

Plate
B - 1





Max. Deviator Stress (ksf): 3.3

Confining Stress (ksf): 1.5

Location: B-101

Depth: 15.0 - 16.5 feet

Description: Brown with multi-color mottling clayey silt with traces of sand and gravel

Test Date: 1/7/2026

Dry Density (pcf)	61.5	Sample Diameter (inches)	2.400
Moisture (%)	38.0	Sample Height (inches)	5.000
Axial Strain at Failure (%)	12.8	Strain Rate (% / minute)	0.70



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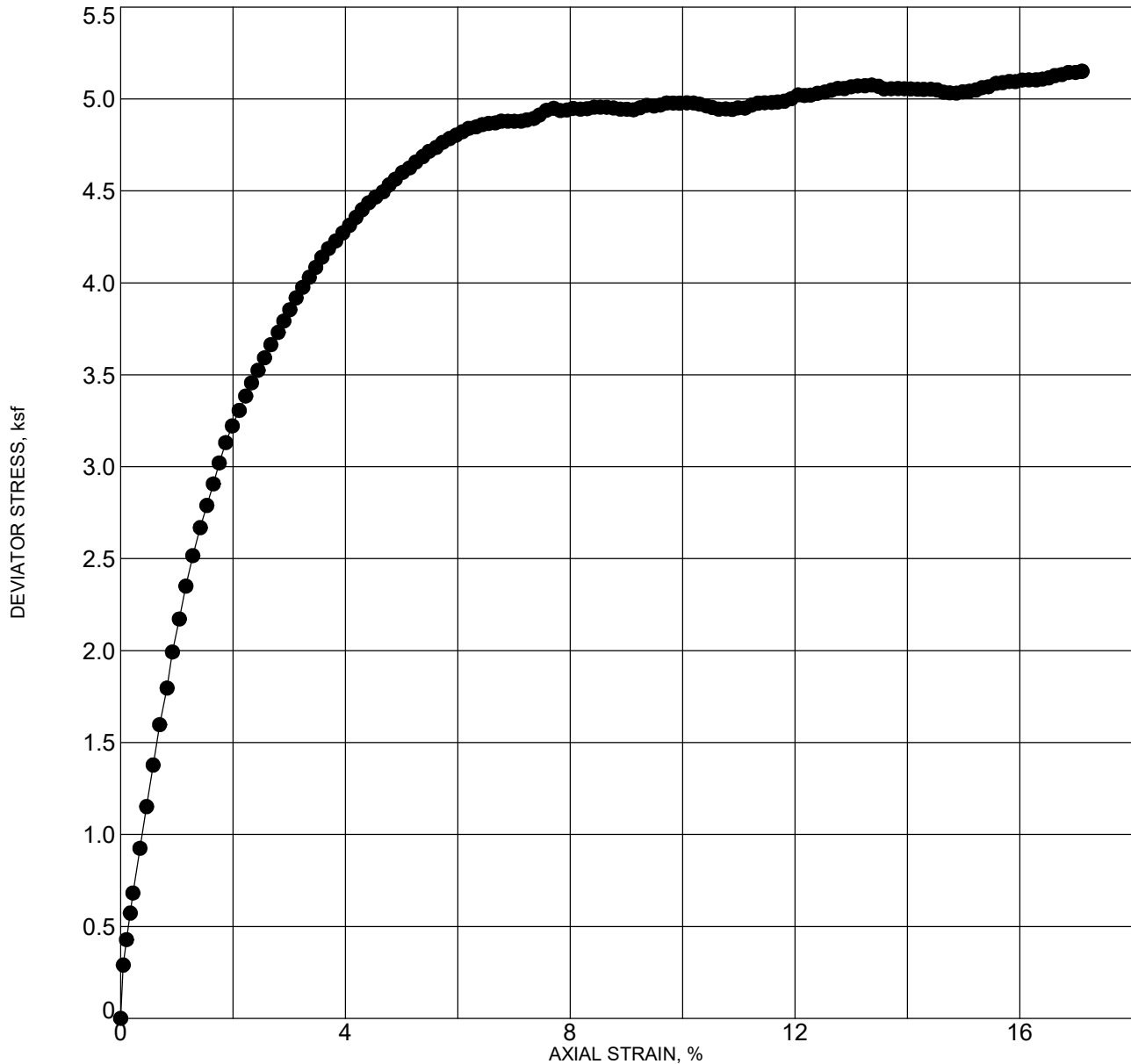
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TRIAXIAL UU COMPRESSION TEST - ASTM D2850

KAUMUALII HIGHWAY DRAINAGE IMPROVEMENTS
VICINITY OF OMAO STREAM BRIDGE
OMAO, KAUAI, HAWAII

Plate
B - 3



Max. Deviator Stress (ksf): 5.2

Confining Stress (ksf): 2.5

Location: B-102

Depth: 25.0 - 26.5 feet

Description: Brown with multi-color mottling sandy silt with a little gravel

Test Date: 1/7/2026

Dry Density (pcf)	73.9	Sample Diameter (inches)	2.400
Moisture (%)	47.4	Sample Height (inches)	5.000
Axial Strain at Failure (%)	13.4	Strain Rate (% / minute)	0.70



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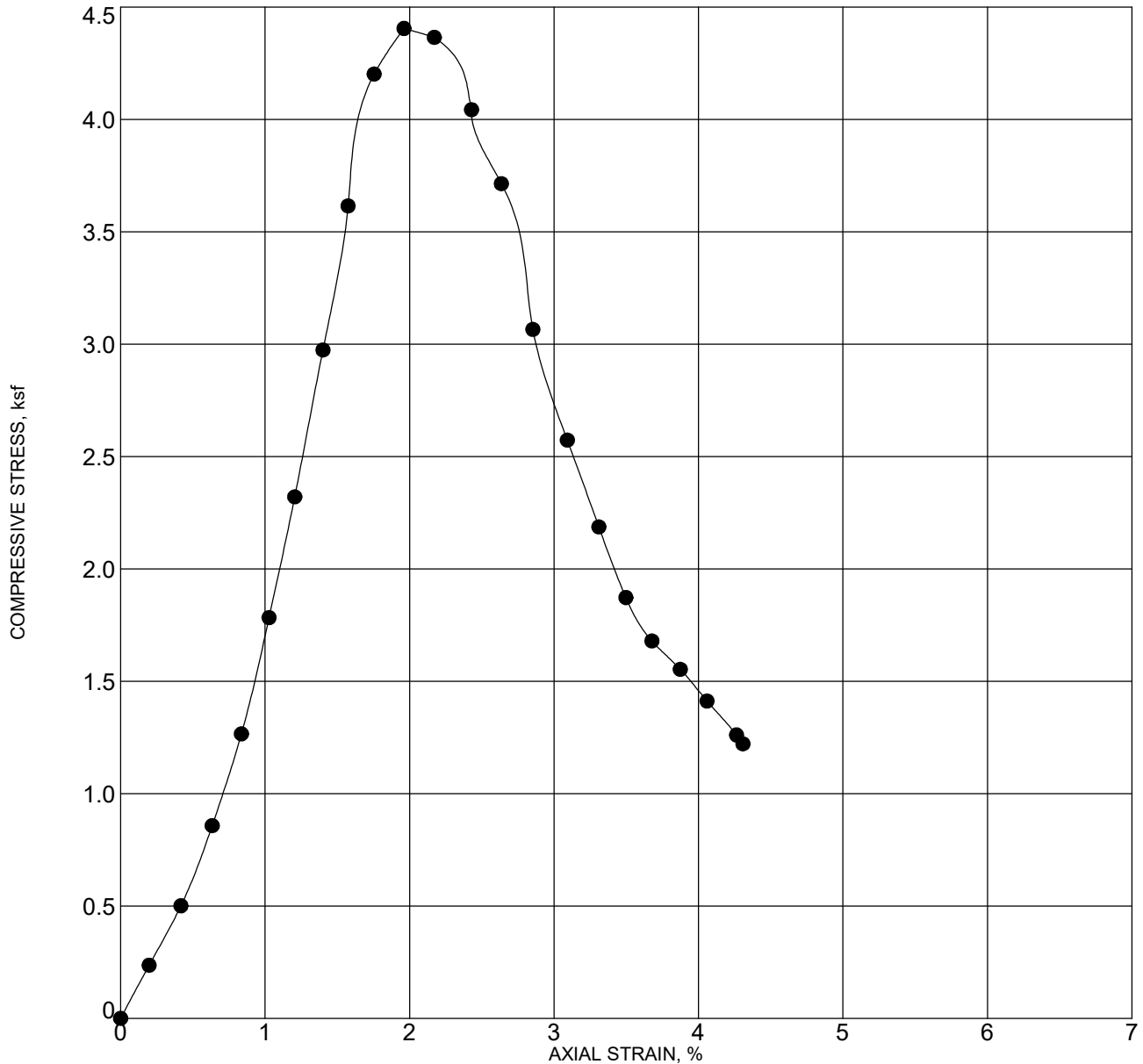
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TRIAXIAL UU COMPRESSION TEST - ASTM D2850

KAUMUALII HIGHWAY DRAINAGE IMPROVEMENTS
VICINITY OF OMAO STREAM BRIDGE
OMAO, KAUAI, HAWAII

Plate
B - 4



Unconfined Compressive Strength (ksf):	4.40
Axial Strain at Failure (%):	2.0
Strain Rate (% / minute):	2.02

Location: B-101
 Depth: 25.0 - 26.5 feet
 Description: Brown silty sand (SM) with traces of gravel
 Test Date: 12/17/2025

Dry Density (pcf)	81.9	Sample Diameter (inches)	2.400
Moisture (%)	39.7	Sample Height (inches)	5.000

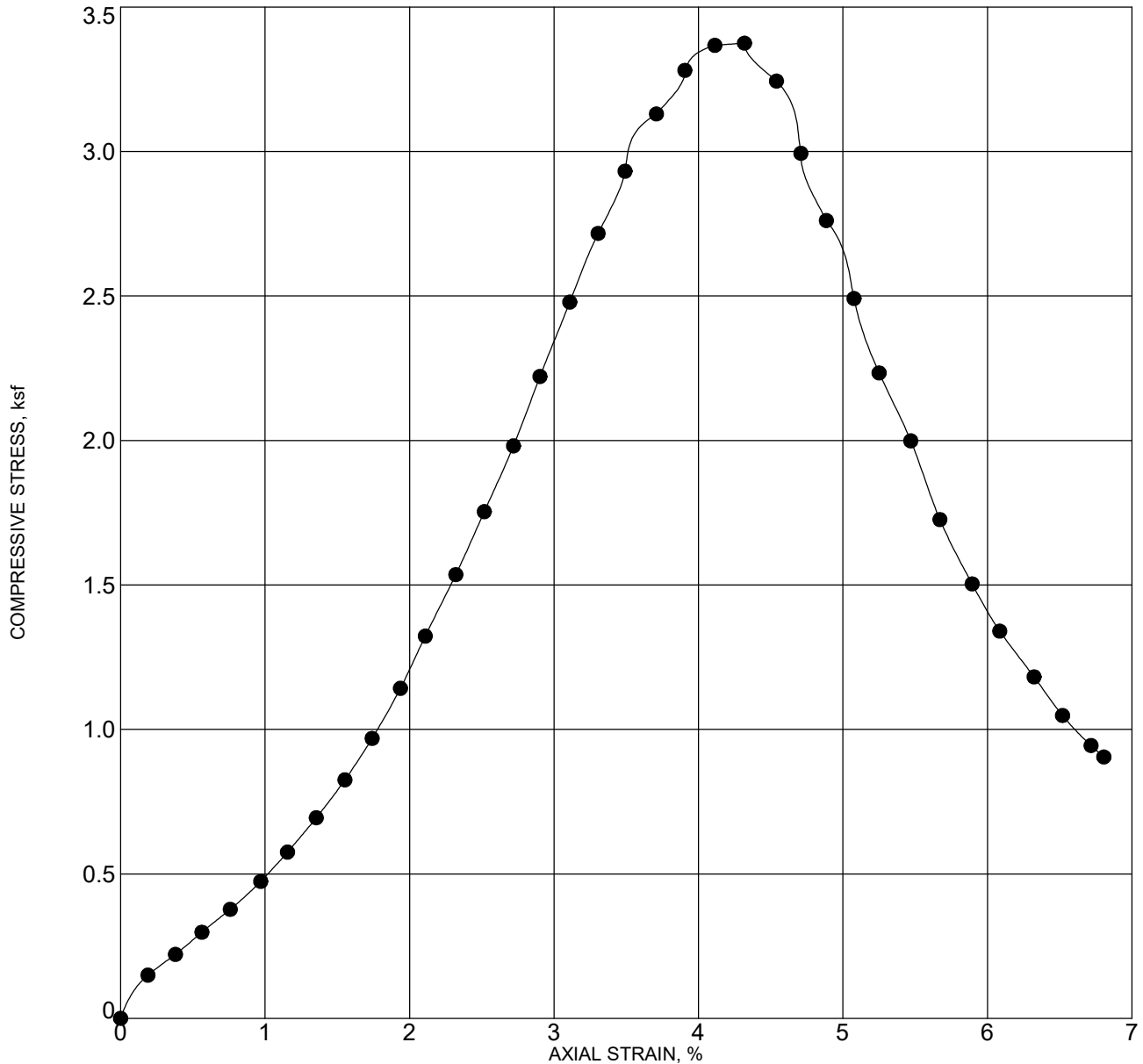


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UNCONFINED COMPRESSION TEST - ASTM D2166

KAUMUALII HIGHWAY DRAINAGE IMPROVEMENTS
 VICINITY OF OMAO STREAM BRIDGE
 OMAO, KAUAI, HAWAII

Plate
B - 5

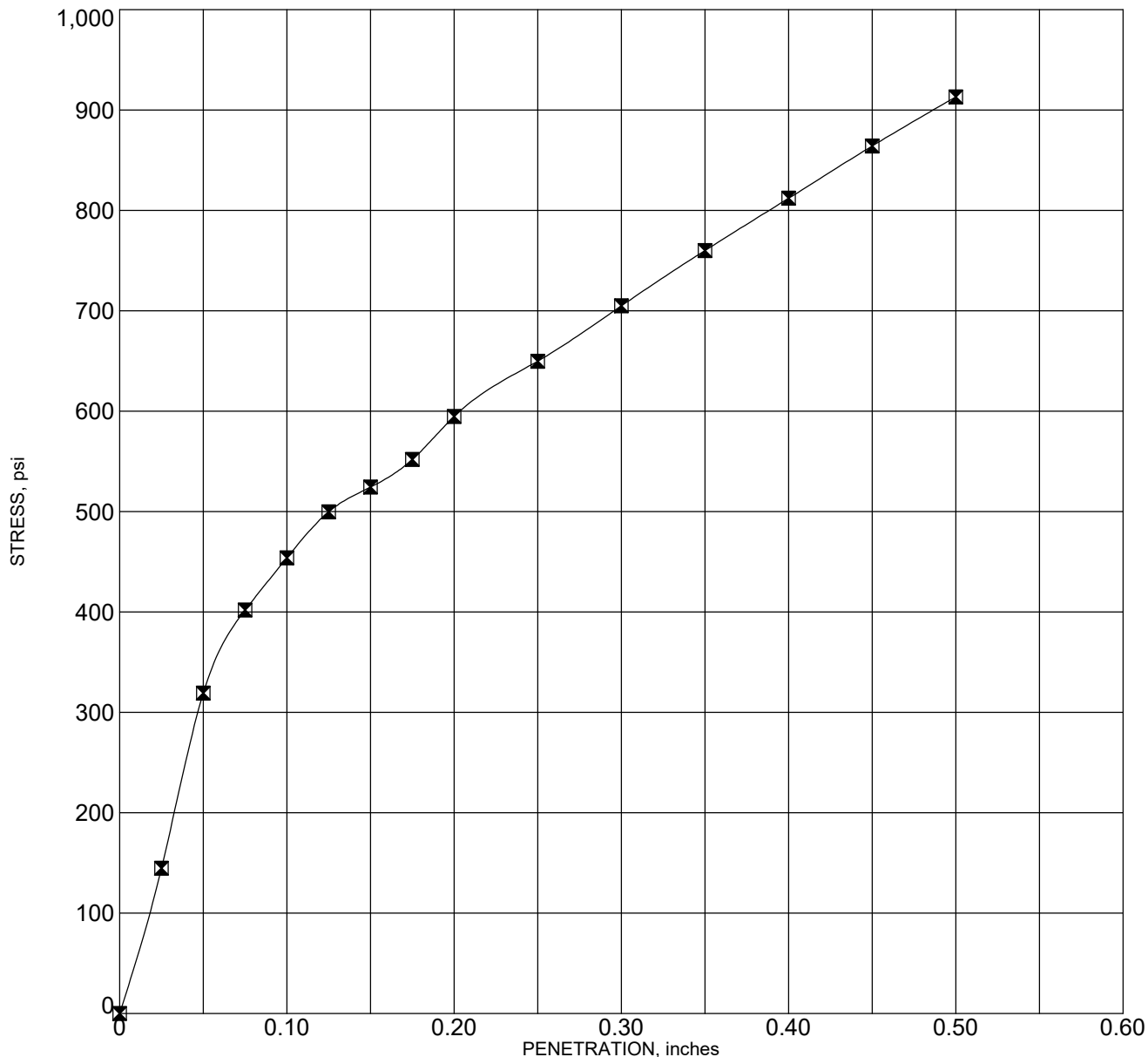


Unconfined Compressive Strength (ksf):	3.37
Axial Strain at Failure (%):	4.3
Strain Rate (% / minute):	0.50

Location: B-102
 Depth: 15.0 - 16.5 feet
 Description: Brown with multi-color mottling sandy silt (ML) with a little gravel
 Test Date: 12/17/2025

Dry Density (pcf)	83.4	Sample Diameter (inches)	2.400
Moisture (%)	37.7	Sample Height (inches)	5.000

	GEOLABS, INC. GEOTECHNICAL ENGINEERING W.O. 8920-10	UNCONFINED COMPRESSION TEST - ASTM D2166	
		KAUMUALII HIGHWAY DRAINAGE IMPROVEMENTS VICINITY OF OMAO STREAM BRIDGE OMAO, KAUAI, HAWAII	Plate B - 6



Sample: Bulk-101
 Depth: 1.0 - 4.0 feet
 Description: Brown clayey silt with some sand

Corr. CBR @ 0.1"	45.4
Corr. CBR @ 0.2"	39.6
Swell (%)	0.55

Molding Dry Density (pcf)	96.7	Hammer Wt. (lbs)	10
Molding Moisture (%)	27.7	Hammer Drop (inches)	18
Days Soaked	7	No. of Blows	56
Aggregate	3/4 inch minus	No. of Layers	5

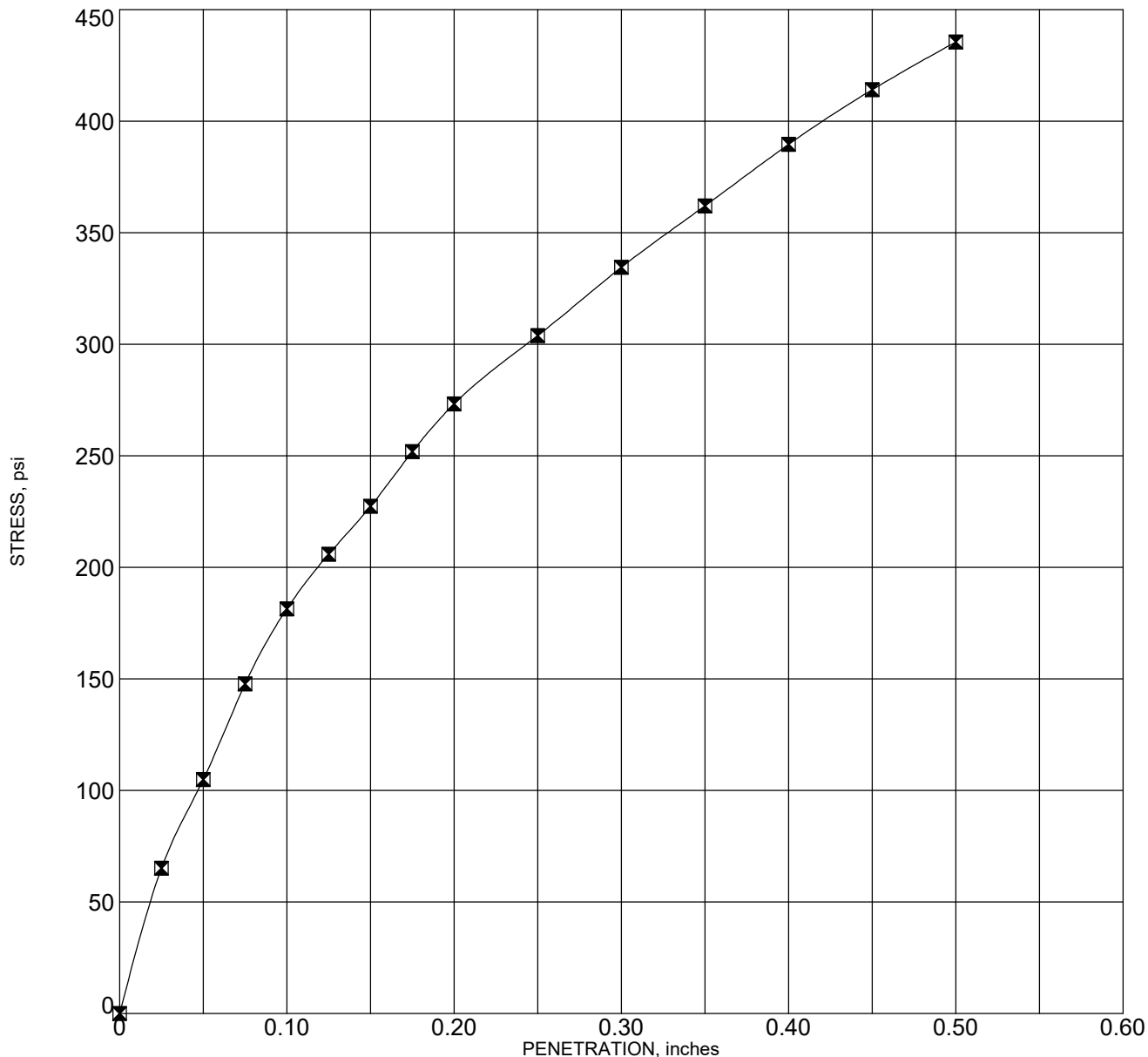


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CALIFORNIA BEARING RATIO - ASTM D1883

KAUMUALII HIGHWAY DRAINAGE IMPROVEMENTS
 VICINITY OF OMAO STREAM BRIDGE
 OMAO, KAUAI, HAWAII

Plate
B - 7



Sample: Bulk-102
 Depth: 1.0 - 4.0 feet
 Description: Brown sandy silt with a little clay

Corr. CBR @ 0.1"	18.1
Corr. CBR @ 0.2"	18.2
Swell (%)	2.44

Molding Dry Density (pcf)	102.4	Hammer Wt. (lbs)	10
Molding Moisture (%)	21.9	Hammer Drop (inches)	18
Days Soaked	7	No. of Blows	56
Aggregate	3/4 inch minus	No. of Layers	5



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CALIFORNIA BEARING RATIO - ASTM D1883

KAUMUALII HIGHWAY DRAINAGE IMPROVEMENTS
 VICINITY OF OMAO STREAM BRIDGE
 OMAO, KAUAI, HAWAII

Plate
B - 8

Location	Depth (feet)	pH Value	Minimum Resistivity (ohm-cm)	Chloride Content (mg/kg)	Sulfate Content (mg/kg)
B-101	20.0 - 21.5	5.4*	7500*	26	47

TEST METHODS (by CERCO Analytical, Inc.)

pH Value N/A
 Minimum Resistivity N/A
 Chloride Content ASTM D4327
 Sulfate Content ASTM D4327

ND: Not Detected Within Reporting Limits

TEST METHODS (by Geolabs, Inc.)*

pH Value ASTM G51
 Minimum Resistivity ASTM G57
 Chloride Content N/A
 Sulfate Content N/A

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GEOTECHNICAL ENGINEERING

W.O. 8920-10

SUMMARY OF CORROSIVITY TESTS

KAUMUALII HIGHWAY DRAINAGE IMPROVEMENTS
 VICINITY OF OMAO STREAM BRIDGE
 OMAO, KAUAI, HAWAII

Plate
B - 9