GEOTECHNICAL ENGINEERING EXPLORATION
CENTRAL MAUI TRANSIT HUB
KAHULUI, MAUI, HAWAII

W.O. 7798-00   JULY 19, 2019

Prepared for

FUKUMOTO ENGINEERING, INC.

GEOLABS, INC.
Geotechnical Engineering and Drilling Services

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Prepared for

FUKUMOTO ENGINEERING, INC.

GERALD Y. SEKI
No. 5635-C

THIS WORK WAS PREPARED BY
ME OR UNDER MY SUPERVISION.

SIGNATURE   4-30-20
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GEOLABS, INC.

Geotechnical Engineering and Drilling Services

July 19, 2019
W.O. 7798-00

Mr. Michael Silva, P.E.
Fukumoto Engineering, Inc.
1721 Wili Pa Loop, Suite 203
Honolulu, HI 96814

Dear **Mr. Silva:**

Geolabs, Inc. is pleased to submit our report entitled “Geotechnical Engineering Exploration, Central Maui Transit Hub, Kahului, Maui, Hawaii,” prepared in support of the design of the new transit hub facility project.

Our work was performed in general accordance with the scope of services outlined in our fee proposal dated June 20, 2018.

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

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

Very truly yours,

GEOLABS, INC.

Gerald Y. Seki, P.E.
Vice President

GS:JS:as

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Hawaii • California

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TABLE OF CONTENTS

Page

SUMMARY OF FINDINGS AND RECOMMENDATIONS iii

1. GENERAL 1
- 1.1 Project Considerations 1
- 1.2 Purpose and Scope 1

2. SITE CHARACTERIZATION 3
- 2.1 Regional Geology 3
- 2.2 Site Description 4
- 2.3 Subsurface Conditions 4
- 2.4 Seismic Design Considerations 5
    - 2.4.1 Earthquakes and Seismicity 5
    - 2.4.2 Liquefaction Potential 6
    - 2.4.3 Soil Profile 7

3. DISCUSSION AND RECOMMENDATIONS 9
- 3.1 Shallow Foundations 10
- 3.2 Slab-On-Grade 11
- 3.3 Drainage 13
- 3.4 Retaining Structures 13
    - 3.4.1 Retaining Structure Foundations 13
    - 3.4.2 Static Lateral Earth Pressures 14
    - 3.4.3 Dynamic Lateral Earth Forces 15
    - 3.4.4 Drainage 16
- 3.5 Site Grading 16
    - 3.5.1 Site Preparation 17
    - 3.5.2 Fills and Backfills 17
    - 3.5.3 Fill Placement and Compaction Requirements 18
    - 3.5.4 Excavation 19
- 3.6 Pavement Design 19
    - 3.6.1 Rigid Pavement Joints 22
    - 3.6.2 Pavement Drainage 22
- 3.7 Underground Utility Lines 23
- 3.8 Design Review 24
- 3.9 Post-Design Services/Services During Construction 24

4. LIMITATIONS 25

Page

CLOSURE.. 27

PLATES
- Project Location Map... Plate 1
- Site Plan .... Plate 2
- Typical Trench Detail... Plate 3

APPENDIX A
- Field Exploration...Pages A-1 and A-2
- Soil Log Legend...Plate A-0.1
- Soil Classification Log Key ....Plate A-0.2
- Logs of Borings...Plates A-1 thru A-7

APPENDIX B
- Laboratory Tests.. Page B-1
- Laboratory Test Data..Plates B-1 thru B-10

GEOTECHNICAL ENGINEERING EXPLORATION
CENTRAL MAUI TRANSIT HUB
KAHULUI, MAUI, HAWAII
W.O. 7798-00 JULY 19, 2019

SUMMARY OF FINDINGS AND RECOMMENDATIONS

Our field exploration generally encountered a pavement section consisting of 3 to 6 inches of asphaltic concrete underlain by 3 to 9 inches of medium dense sandy gravel and stiff sandy silts. The pavement section was generally underlain by loose to dense dune sand and beach deposits consisting of sandy and gravelly soils extending to depths of about 21.25 feet below the existing ground surface. The dune sand and beach deposits were underlain by alluvium consisting of loose to medium dense clayey gravel extending to the maximum depth explored of about 26.5 feet below the ground surface.

We encountered groundwater in the borings drilled during our field exploration at depths of about 5.3 and 5.7 feet below the existing ground surface. The groundwater levels encountered generally correspond to elevations of about +3.2 and +3.1 feet MSL, respectively, at the time of our field exploration. Due to the proximity of the project site to the Pacific Ocean, groundwater levels are expected to vary with tidal fluctuations and storm surge conditions. In addition, groundwater levels may change due to seasonal precipitation, surface water runoff, and other factors.

Based on the subsurface conditions encountered and anticipated loading for the new structures, we recommend supporting the new building and roof overhang structures on shallow foundations consisting of isolated spread and/or continuous strip footings. An allowable bearing pressure of up to 2,500 psf may be used to design the shallow foundations bearing on the recompacted on-site soils and/or new compacted fills needed to achieve the finished grades. This bearing value is for supporting dead-plus-live loads and may be increased by one-third for transient loads, such as those caused by wind or seismic forces. Bottom of footings should be embedded a minimum of 18 inches below the lowest adjacent finished grades.

We understand that both a flexible pavement section consisting of Asphaltic Concrete (AC) and a rigid pavement section consisting of Portland Cement Concrete (PCC) pavement will be considered for the project. Based on the traffic data provided and the strength of the subgrade materials, we recommend using a flexible pavement section consisting of 2.5 inches of asphaltic concrete on 4 inches of asphaltic concrete base on 6 inches of aggregate base course, or a rigid pavement structural section consisting of 9 inches of Portland cement concrete on 6 inches of aggregate subbase.

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 and engineering analyses performed in support of the design of the proposed Central Maui Transit Hub project in Kahului on the Island of Maui, Hawaii. The project location and general vicinity are shown on the Project Location Map, Plate 1.

This report summarizes the findings and presents our geotechnical recommendations based on our field exploration, laboratory testing, and engineering analyses. The recommendations presented herein are intended for the design of foundations, slabs-on-grade, retaining structures, site grading, pavements, and underground utility lines 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 at the southeast corner of parcel TMK: [2] 3-7-004: 003 in Kahului on the Island of Maui, Hawaii. The parcel is generally bounded by Vevau Street to the south, Kane Street to the west, School Street to the east, and Kaahumanu Avenue to the north.

Based on the information provided, it is desired to develop the site into a new bus facility with two small single-story buildings to house a ticket booth/office, electrical utility room, and restrooms. A new large roof overhang structure will be constructed to connect the new buildings and to provide cover for a new paved pedestrian waiting and bus loading/unloading area. The proposed roof overhang structure will have heights ranging between 16 and 29 feet. New pavements and a public sidewalk will also be constructed along Vevau Street. Furthermore, we understand that an employee parking area will be provided.

1.2 Purpose and Scope

The purpose of our field exploration was to obtain an overview of the surface and subsurface conditions to develop a generalized subsurface data set to formulate geotechnical recommendations for design of foundations, slabs-on-grade, retaining structures, site grading, pavements, and underground utility lines only. The scope of work for this exploration included the following tasks and work efforts:

1. Research and review of the available geotechnical engineering reports and in-house soil and geologic information related to the project area.
2. Staking out of boring locations and coordination of underground utility line clearance.
3. Obtain a permit to perform work on the County of Maui roadway.
4. Mobilization and demobilization of a truck-mounted drill rig and two operators to the project site and back.
5. Drilling and sampling of seven boreholes extending to depths ranging from about 6.5 to 26.5 feet below the existing ground surface for a total of approximately 85.5 lineal feet of exploration.
6. Coordination of the field exploration and logging of the borings by our geologist.
7. Laboratory testing of selected soil samples obtained during the field exploration as an aid in classifying the materials and evaluating their engineering properties.
8. Analyses of the field and laboratory test data to formulate geotechnical recommendations for the design of foundations, slabs-on-grade, retaining structures, site grading, pavements, and underground utility lines for the proposed project.
9. Preparation of this report summarizing our work and presenting our findings and recommendations.
10. Coordination of our overall work on the project by our engineer.
11. Quality assurance and client/design team consultation by our principal engineer.
12. 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 retrieved from our field exploration are presented in Appendix B.

END OF GENERAL

SECTION 2. SITE CHARACTERIZATION

2.1 Regional Geology

The Island of Maui was built by two major volcanoes, the older West Maui (Tertiary Epoch) and the more recent East Maui, also known as Haleakala (Pleistocene Epoch). The Isthmus of Maui is a narrow, gently sloping plain located between these two volcanoes. The project site is located at the northern portion of this gently sloping plain.

The Isthmus of Maui was created by lava flows from Haleakala ponding on West Maui. It is comprised of alluvium washed from the slopes of West Maui and East Maui (Haleakala). The erosional processes were dominated by the detachment of soil and rock masses from the mountain walls, and the soil materials were transported downslope toward the Isthmus primarily by gravity as colluvium. Once these materials reached the stream in the central portion of a valley, alluvial processes became dominant, and the sediments were transported and deposited as alluvium.

During the Pleistocene Epoch, the sea stood about 350 feet lower than the present sea level. During this period, sand was blown inland and extensive lithified calcareous dunes were formed on the Isthmus over the alluvial fans. Near the northern coast, some of the dunes are as much as 180 feet high.

In general, stream flows in Hawaii are intermittent and flashy, such that the stream flows transmit large volumes of water for a very short duration. Because of this situation, the 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 materials. Chemical processes also cause induration, or cementation, of the coarse-grained portion of the sediment into 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 downslope covering the older alluvial soil deposits. Depending on the local base level and rate of transport, these newer sediments are generally transient in terms of geologic time. In addition, their consistency and density are generally less than those of the older, partially consolidated deposits.

Underlying the alluvial soil deposits are overlapping lava flows from the West Maui and East Maui Volcanoes. The bulk of the Haleakala shield was built during the late Pliocene and early Pleistocene Epoch by thinly bedded basaltic lava flows of the Honomanu Volcanic Series. During the Pleistocene Epoch, the characteristics of the lava changed to very hard, thickly bedded flows of andesitic composition. These lava flows have been grouped as the Kula Volcanic Series. Typically, the basalt rock formation consists of thinly to thickly bedded a’a and pahoehoe type lava flows. Development of areas surrounding Kahului in the past several decades has brought the project site to its present conditions.

2.2 Site Description

The project site is located at the southeast corner of parcel TMK: [2] 3-7-004: 003 in Kahului on the Island of Maui, Hawaii. The parcel is located north of Vevau Street as shown on the Site Plan, Plate 2.

The project site was previously used for bus parking and is generally covered with asphaltic concrete pavement and landscaped areas. Based on our field observations and the topographic survey map provided, the project site is slightly sloping from south to north with existing ground surface elevations between about +8 and +10 feet Mean Sea Level (MSL).

2.3 Subsurface Conditions

Based on our field exploration results, the subsurface conditions encountered in our borings generally consists of a pavement section consisting of 3 to 6 inches of asphaltic concrete underlain by 3 to 9 inches of medium dense sandy gravel and stiff sandy silts. The pavement section was underlain by dune sand and beach deposits consisting of loose to dense sand, silty gravel, and gravelly sand extending to depths of about 21.25 feet below the existing ground surface. The dune sand and beach deposits were underlain by alluvium consisting of loose to medium dense clayey gravel extending to the maximum depth explored of about 26.5 feet below the ground surface.

We encountered groundwater in the borings drilled during our field exploration at depths of about 5.3 and 5.7 feet below the existing ground surface. The groundwater levels encountered generally correspond to elevations of about +3.2 and +3.1 feet MSL, respectively, at the time of our field exploration. Due to the proximity of the project site to the Pacific Ocean, groundwater levels are expected to vary with tidal fluctuations and storm surge conditions. In addition, groundwater levels may change due to seasonal precipitation, surface water runoff, and other factors.

Detailed descriptions of the field exploration methodology and graphic representations of the materials encountered in the borings are presented on the Logs of Borings in Appendix A. We performed laboratory tests on selected samples obtained during our field exploration, and the test results are presented in Appendix B.

2.4 Seismic Design Considerations

Based on the International Building Code, 2006 Edition (IBC 2006), the project site may be subject to seismic activity and seismic design considerations will need to be addressed. The following sections provide discussions on the seismicity, the potential for liquefaction at the project site, and the soil profile type for seismic design.

2.4.1 Earthquakes and Seismicity

In general, earthquakes that occur throughout the world are caused by shifts in the tectonic plates. In contrast, earthquake activity in Hawaii is linked primarily to volcanic activity. Therefore, earthquake activity in Hawaii generally occurs before or during volcanic eruptions. In addition, earthquakes may result from the underground movement of magma that comes close to the surface but does not erupt. The Island of Hawaii experiences thousands of earthquakes each year, but most of the earthquakes are so small that they only can be detected by sensitive instruments. However, some of the earthquakes are strong enough to be felt, and a few cause minor to moderate damage.

In general, earthquakes associated with volcanic activity are most common on the Island of Hawaii. Earthquakes directly associated with the movement of magma are concentrated beneath the active Kilauea and Mauna Loa Volcanoes on the Island of Hawaii. Because the majority of the earthquakes in Hawaii (over 90 percent) are related to volcanic activity, the risk of seismic activity and degree of ground shaking diminishes with increased distance from the Island of Hawaii.

The Island of Hawaii has experienced numerous earthquakes greater than Magnitude 5 (M5+); however, earthquakes are not confined only to the Island of Hawaii. To a lesser degree, the Island of Maui also has experienced earthquakes greater than M5+. Therefore, moderate to strong earthquakes have occurred in the County of Maui.

2.4.2 Liquefaction Potential

Based on the International Building Code (2006 Edition), the project site should be evaluated for the potential for soil liquefaction. Soil liquefaction is a condition where saturated cohesionless soils located near the ground surface undergo a substantial loss of strength due to the build-up of excess pore water pressures resulting from cyclic stress applications induced by earthquakes. In this process, when the loose saturated sand deposit is subjected to vibration (such as during an earthquake), the soil tends to densify and decrease in volume causing an increase in pore water pressure. If drainage is unable to occur rapidly enough to dissipate the build-up of pore water pressure, the effective stress (internal strength) of the soil is reduced. Under sustained vibrations, the pore water pressure build-up could equal the overburden pressure, essentially reducing the soil shear strength to zero and causing it to behave as a viscous fluid. During liquefaction, the soil acquires sufficient mobility to permit both horizontal and vertical movements, and if not confined, will result in significant deformations.

Soils most susceptible to liquefaction are loose, uniformly graded, fine-grained sands and loose silts with little cohesion. The major factors affecting the liquefaction characteristics of a soil deposit are as follows:

FACTORS	LIQUEFACTION SUSCEPTIBILITY
Grain Size Distribution	Fine and uniform sands and silts are more susceptible to liquefaction than coarse or well-graded sands.
Initial Relative Density	Loose sands and silts are most susceptible to liquefaction. Liquefaction potential is inversely proportional to relative density.
Magnitude and Duration of Vibration	Liquefaction potential is directly proportional to the magnitude and duration of the earthquake.

Our borings generally encountered medium dense sandy soils below the groundwater table. Loose sandy gravel soil was encountered in one of the drilled borings below the groundwater table; however, the layer was relatively thin in thickness and contained a significant amount of gravel. Therefore, it is our opinion that the phenomenon of soil liquefaction is not a design consideration for this project site. The risk for potential liquefaction is very low at this project site based on the subsurface conditions encountered (medium dense sandy soils overlying alluvium below the groundwater table).

2.4.3 Soil Profile

Based on the subsurface materials anticipated at the project site and the geologic setting of the area, we anticipate the project site may be classified from a seismic analysis standpoint as being a "Stiff Soil Profile" site corresponding to a Site Class D soil profile type based on the 2006 International Building Code (Table No. 1613.5.2). Based on Site Class D, the following seismic design parameters were estimated and may be used for seismic analysis of the project.

SEISMIC DESIGN PARAMETERS	
Parameter	**Value**
Mapped MCE Spectral Response Acceleration, S_S	0.979g
Mapped MCE Spectral Response Acceleration, S_1	0.251g
Site Class	“D”
Site Coefficient, F_a	1.108
Site Coefficient, F_v	1.898
Adjusted MCE Spectral Response Acceleration, S_{MS}	1.085g
Adjusted MCE Spectral Response Acceleration, S_{M1}	0.476g
Design Spectral Response Acceleration, S_{DS}	0.724g
Design Spectral Response Acceleration, S_{D1}	0.317g
Peak Bedrock Acceleration, PBA (Site Class B)	0.363g
Peak Ground Acceleration, PGA (Site Class D)	0.289g

END OF SITE CHARACTERIZATION

SECTION 3. DISCUSSION AND RECOMMENDATIONS

Our field exploration generally encountered a pavement section consisting of 3 to 6 inches of asphaltic concrete underlain by 3 to 9 inches of medium dense sandy gravel and stiff sandy silts. The pavement section was generally underlain by loose to dense dune sand and beach deposits consisting of sandy and gravelly soils extending to depths of about 21.25 feet below the existing ground surface. The dune sand and beach deposits were underlain by alluvium consisting of loose to medium dense clayey gravel extending to the maximum depth explored of about 26.5 feet below the ground surface. The groundwater levels encountered in the drilled borings generally correspond to elevations of about +3.1 and +3.2 feet MSL at the time of our field exploration.

Based on the subsurface conditions encountered and anticipated loading for the new structures, we recommend supporting the buildings and roof overhang structures on shallow foundations consisting of isolated spread and/or continuous strip footings. An allowable bearing pressure of up to 2,500 psf may be used to design the shallow foundations bearing on the recompacted on-site soils and/or new compacted fills needed to achieve the finished grades. Bottom of footings should be embedded a minimum of 18 inches below the lowest adjacent finished grades.

We understand that both a flexible pavement section consisting of Asphaltic Concrete (AC) and a rigid pavement section consisting of Portland Cement Concrete (PCC) pavement will be considered for the project. Based on the traffic data provided and the strength of the subgrade materials, we recommend using a flexible pavement section consisting of 2.5 inches of asphaltic concrete on 4 inches of asphaltic concrete base on 6 inches of aggregate base course, or a rigid pavement structural section consisting of 9 inches of Portland cement concrete on 6 inches of aggregate subbase.

Detailed discussions and recommendations for design of foundations, retaining structures, site grading, pavements and other geotechnical aspects of the project are presented in the following sections.

3.1 Shallow Foundations

In general, we believe shallow spread and/or continuous strip footings may be used to support the new single-story structures and roof overhang structure planned at the project site.

An allowable bearing pressure of up to 2,500 pounds per square foot (psf) may be used for the design of footings bearing on the recompacted on-site materials or compacted fill. This bearing value is for supporting dead-plus-live loads and may be increased by one-third for transient loads, such as those caused by wind or seismic forces. In general, bottom of foundations should be embedded a minimum of 18 inches below the lowest adjacent finished grade. Bottom of footings constructed near tops of slopes or on sloping ground should be embedded deep enough to provide a minimum horizontal setback distance of 6 feet measured from the outside edge of the footings to the face of the slope.

Due to the presence of relatively shallow groundwater, we recommend that the bottom of footing elevation for the shallow foundations be placed above +3.5 feet MSL to prevent the need for dewatering during preparation of the shallow foundation subgrade.

Foundations next to other foundations, utility trenches, or easements should be embedded below a 45-degree imaginary plane extending upward from the bottom edge of the utility trench, or the footings should extend to a depth as deep as the inverts of the utility lines. This requirement is necessary to avoid surcharging adjacent below-grade structures with additional structural loads and to reduce the potential for appreciable foundation settlement.

If soft and/or loose materials are encountered at the bottom of footing excavations, the soft and/or loose materials should be over-excavated until dense and/or stiff materials are exposed in the footing excavations. The over-excavation should be backfilled with select granular fill materials moisture-conditioned to above the optimum moisture content and compacted to a minimum of 90 percent relative compaction. Alternatively, the bottom of footing may be extended down to bear directly on the underlying competent materials.

If the foundations for the new structures are designed and constructed in strict accordance with our recommendations, total settlement of foundations is estimated to be on the order of 1 inch or less with differential settlements on the order of about 0.5 inches.

Lateral loads acting on the structures may be resisted by friction between the base of the foundation and the bearing materials and by passive earth pressure developed against the near-vertical faces of the embedded portion of foundations. A coefficient of friction of 0.35 may be used for foundations bearing on the recompacted on-site materials or compacted fill. Resistance to lateral loads due to passive earth pressure may be calculated using an equivalent fluid pressure of 350 pcf for foundations embedded in the on-site materials and/or compacted structural fill materials. These values assume the soils around the foundations are well-compacted. Unless covered by pavements or slabs, the passive pressure resistance in the upper 12 inches below the finished grade should be neglected.

A Geolabs representative should observe the footing excavations prior to the placement of reinforcing steel and concrete to confirm the foundation bearing conditions and the required embedment depths.

3.2 Slab-On-Grade

Based on the subsurface conditions and anticipated grading, we envision new concrete slabs-on-grade for the structures likely will bear on the on-site granular soils, and/or select granular fill materials; therefore, a conventional slab-on-grade design may be utilized for the on-grade structures.

If loose/soft soils are encountered at the subgrade level, these soils should be over-excavated by 2 feet or more below the slab subgrade elevation and replaced with select granular fill material compacted to a minimum of 90 percent relative compaction. This condition should be determined in the field based on the exposed soil conditions by a Geolabs representative. Therefore, subgrade preparation and excavation operations should be conducted under the observation of a Geolabs representative.

Prior to placing fill material, the subgrade should be scarified to a depth of about 8 inches, moisture-conditioned to above the optimum moisture content, and recompacted to a minimum of 90 percent relative compaction. Slab subgrades should be kept moist until covered by concrete. Saturation and subsequent yielding of the exposed subgrade due to inclement weather and poor drainage may require over-excavation of the soft areas and replacement with well-compacted fill.

For interior building slabs (not subjected to vehicular traffic or machinery vibration), we recommend placing a minimum 4-inch thick layer of cushion fill consisting of open-graded gravel, such as No. 3 Fine gravel (ASTM C33, No. 67 gradation), below the slabs and above the non-expansive select granular fill layer. The open-graded gravel cushion fill would provide uniform support of the slabs and would serve as a capillary moisture break. To reduce the potential for future moisture infiltration through the slab and subsequent damage to floor coverings, an impervious moisture barrier is recommended on top of the gravel cushion fill layer. Flexible floor coverings, such as carpet or sheet vinyl, should be considered because they can better mask minor slab cracking.

Where the slabs will be subjected to equipment vibration and/or vehicular traffic, we recommend placing the floor slab over 6 inches of aggregate subbase in lieu of the minimum 4-inch thick layer of open-graded gravel cushion fill. The aggregate subbase should consist of crushed basaltic aggregates compacted to a minimum of 95 percent relative compaction. The impervious moisture barrier may be omitted for these slabs.

We understand that exterior concrete walkway paths will be required for the proposed project. The concrete walkways may be supported on the on-site granular materials or select granular fill materials compacted to at least 90 percent relative compaction. Control joints should be provided at intervals equal to the width of the walkways with expansion joints at right-angle intersections.

It should be emphasized that the areas adjacent to the slab edges should be backfilled tightly against the edges of the slabs with relatively impervious soils. These areas should also be graded to divert water away from the slabs and to reduce the potential for water ponding around the slabs.

3.3 Drainage

Finished grades outside the new buildings should be sloped to shed water away from the slabs and foundations and to reduce the potential for ponding around the structure. It is also advised to install gutter systems around the building and divert the discharge away from the slab and foundation areas. Excessive landscape watering near the slabs and foundations also should be avoided. Planters next to foundations should be avoided or have concrete bottoms and drains to reduce the potential for excessive water infiltration into the subsurface.

These drainage requirements are essential for the proper performance of the above foundation recommendations because ponded water could cause subsurface soil saturation and loss of strength. The foundation excavations should be properly backfilled against the walls or slab edges immediately after setting of the concrete to reduce the potential for excessive water infiltration into the subsurface. Drainage swales should be provided as soon as possible and should be maintained to drain surface water runoff away from the slabs and foundations.

3.4 Retaining Structures

We understand retaining structures may be required for the project. In general, retaining structures should be designed to resist the lateral earth pressures due to the adjacent soils and surcharge effects.

Design of foundations for retaining structures may be based on the parameters presented in the following “Retaining Structure Foundations” subsection. Items of retaining structures that are addressed in the subsequent subsections include the following:

1. Retaining Structure Foundations
2. Static Lateral Earth Pressures
3. Dynamic Lateral Earth Pressures
4. Drainage

3.4.1 Retaining Structure Foundations

In general, we believe retaining wall foundations may be designed in accordance with the recommendations and parameters presented in the “Shallow Foundations” section herein. In addition, retaining wall foundations should be at least 18 inches wide and should be embedded a minimum of 24 inches below the lowest adjacent finished grades.

Foundations next to utility trenches or easements should be embedded below a 45-degree imaginary plane extending upward from the bottom edge of the utility trench, or they should extend to a depth as deep as the inverts of the utility lines. This requirement is necessary to avoid surcharging adjacent below-grade structures with additional structural loads and to reduce the potential for appreciable foundation settlement. For sloping ground conditions, the footing should extend deeper to obtain a minimum 6-foot setback distance measured horizontally from the outside edge of the footing to the face of the slope. Wall footings oriented parallel to the direction of the slope should be constructed in stepped footings.

3.4.2 Static Lateral Earth Pressures

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

LATERAL EARTH PRESSURES
FOR DESIGN OF RETAINING STRUCTURES

Backfill Condition	**Earth Pressure Component**	**Active** (pcf)	**At-Rest** (pcf)
Level Backfill	Horizontal	34	52
	Vertical	None	None
Maximum 2H:1V Sloping Backfill	Horizontal	47	65
	Vertical	12	16

The values provided in the table above assume the on-site granular soils with a maximum particle size of 3 inches or less or select granular fill materials will be used to backfill behind the retaining structures. The backfill behind retaining structures should be compacted to between 90 and 95 percent relative compaction. In general, an active condition may be used for gravity retaining walls or walls that are free to deflect by as much as 0.5 percent of the wall height. If the tops of walls are not free to deflect beyond this degree or are restrained, the walls should be designed for the at-rest condition. These lateral earth pressures do not include hydrostatic pressures that might be caused by groundwater trapped behind the walls.

Surcharge stresses due to areal surcharges, line loads, and point loads within a horizontal distance equal to the depth of the wall should be considered in the design. For uniform surcharge stresses imposed on the loaded side of the wall, a rectangular distribution with a uniform pressure equal to 29 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 46 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.

3.4.3 Dynamic Lateral Earth Forces

Dynamic lateral earth forces due to seismic loading will need to be considered in the design of retaining structures. For design in accordance with the 2006 IBC, the force due to dynamic lateral earth pressures associated with seismic loading (PGA = 0.289g) may be estimated using $6.0H^2$ pounds per linear foot of wall length for level backfill conditions, where H is the height of the wall in feet. It should be noted that the dynamic lateral earth forces provided assume that the wall will be allowed to move laterally by up to about 1 to 2 inches in the event of an earthquake.

If the estimated amount of lateral movement is not attainable or the retaining structure is restrained, the retaining structure should be designed with higher dynamic lateral forces for a restrained condition. For a restrained condition (less than 0.5 inches of lateral movement), dynamic lateral forces due to seismic loading may be estimated using $11.9H^2$ pounds per linear foot of wall (H measured in feet) for level backfill conditions.

The resultant force should be assumed to act through the mid-height of the wall. The above dynamic lateral earth forces are in addition to the static lateral earth pressures provided previously. An appropriately reduced factor of safety may be used when dynamic lateral earth pressures are accounted for in the design of the retaining structure.

3.4.4 Drainage

Retaining structures (above the groundwater) should be well-drained to reduce the potential for build-up of hydrostatic pressures. A typical drainage system for site retaining walls would consist of a 12-inch wide zone of permeable material, such as No. 3B Fine gravel (ASTM C33, No. 67 gradation), placed directly around a perforated pipe (perforations facing down) at the base of the wall discharging to an appropriate outlet or weepholes. As an alternative, a prefabricated drainage product, such as MiraDrain or EnkaDrain, may be used instead of the drainage material. The prefabricated drainage product also should be hydraulically connected to a perforated pipe at the base of the wall. The weepholes should be spaced no more than 6 feet apart.

The backfill from the bottom of the wall to the bottom of the weephole should consist of relatively impervious material to reduce the potential for significant water infiltration into the subsurface. In addition, the upper 12 inches of the retaining wall backfill should consist of relatively impervious material to reduce the potential for significant water infiltration behind the retaining structure unless covered by concrete slabs at the surface.

3.5 Site Grading

Based on the information provided, we envision minimal grading work will be required for the project construction. The following site grading items are addressed in the succeeding subsections:

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

A Geolabs representative should monitor the 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 field exploration.

3.5.1 Site Preparation

At the on-set of earthwork, areas within the contract grading limits should be cleared and grubbed thoroughly. Vegetation, debris, deleterious materials, existing pavements and structures to be demolished, and other unsuitable materials, should be removed and disposed of properly off-site to reduce the potential for contaminating the excavated materials.

Soft and yielding areas encountered during clearing and grubbing below areas designated to receive fill and/or future improvements should be over-excavated to expose firm natural material, and the resulting excavation should be backfilled with well-compacted fill. The excavated soft soils should not be reused as fill materials and should be properly disposed of off-site or used in landscape areas, if appropriate.

In general, the over-excavated subgrades should be scarified to a depth of about 8 inches, moisture-conditioned to above the optimum moisture content, and recompacted to a minimum of 90 percent relative compaction. The compaction requirement of the finished subgrades for areas subjected to vehicular traffic should be increased to a minimum of 95 percent relative compaction. Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density as determined by ASTM D1557. Optimum moisture is the water content (percentage by dry weight) corresponding to the maximum dry density.

3.5.2 Fills and Backfills

In general, the near-surface sandy soils encountered during our field exploration should be suitable for use as general fill and backfill materials, provided that the maximum particle size is less than 3 inches in largest dimension. The on-site materials generated from the excavations may be used as a source of general fill or backfill materials provided that they are free of vegetation and deleterious materials and screened of the over-sized materials and/or processed to meet the above gradation requirements (less than 3 inches in largest dimension).

Imported materials required for the project should consist of non-expansive, select granular material, such as crushed coral or basalt. The select granular fill should be well-graded from coarse to fine with particles no larger than 3 inches in largest dimension. The material should have a California Bearing Ratio (CBR) value of 20 or higher, and a swell potential of 1 percent or less when tested in accordance with ASTM D1883. The material also should contain between 10 and 30 percent particles passing the No. 200 sieve. Imported fill materials should be tested for conformance with these recommendations prior to delivery to the project site for the intended use.

Where groundwater is encountered (within the excavations), backfill materials should consist of free-draining granular materials, such as No. 3B Fine gravel (ASTM C33, No. 67 gradation), wrapped on all sides with non-woven filter fabric. 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 fill materials.

Aggregate base and subbase courses required for the project should consist of crushed basaltic aggregates and should meet the requirements of Sections 31 and 30 of the County of Maui Standard Specifications (September 1986), respectively. Imported fill materials should be tested for conformance with these recommendations prior to delivery to the project site for the intended use.

3.5.3 Fill Placement and Compaction Requirements

In general, fills and backfills should be moisture-conditioned to above the optimum moisture content, placed in level lifts not exceeding 8 inches in loose thickness, and compacted to at least 90 percent relative compaction. The compaction requirement for the last lift of fill (finished subgrade) below areas subjected to vehicular traffic should be increased 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. Compaction should be accomplished by sheepsfoot rollers, vibratory rollers, or other types of acceptable compaction equipment. Water tamping, jetting, or ponding should not be allowed to compact the fills.

3.5.4 Excavation

In general, we anticipate relatively shallow excavations may be required for the project. Where deep excavations (greater than 5 feet in depth) are planned, temporary shoring or sloping and benching should be implemented for the trench excavations.

In general, the contractor should determine the method and equipment to be used for excavations, subject to practical limits and safety considerations. Our field exploration generally encountered loose to medium dense sandy soils in the upper 10 feet of subsoils. Therefore, we envision that conventional excavation techniques using a backhoe excavator may be used for the planned excavations.

We encountered groundwater in our borings at depths of about 5.3 and 5.7 feet below the ground surface at the time of our field exploration. The groundwater levels encountered generally correspond to about Elevations +3.2 and +3.1 feet MSL, respectively. Dewatering of excavations will be necessary where the existing groundwater level is above the bottom of the proposed excavation.

3.6 Pavement Design

We understand new pavements for the driveways and parking areas will be constructed for the transit hub project. In general, we anticipate the vehicle loading for the new pavements would consist of primarily buses, passenger vehicles, and light trucks with some occasional refuse trucks.

We understand that both a flexible pavement section consisting of Asphaltic Concrete (AC) and a rigid pavement section consisting of Portland Cement Concrete Pavement (PCC) will be considered for the project. We have assumed the new flexible and rigid pavements will require a design life of 20 and 40 years, respectively. Traffic assumptions were determined based on the information provided and are summarized in the table below.

DESIGN TRAFFIC PARAMETERS FLEXIBLE PAVEMENT	
Design Period	20 Years
Average Daily Traffic (ADT)	Vehicles per day per direction
Year 2020	440
Year 2040	440
24-Hour Truck Traffic	96.4%
Type of Axle	Truck Traffic Distribution
2-axle	100.00%
3-axle	0.00%
4-axle	0.00%
5-axle	0.00%
6-axle	0.00%
7-axle	0.00%
Total ESAL in 20 Years	5,062,369

DESIGN TRAFFIC PARAMETERS RIGID PAVEMENT	
Design Period	40 Years
Average Daily Traffic (ADT)	Vehicles per day per direction
Year 2020	440
Year 2060	440
24-Hour Truck Traffic	96.4%
Type of Axle	Truck Traffic Distribution
2-axle	100.00%
3-axle	0.00%
4-axle	0.00%
5-axle	0.00%
6-axle	0.00%
7-axle	0.00%
Total ESAL in 40 Years	10,124,738

We performed the pavement design analyses with the assumption that the pavement subgrade soils generally will be similar to the medium dense sandy soils and stiff sandy silts encountered during our field exploration or select granular fill placed to achieve the design finished grades. Therefore, an CBR value of 11 has been adopted for the subgrade materials in our pavement design analyses. Based on the above, we recommend utilizing the following pavement design sections for this project.

Flexible Pavement for Employee Parking Areas

2.0-Inch Asphaltic Concrete
6.0-Inch Aggregate Base Course (95 Percent Relative Compaction)
8.0-Inch Minimum Total Pavement Thickness on a Moist Compacted Subgrade

Flexible Pavement for Driveways

2.5-Inch Asphaltic Concrete
4.0-Inch Asphaltic Concrete Base (92 Percent Max Theoretical Specific Gravity)
6.0-Inch Aggregate Base Course (95 Percent Relative Compaction)
12.5-Inch Minimum Total Pavement Thickness on a Moist Compacted Subgrade

We anticipate rigid pavements may be used in lieu of flexible pavements at the project. The pavement section being evaluated assumes a Portland cement concrete minimum flexural strength of 650 pounds per square inch (psi) at 28 days in accordance with ASTM C78. Based on the above, we recommend utilizing the following rigid pavement section for the project:

Rigid Pavement

9.0-Inch Portland Cement Concrete (Minimum 650 psi flexural strength)
6.0-Inch Aggregate Subbase (95 Percent Relative Compaction)
15.0-Inch Total Pavement Thickness on a Moist Compacted Subgrade

The subgrade soils under the pavement areas should be scarified to a minimum depth of 8 inches, moisture-conditioned to above the optimum moisture, and compacted to at least 95 percent relative compaction.

A Geolabs representative should monitor the pavement subgrade preparation 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 field exploration. California Bearing Ratio (CBR) tests and/or field observations should be performed on the actual subgrade soils during construction to confirm that the above design sections are adequate.

Aggregate base and subbase courses required for the project should consist of crushed basaltic aggregates and should meet the requirements of Sections 31 and 30 of the County of Maui Standard Specifications (September 1986), respectively. The materials should be moisture-conditioned to above optimum moisture content, placed in 8-inch level loose lifts, and compacted to a minimum of 95 percent relative compaction.

3.6.1 Rigid Pavement Joints

Considering the large areal extent and rigidity of the concrete pavement for the project, an adequate amount of pavement joints with proper design pattern should be provided to reduce the potential for distresses to the pavement structures. In general, joints should be provided in concrete pavements for the following reasons:

1. To reduce potential curling and warping stresses in the pavement caused by temperature and moisture gradients across the pavement slabs.
2. To reduce and control cracking due to volume changes in the concrete.
3. To reduce damage to immovable structures.
4. To facilitate construction.

3.6.2 Pavement Drainage

One of the primary distress mechanisms in pavement structures is pumping due to saturation of the subbase and/or subgrade soils. Therefore, the pavement surface should be sloped and drainage gradients maintained to carry surface water off the pavement to appropriate drainage structures. Surface water ponding should not be allowed on the site during or after construction.

Where concrete curbs are used to isolate landscaping in or adjacent to the pavement areas, we recommend extending the curbs a minimum of 2 inches below the aggregate subbase layer to reduce the potential for migration of excessive landscape water into the pavement section. We strongly recommend a development of good shoulder drainage, to reduce the potential for pavement deterioration or premature failure of the pavements.

3.7 Underground Utility Lines

We envision new on-site utility lines (i.e., water, sewer, and drain lines) and utility line connections adjacent to the project site may be required for the development. We anticipate most of the utility line trenches will be excavated in the sandy soils encountered in our field exploration. In general, we recommend using granular bedding consisting of 6 inches of free-draining granular materials (ASTM C33, No. 67 gradation) below the pipes for uniform support. Free-draining granular materials, such as No. 3B Fine gravel (ASTM C33, No. 67 gradation), also should be used for the initial trench backfill up to about 12 inches above the pipes.

It is critical to use this free-draining material to reduce the potential for formation of voids below the haunches of the pipes and to provide adequate support for the sides of the pipes. Improper backfill material around the pipes and improper placement of the backfill could result in backfill settlement and pipe damage. Where groundwater is encountered, the bedding should be wrapped on all sides by non-woven filter fabric (Mirafi 180N or equivalent).

Where soft and/or loose soils are encountered at or near the invert elevations of the on-site utility lines planned, we recommend providing a subgrade stabilization layer consisting of 24 inches of No. 2 Rock (ASTM C33, No. 4 gradation) wrapped in a non-woven filter fabric (Mirafi 180N or equivalent) below the bedding layer for uniform support. The stabilization layer should extend beyond the sides of the pipe a minimum width of one-fourth the outside diameter of the pipe or 12 inches, whichever is greater. A typical trench detail is provided on Plate 3.

The upper portion of the trench backfill from a level of 12 inches above the pipes to the top of the subgrade or finished grade may consist of the excavated granular materials with a maximum particle size of 6 inches or select granular fill materials. 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 at least 90 percent relative compaction. In areas where trenches will be in paved areas, the upper 3 feet of the trench backfill below the pavement finished grade should be compacted to no less than 95 percent relative compaction.

3.8 Design Review

Final drawings and specifications for the proposed construction should be forwarded to Geolabs for review and written comments prior to bid solicitation and/or construction. This review is necessary to evaluate conformance of the plans and specifications with 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 herein.

3.9 Post-Design Services/Services During Construction

It is highly recommended to retain Geolabs for geotechnical engineering support and continued services during construction. The following are critical items of construction monitoring that require "Special Inspections" as stipulated in Section 1704 of IBC 2006.

- Observation of shallow foundation excavations
- Observation of the subgrade soil preparation
- Observation of fill placement and compaction

A Geolabs representative should monitor the other aspects of the earthwork construction. 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 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 the field borings. Variations of the subsurface conditions between and beyond the field borings may occur, and the nature and extent of these variations may not become evident until construction is underway. If variations then appear evident, it will be necessary to re-evaluate the recommendations presented herein.

The field boring locations indicated in this report are approximate, having been estimated using a hand-held Garmin™ eTrex Vista HCx. Elevations of the borings were interpolated based on the spot elevations shown on the Grading and Drainage plan dated February 19, 2019 by Fukumoto Engineering, Inc. The locations and elevations of the field borings should be considered accurate only to the degree implied by the methods used.

The stratification breaks shown on the graphic representations of the borings depict 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. 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 tides, rainfall, perched groundwater conditions, groundwater withdrawal, and other factors.

This report has been prepared for the exclusive use of Fukumoto Engineering, Inc. and their project consultants for specific application to the *Central Maui Transit Hub* project in Kahului on the Island of Maui as described herein 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 architects and engineers in the preparation of the design documents for the new transit hub 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 subsurface conditions are commonly encountered. Unforeseen subsurface 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 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

Site Plan .... Plate 2

Typical Trench Detail .... Plate 3

Field Exploration .... Appendix A

Laboratory Tests .... Appendix B

-ΩΩΩΩΩΩΩΩΩ-

Respectfully submitted,

GEOLABS, INC.

By ______________________
Jason Seidman, P.E.
Project Engineer

By ______________________
Gerald Y. Seki, P.E.
Vice President

GS:JS:as

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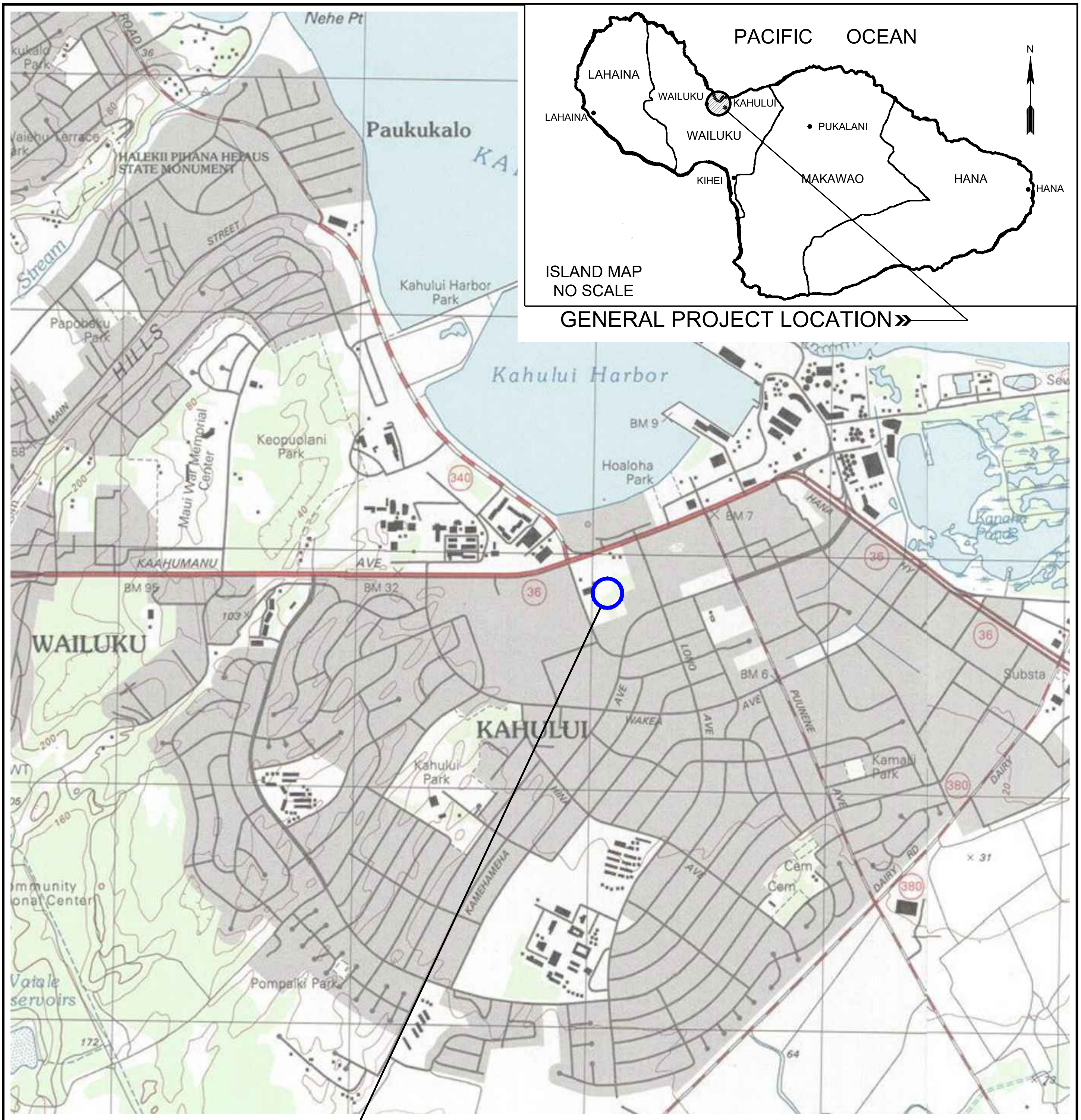
PLATES



PROJECT LOCATION »

2000 1000 0 2000 FT.

GRAPHIC SCALE

N

REFERENCE: MAP CREATED WITH TOPO!® ©2010 NATIONAL GEOGRAPHIC; ©2007 TELE ATLAS, REL. 1/2007.

PROJECT LOCATION MAP

CENTRAL MAUI TRANSIT HUB
KAHULUI, MAUI, HAWAII

GEOLABS, INC.

Geotechnical Engineering

DATE	DRAWN BY	PLATE
JULY 2019	HYC	
SCALE	W.O.	
1" = 2,000'	7798-00	1

CAD User: HENRY File Last Updated: July 01, 2019 5:04:21pm Plot Date: July 01, 2019 - 5:07:20pm
File: T:\Drafting\Working\7798-00_Central_Maui_Transit_Hub\7798-00PLM.dwg\PLM
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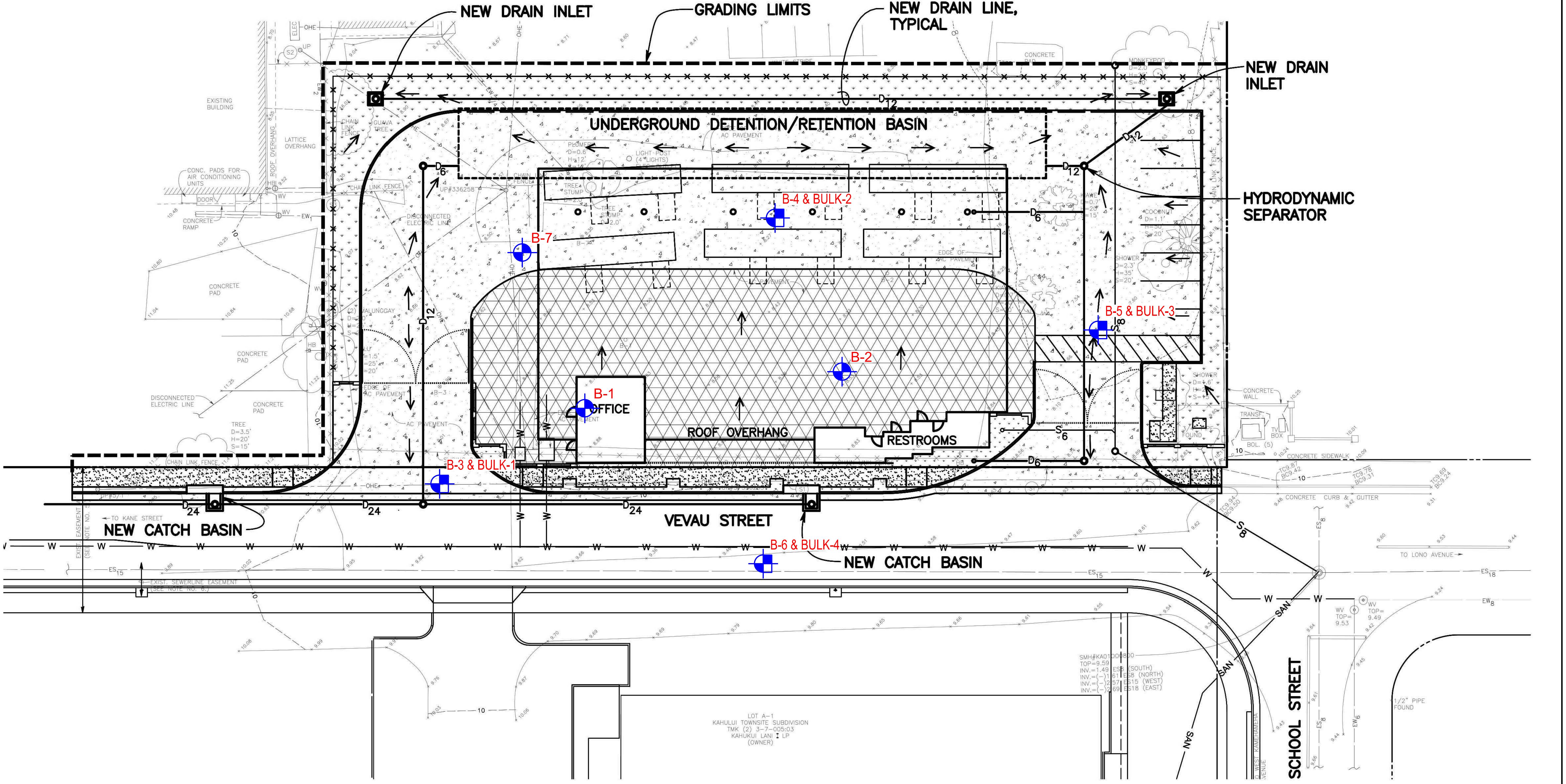

SITE PLAN

CENTRAL MAUI TRANSIT HUB
KAHULUI, MAUI, HAWAII

LEGEND:

⊕ APPROXIMATE BORING LOCATION

▣ APPROXIMATE BULK SAMPLE LOCATION

REFERENCE: PRELIMINARY GRADING AND DRAINAGE PLAN DATED FEBRUARY 19, 2019 PREPARED BY FUKUMOTO ENGINEERING, INC.

0 10 20 30 60 90 FT.
GRAPHIC SCALE

N

GEOLABS, INC.
Geotechnical Engineering

DATE	DRAWN BY	PLATE
JULY 2019	HYC	
SCALE	W.O.	2
1" = 30'	7798-00	

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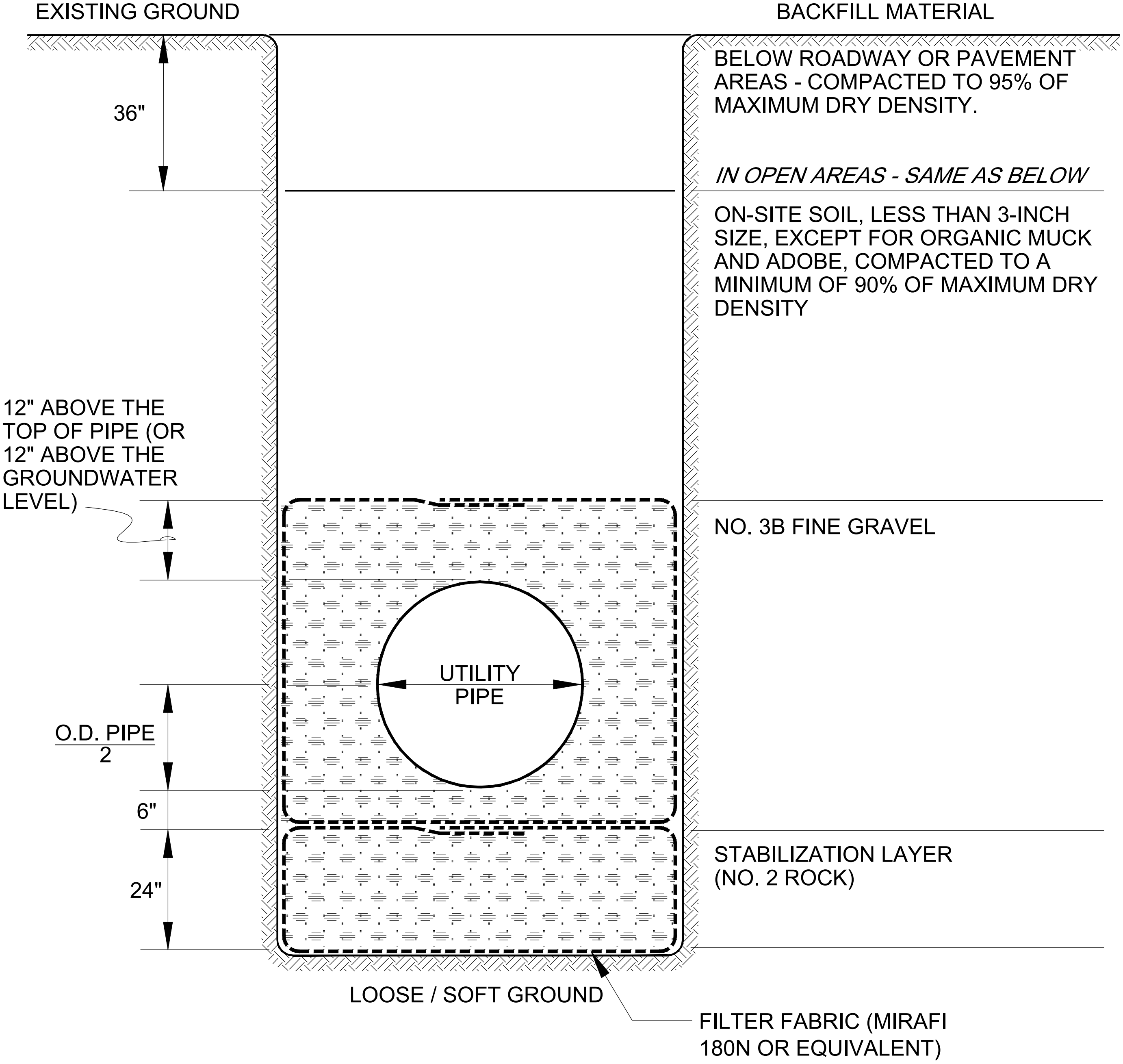

TYPICAL TRENCH DETAIL

CENTRAL MAUI TRANSIT HUB
KAHULUI, MAUI, HAWAII

GEOLABS, INC.
Geotechnical Engineering

DATE JULY 2019	DRAWN BY HYC	PLATE
SCALE NOT TO SCALE	W.O. 7798-00	3

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File: T:\Drafting\Working\7798-00_Central_Maui_Transit_Hub\7798-00TrenchDetail.dwg\Model

APPENDIX A

APPENDIX A

Field Exploration

We explored the subsurface conditions at the project site by drilling and sampling seven borings, designated as Boring Nos. 1 through 7, extending to depths of about 6.5 to 26.5 feet below the existing ground surface. The approximate boring locations are shown on the Site Plan, Plate 2. The borings were drilled using a truck-mounted drill rig equipped with continuous flight hollow stem 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. Graphic representations of the materials encountered are presented on the Logs of Borings, Plates A-1 through A-7.

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.

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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

◪	(2-INCH) O.D. STANDARD PENETRATION TEST
⧗	(3-INCH) O.D. MODIFIED CALIFORNIA SAMPLE
S	SHELBY TUBE SAMPLE
G	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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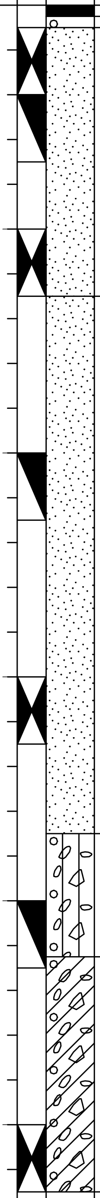
CENTRAL MAUI TRANSIT HUB
KAHULUI, MAUI, HAWAII

Log of Boring

1

Approximate Ground Surface Elevation (feet): 8.8 *

Laboratory			Field						
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)	Depth (feet)	USCS	Description
									3-inch **ASPHALTIC CONCRETE**
Sieve - #200 = 6.2%	7	106			33			GP	Dark grayish brown **SANDY GRAVEL**, medium dense, dry (fill)
	28				6			SP-SM	Tan **SAND (CORALLINE AND BASALTIC)** with a little silt, loose to medium dense, moist (dune sand)
	25	94			18	⊻	5		
								SP	Light grayish tan **SAND (CORALLINE AND BASALTIC)**, medium dense (dune sand)
	29				29		10		grades to multi-color mottling
	27	108			31		15		
								GM	Black and white **SILTY GRAVEL (CORALLINE AND BASALTIC)**, loose (beach deposit)
	35				8		20		
								GC	Dark orangish brown **CLAYEY GRAVEL (BASALTIC)**, loose to medium dense (alluvium)
	45	97			35		25		
									Boring terminated at 26.5 feet
							30		* Elevations estimated from Preliminary Grading and Drainage Plan dated February 19, 2019 prepared by Fukumoto Engineering, Inc.



Date Started:	November 7, 2018	Water Level: ⊻	5.7 ft. 11/07/2018 1127 HRS
Date Completed:	November 7, 2018		
Logged By:	K. Gerstnecker	Drill Rig:	CME-75DG1
Total Depth:	26.5 feet	Drilling Method:	6" Hollow Stem Auger
Work Order:	7798-00	Driving Energy:	140 lb. wt., 30 in. drop

Plate

A - 1

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CENTRAL MAUI TRANSIT HUB
KAHULUI, MAUI, HAWAII

Log of Boring

2

Approximate Ground Surface Elevation (feet): 8.5 *

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
											3-inch **ASPHALTIC CONCRETE**
	6	104			20					GP	Dark brown **SANDY GRAVEL**, medium dense, dry (fill)
	18				13					SP-SM	Light orangish tan and dark brown **SAND (CORALLINE AND BASALTIC)** with a little silt, medium dense, moist (dune sand)
										SW-SM	Tan and black **GRAVELLY SAND** with a little silt and traces of shell fragments, medium dense, moist to wet (beach deposit)
Direct Shear	24	103			25		▽ 5				
Sieve - #200 = 5.1%	9				23		10				
	27	106			31		15				
											Boring terminated at 16.5 feet
							20				
							25				
							30				

Date Started:	November 8, 2018	Water Level: ▽	5.3 ft. 11/08/2018 0916 HRS	Plate
Date Completed:	November 8, 2018			
Logged By:	K. Gerstnecker	Drill Rig:	CME-75DG1	A - 2
Total Depth:	16.5 feet	Drilling Method:	6" Hollow Stem Auger	
Work Order:	7798-00	Driving Energy:	140 lb. wt., 30 in. drop	

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CENTRAL MAUI TRANSIT HUB
KAHULUI, MAUI, HAWAII

Log of Boring

3

Approximate Ground Surface Elevation (feet): 9.3 *

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
											3-inch **ASPHALTIC CONCRETE**
Sieve - #200 = 3.9%	6	114			20					GP	Dark brown **SANDY GRAVEL**, medium dense, dry (fill)
	30				4					SP	Tan **SAND (CORALLINE AND BASALTIC)** with some gravel, loose to medium dense, moist (dune sand)
	25	106			12		5				grades to brown and tan
											Boring terminated at 6.5 feet
							10				
							15				
							20				
							25				
							30				

Date Started:	November 7, 2018	Water Level: ⊻	Not Encountered
Date Completed:	November 7, 2018		
Logged By:	K. Gerstnecker	Drill Rig:	CME-75DG1
Total Depth:	6.5 feet	Drilling Method:	6" Hollow Stem Auger
Work Order:	7798-00	Driving Energy:	140 lb. wt., 30 in. drop

Plate

A - 3

Approximate Ground Surface Elevation (feet): 8.3 *

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
											3-inch **ASPHALTIC CONCRETE**
Sieve - #200 = 5.9%	7	108			20					GP	Dark brown **SANDY GRAVEL**, medium dense, dry (fill)
	4				25					SP-SM	Light tan **SAND (CORALLINE)** with some gravel and a little silt, medium dense, moist (dune sand)
	7	98			22		5				
											Boring terminated at 6.5 feet
							10				
							15				
							20				
							25				
							30				

Date Started:	November 8, 2018	Water Level: ⊻	Not Encountered
Date Completed:	November 8, 2018		
Logged By:	K. Gerstnecker	Drill Rig:	CME-75DG1
Total Depth:	6.5 feet	Drilling Method:	6" Hollow Stem Auger
Work Order:	7798-00	Driving Energy:	140 lb. wt., 30 in. drop

Plate

A - 4

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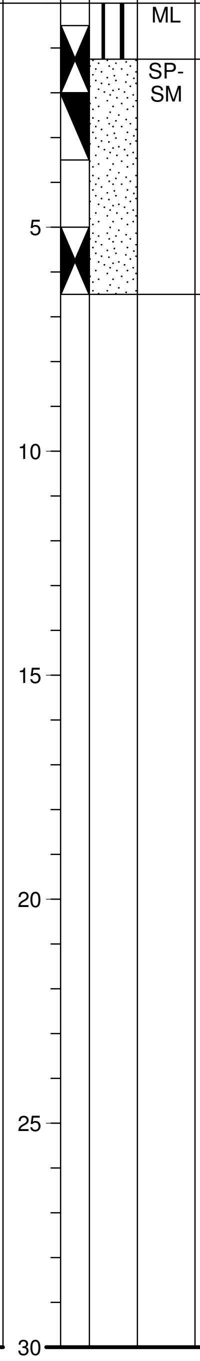
CENTRAL MAUI TRANSIT HUB
KAHULUI, MAUI, HAWAII

Log of Boring

5

Approximate Ground Surface Elevation (feet): 7.6 *

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
Sieve - #200 = 9.7%	7	88			19					ML	Dark brown with tan **SANDY SILT**, stiff, moist (fill)
	22				5					SP-SM	Tan and black **SAND (CORALLINE AND BASALTIC)** with a little silt, loose to medium dense, moist (dune sand)
Direct Shear	26	102			29		5				
											Boring terminated at 6.5 feet



Date Started:	November 8, 2018
Date Completed:	November 8, 2018
Logged By:	K. Gerstnecker
Total Depth:	6.5 feet
Work Order:	7798-00

Water Level: ⊻	Not Encountered
Drill Rig:	CME-75DG1
Drilling Method:	6" Hollow Stem Auger
Driving Energy:	140 lb. wt., 30 in. drop

Plate

A - 5

GEOLABS, INC. Geotechnical Engineering	CENTRAL MAUI TRANSIT HUB KAHULUI, MAUI, HAWAII	Log of Boring 6

Approximate Ground Surface Elevation (feet): 9.5 *

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
											6-inch **ASPHALTIC CONCRETE**
Sieve - #200 = 22.4%	16	100			20					ML	Dark brown and tan **SANDY SILT**, stiff, dry (fill)
	8				2					SM	Dark brown **SILTY SAND (BASALTIC)**, very loose, moist (dune sand)
Sieve - #200 = 2.9%	12	102			5		5			SP	Black and white **SAND (CORALLINE AND BASALTIC)**, very loose, moist (dune sand)
											Boring terminated at 6.5 feet
							10				
							15				
							20				
							25				
							30				

Date Started:	November 8, 2018	Water Level: ⊻	Not Encountered	Plate A - 6
Date Completed:	November 8, 2018			
Logged By:	K. Gerstnecker	Drill Rig:	CME-75DG1	
Total Depth:	6.5 feet	Drilling Method:	6" Hollow Stem Auger	
Work Order:	7798-00	Driving Energy:	140 lb. wt., 30 in. drop	

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CENTRAL MAUI TRANSIT HUB
KAHULUI, MAUI, HAWAII

Log of Boring

7

Approximate Ground Surface Elevation (feet): 8.4 *

Laboratory			Field						
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)	Depth (feet)	USCS	Description
									3-inch **ASPHALTIC CONCRETE**
Direct Shear	7	105			18			GP	Dark brown **SANDY GRAVEL**, medium dense, dry (fill)
								SP	Light tan and black **SAND (CORALLINE AND BASALTIC)**, medium dense, moist to wet (dune sand)
	18				15				
							5		
	-	-			32				
								GM	Tan and black **SILTY GRAVEL (CORALLINE AND BASALTIC)**, medium dense (beach deposit)
	25	103			26		10		
	9	124			31		15		
									Boring terminated at 16.5 feet
							20		
							25		
							30		

Date Started:	November 7, 2018	Water Level: ⊻	Hole Collapsed Prior to Reading	Plate
Date Completed:	November 7, 2018			
Logged By:	K. Gerstnecker	Drill Rig:	CME-75DG1	A - 7
Total Depth:	16.5 feet	Drilling Method:	6" Hollow Stem Auger	
Work Order:	7798-00	Driving Energy:	140 lb. wt., 30 in. drop	

APPENDIX B

APPENDIX B

Laboratory Tests

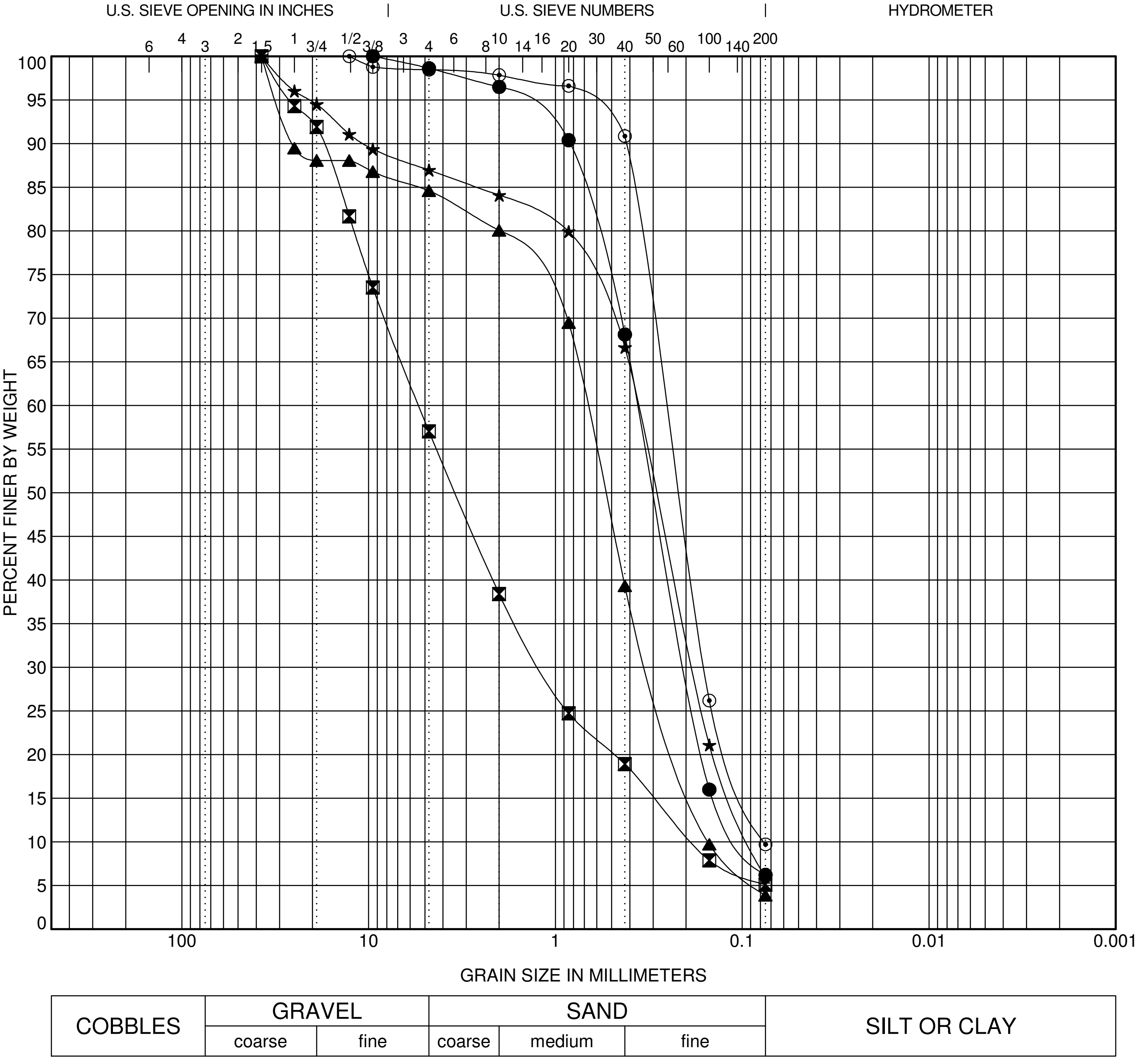
Moisture Content (ASTM D2216) and Unit Weight (ASTM D2937) determinations were performed on selected soil samples as an aid in the classification and evaluation of soil properties. The test results are presented on the Logs of Borings at the appropriate sample depths.

Seven Sieve Analysis tests (ASTM C117 & C136) 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 Plates B-1 and B-2.

Three Direct Shear tests (ASTM D3080) were performed on selected samples to evaluate the shear strength characteristics of the materials tested. Direct shear test results are presented on Plates B-3 through B-5.

Four laboratory California Bearing Ratio tests (ASTM D1883) were performed on bulk samples of the near-surface soils to evaluate the pavement support characteristics of the soils. The test results are presented on Plates B-6 through B-9.

One Modified Proctor compaction test (ASTM D1557, Method A) was performed on a bulk sample of the near-surface soils to evaluate the relationships between the moisture content and the dry density. The test results are presented on Plate B-10.



	Sample	Depth (ft)	Description	LL	PL	PI	Cc	Cu
●	B-1	0.5-2.0	Tan sand (SP-SM) with a little silt				1.1	3.7
⊠	B-2	10.0-11.5	Tan and black gravelly sand (SW-SM) with a little silt				1.4	29.3
▲	B-3	0.5-2.0	Tan sand (SP) with some gravel				0.9	4.5
★	B-4	0.5-2.0	Light tan sand (SP-SM) with some gravel and a little silt				1.0	4.0
⊙	B-5	0.5-2.0	Tan and black sand (SP-SM) with a little silt				1.3	3.4

	Sample	Depth (ft)	D100 (mm)	D60 (mm)	D30 (mm)	D10 (mm)	%Gravel	%Sand	%Fine
●	B-1	0.5-2.0	9.5	0.361	0.198	0.098	1.4	92.4	6.2
⊠	B-2	10.0-11.5	37.5	5.382	1.183	0.183	43.0	51.9	5.1
▲	B-3	0.5-2.0	37.5	0.684	0.306	0.152	15.4	80.7	3.9
★	B-4	0.5-2.0	37.5	0.365	0.184	0.09	13.0	81.0	5.9
⊙	B-5	0.5-2.0	12.7	0.259	0.16	0.076	1.5	88.8	9.7

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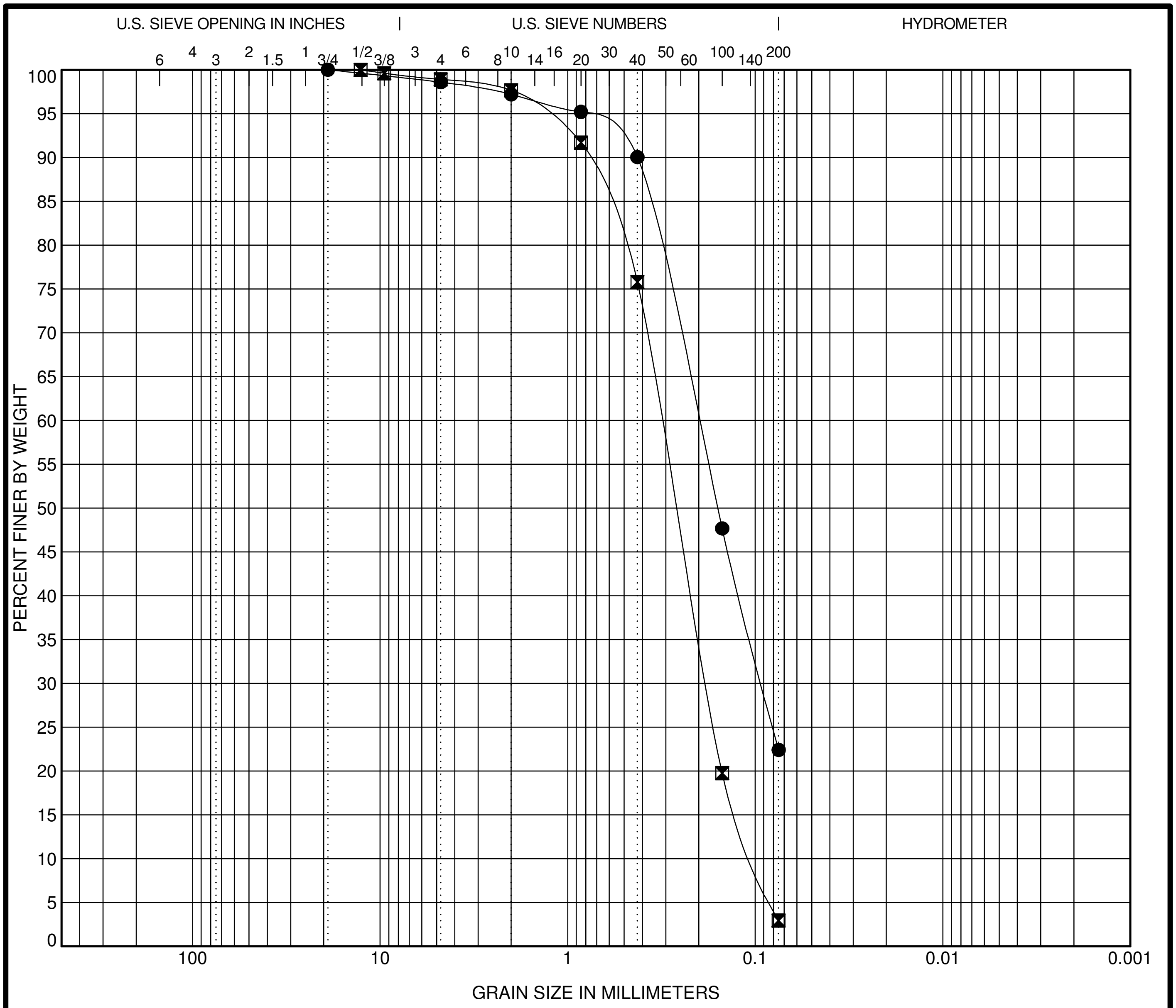
GRAIN SIZE DISTRIBUTION - ASTM C117 & C136

CENTRAL MAUI TRANSIT HUB
KAHULUI, MAUI, HAWAII

Plate
B - 1



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Sample	Depth (ft)	Description	LL	PL	PI	Cc	Cu
●	B-6	0.5-2.0	Dark brown silty sand (SM)					
☒	B-6	5.0-6.5	Black and white sand (SP)				1.0	3.2

	Sample	Depth (ft)	D100 (mm)	D60 (mm)	D30 (mm)	D10 (mm)	%Gravel	%Sand	%Fine
●	B-6	0.5-2.0	19	0.203	0.092		1.4	76.2	22.4
☒	B-6	5.0-6.5	12.7	0.317	0.182	0.1	1.1	96.0	2.9

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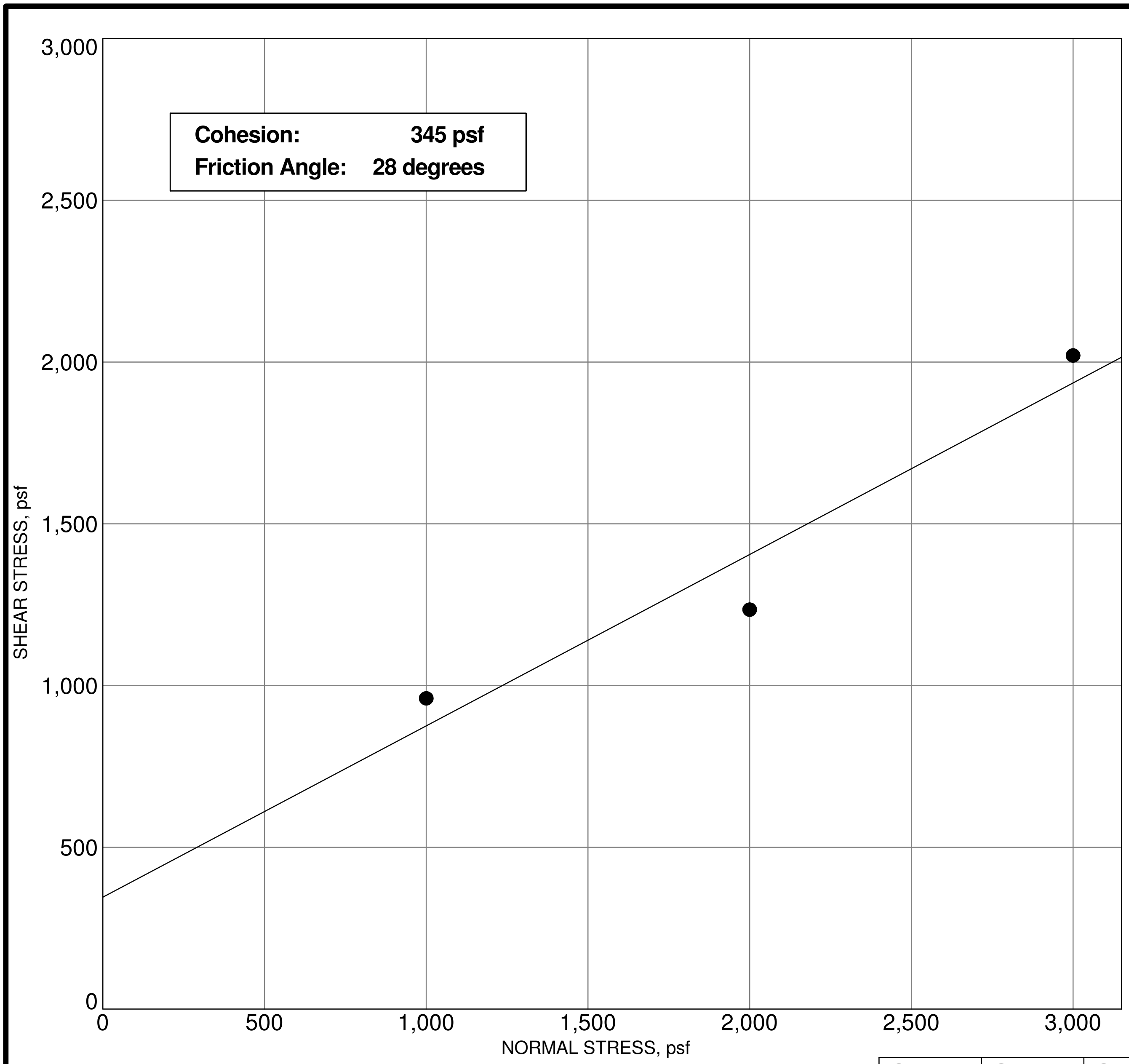
GRAIN SIZE DISTRIBUTION - ASTM C117 & C136

CENTRAL MAUI TRANSIT HUB
KAHULUI, MAUI, HAWAII

Plate
B - 2



		Sample #1	Sample #2	Sample #3
INITIAL	Moisture Content, %	29.1	26.4	28.9
	Dry Density, pcf	95.0	96.7	94.7
	Height, inches	1.00	1.00	1.00
FINAL	Moisture Content, %	29.8	27.7	27.4
	Dry Density, pcf	93.9	97.8	95.1
	Height, inches	1.011	0.989	0.996
	Diameter, inches	2.42	2.42	2.42
	Deformation Rate, inch/minute	0.0025	0.0022	0.0021
	Normal Stress, psf	**1000**	**2000**	**3000**
	Peak Shear Stress, psf	**960**	**1235**	**2020**
	Shear Displacement, inches	0.33	0.42	0.41

Sample: B-2
Depth: 5.0 - 6.5 feet
Description: Tan and black gravelly sand (SW-SM) with a little silt

G_DIRECT_SHEAR 7798-00.GPJ GEOLABS.GDT 7/2/19

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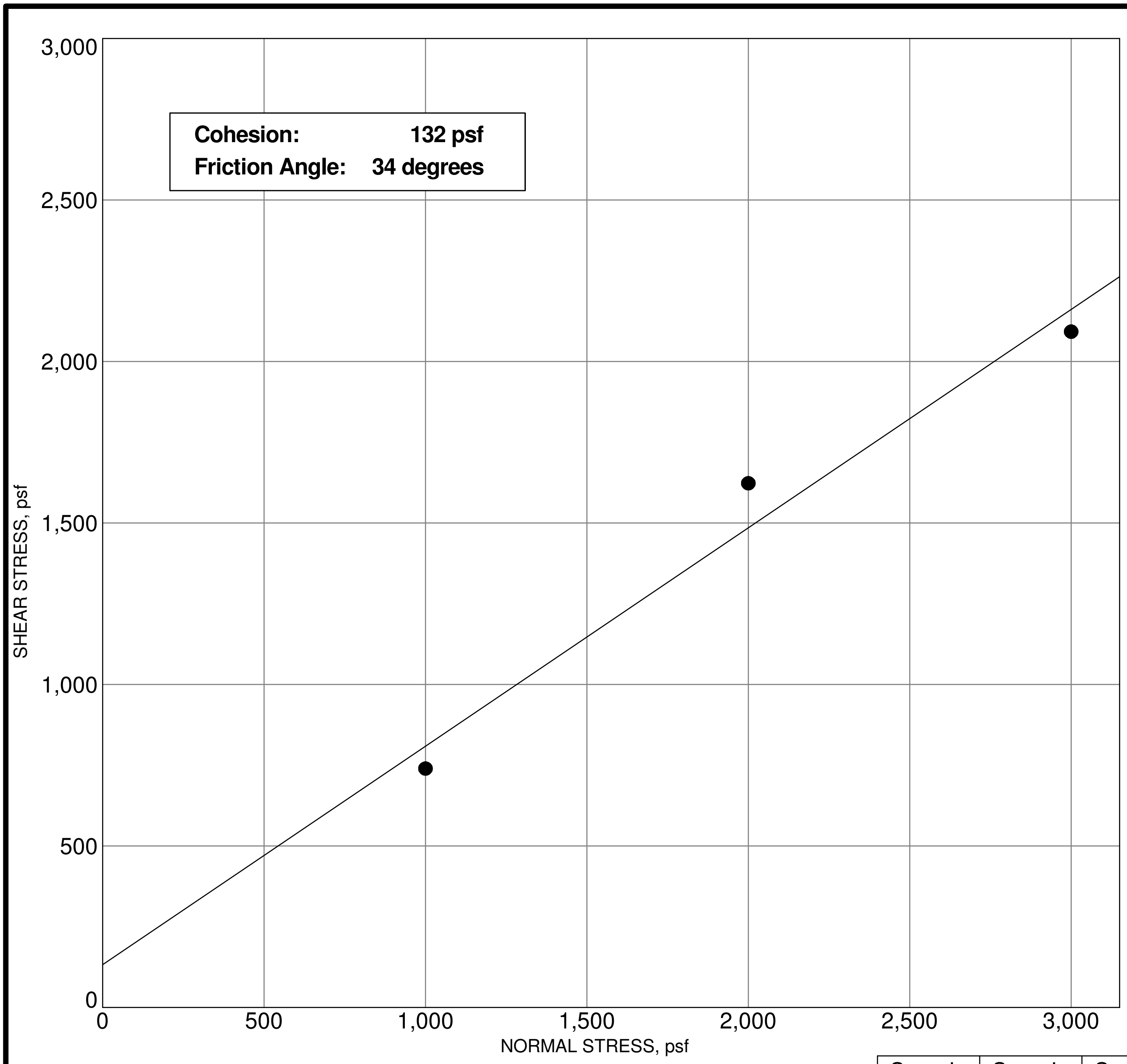
DIRECT SHEAR TEST - ASTM D3080

CENTRAL MAUI TRANSIT HUB
KAHULUI, MAUI, HAWAII

Plate
B - 3



		Sample #1	Sample #2	Sample #3
INITIAL	Moisture Content, %	25.8	37.2	44.0
	Dry Density, pcf	93.4	89.9	85.7
	Height, inches	1.00	1.00	1.00
FINAL	Moisture Content, %	29.5	31.5	31.4
	Dry Density, pcf	93.6	91.5	85.7
	Height, inches	0.998	0.983	1.000
	Diameter, inches	2.42	2.42	2.42
	Deformation Rate, inch/minute	0.0024	0.0020	0.0024
	Normal Stress, psf	**1000**	**2000**	**3000**
	Peak Shear Stress, psf	**740**	**1623**	**2092**
	Shear Displacement, inches	0.39	0.41	0.42

Sample: B-5
Depth: 5.0 - 6.5 feet
Description: Tan and black sand (SP-SM) with a little silt

G_DIRECT_SHEAR 7798-00.GPJ GEOLABS.GDT 7/17/19

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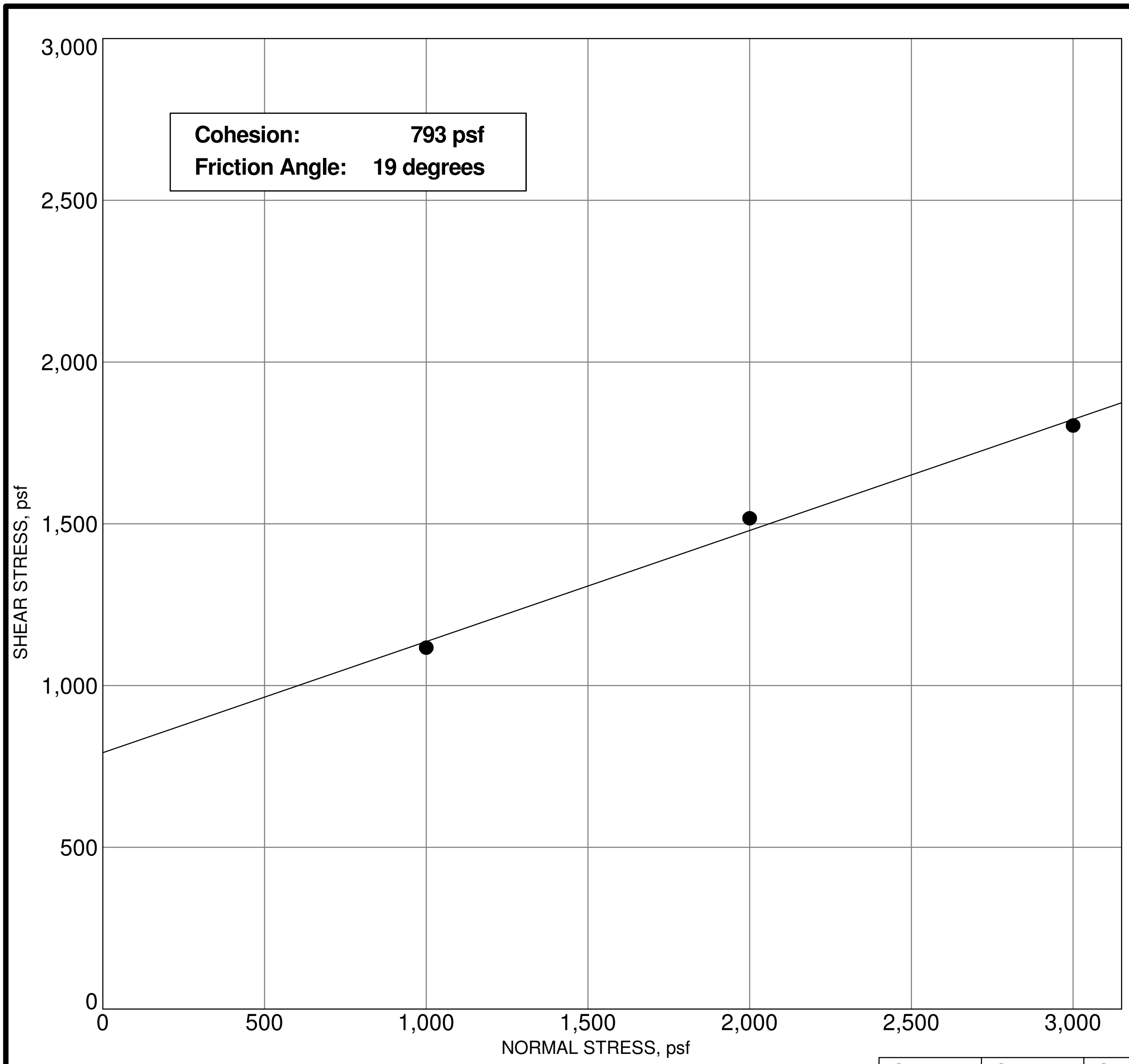
DIRECT SHEAR TEST - ASTM D3080

CENTRAL MAUI TRANSIT HUB
KAHULUI, MAUI, HAWAII

Plate
B - 4



		Sample #1	Sample #2	Sample #3
INITIAL	Moisture Content, %	37.8	24.1	28.9
	Dry Density, pcf	87.8	95.3	87.4
	Height, inches	1.00	1.00	1.00
FINAL	Moisture Content, %	30.5	25.6	32.6
	Dry Density, pcf	93.5	94.1	86.6
	Height, inches	0.939	1.013	1.009
	Diameter, inches	2.42	2.42	2.42
	Deformation Rate, inch/minute	0.0025	0.0022	0.0021
	Normal Stress, psf	**1000**	**2000**	**3000**
	Peak Shear Stress, psf	**1117**	**1517**	**1804**
	Shear Displacement, inches	0.43	0.42	0.41

Sample:	B-7
Depth:	0.5 - 2.0 feet
Description:	Light tan and black sand (SP)

G_DIRECT_SHEAR 7798-00.GPJ GEOLABS.GDT 7/2/19

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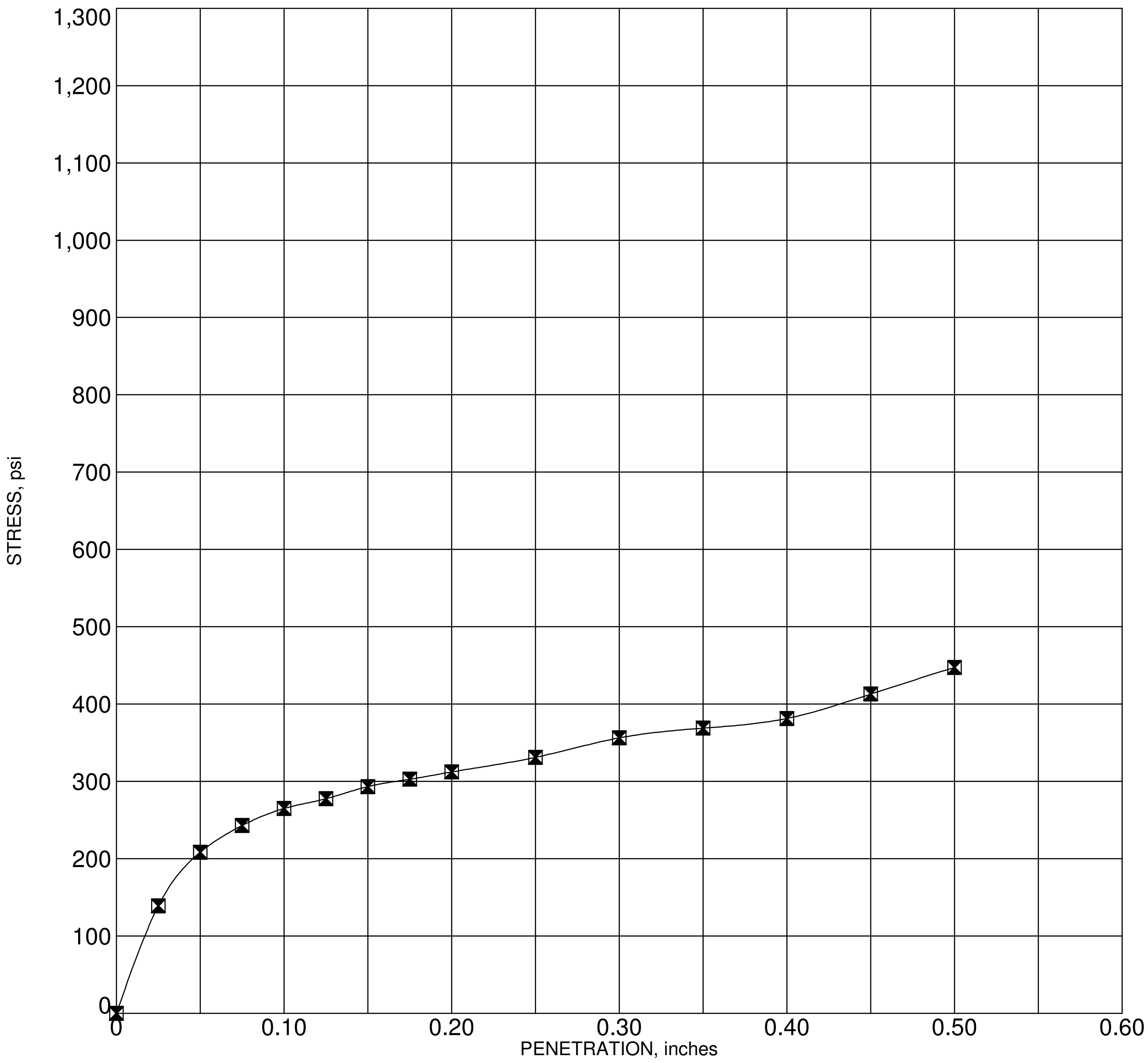
DIRECT SHEAR TEST - ASTM D3080

CENTRAL MAUI TRANSIT HUB
KAHULUI, MAUI, HAWAII

Plate
B - 5



Corr. CBR @ 0.1"	25.2
Corr. CBR @ 0.2"	20.2
Swell (%)	0.09

Sample: Bulk-1
Depth: 0.0 - 2.0 feet
Description: Tan sand (SP) with some gravel

Molding Dry Density (pcf)	123.8	Hammer Wt. (lbs)	10
Molding Moisture (%)	7.1	Hammer Drop (inches)	18
Days Soaked	4	No. of Blows	56
Aggregate	3/4 inch minus	No. of Layers	5

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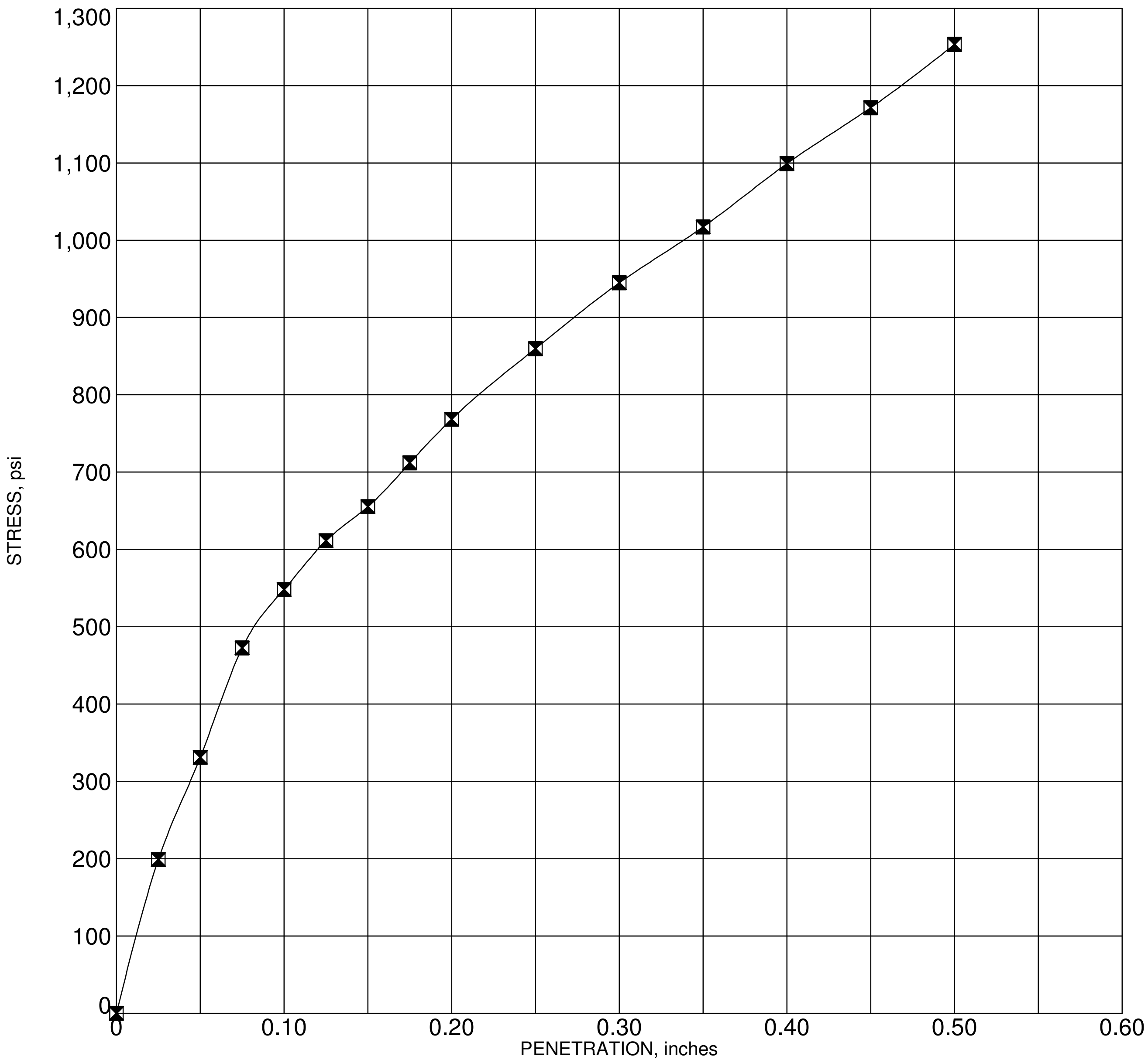
CALIFORNIA BEARING RATIO - ASTM D1883

CENTRAL MAUI TRANSIT HUB
KAHULUI, MAUI, HAWAII

Plate
B - 6



Corr. CBR @ 0.1"	54.8
Corr. CBR @ 0.2"	51.2
Swell (%)	0.00

Sample: Bulk-2
Depth: 0.0 - 2.0 feet
Description: Light tan sand (SP-SM) with some gravel and a little silt

Molding Dry Density (pcf)	129.2	Hammer Wt. (lbs)	10
Molding Moisture (%)	8.4	Hammer Drop (inches)	18
Days Soaked	4	No. of Blows	56
Aggregate	3/4 inch minus	No. of Layers	5

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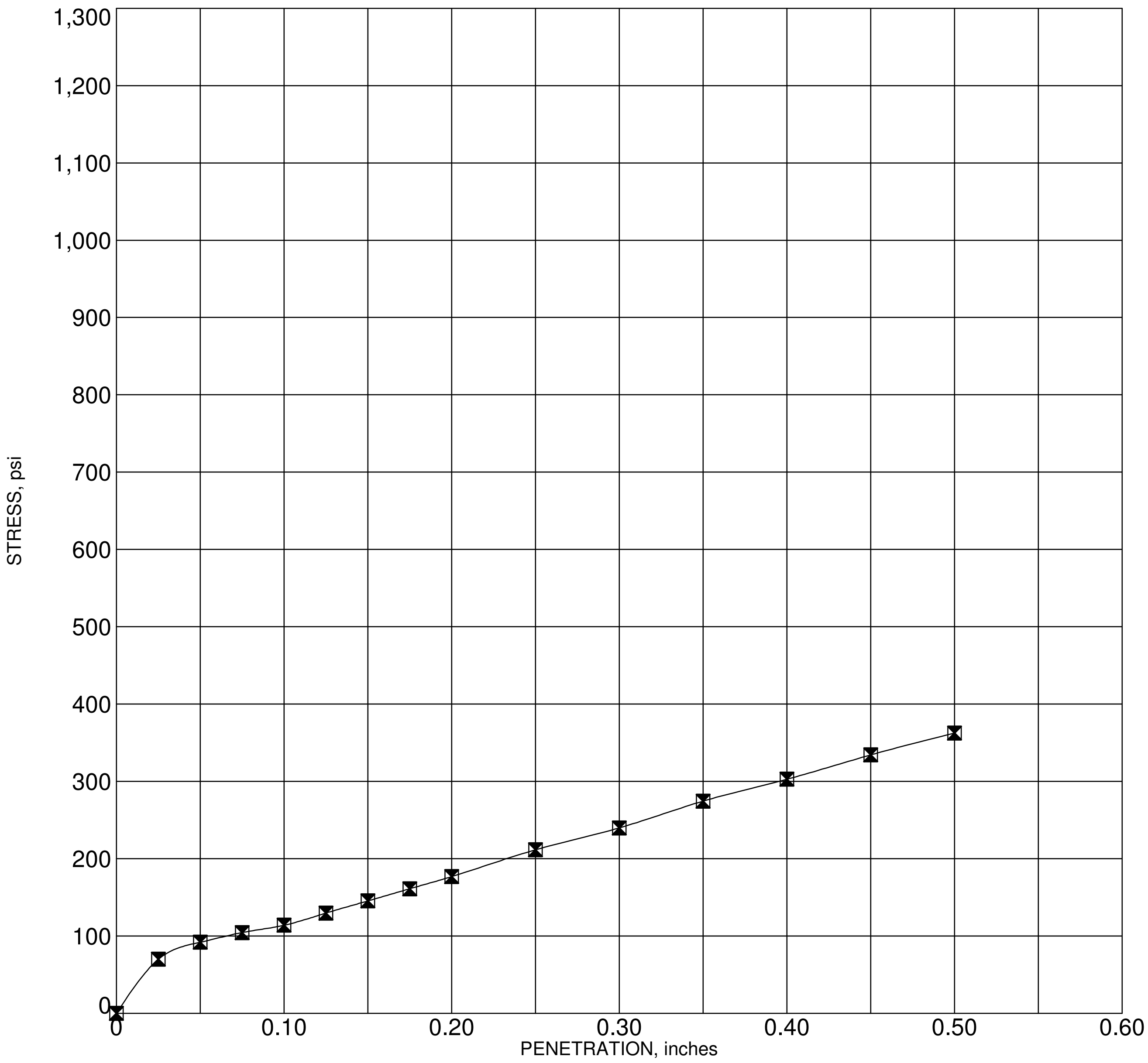
CALIFORNIA BEARING RATIO - ASTM D1883

CENTRAL MAUI TRANSIT HUB
KAHULUI, MAUI, HAWAII

Plate
B - 7

G_CBR 7798-00.GPJ GEOLABS.GDT 7/2/19



Corr. CBR @ 0.1"	11.4
Corr. CBR @ 0.2"	11.8
Swell (%)	0.02

Sample: Bulk-3
Depth: 0.0 - 2.0 feet
Description: Tan and black sand (SP-SM) with a little silt

Molding Dry Density (pcf)	113.5	Hammer Wt. (lbs)	10
Molding Moisture (%)	9.8	Hammer Drop (inches)	18
Days Soaked	4	No. of Blows	56
Aggregate	3/4 inch minus	No. of Layers	5

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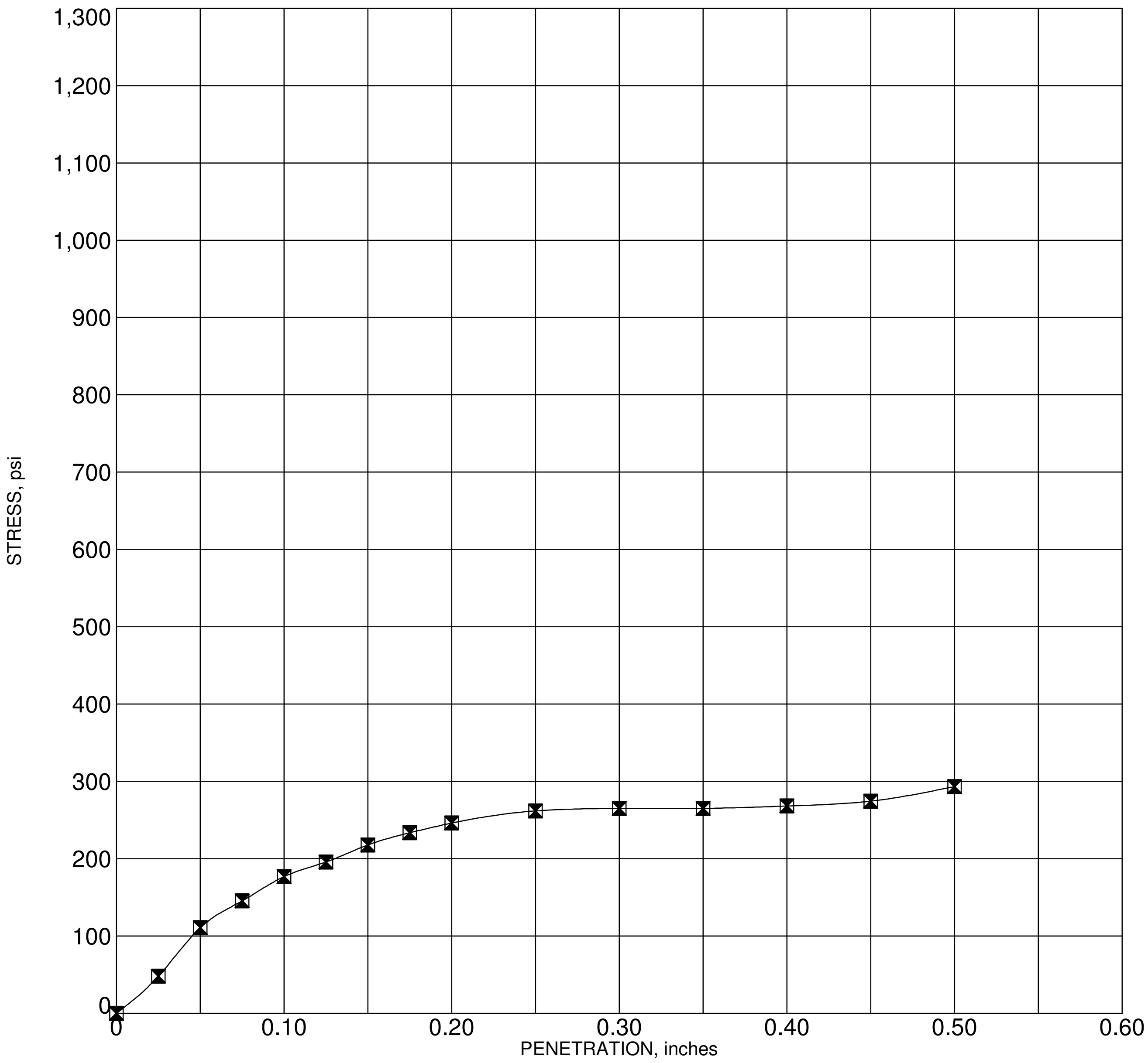
CALIFORNIA BEARING RATIO - ASTM D1883

CENTRAL MAUI TRANSIT HUB
KAHULUI, MAUI, HAWAII

Plate
B - 8



Corr. CBR @ 0.1"	17.7
Corr. CBR @ 0.2"	16.4
Swell (%)	0.94

Sample: Bulk-4
Depth: 0.0 - 2.0 feet
Description: Dark brown silty sand (SM)

Molding Dry Density (pcf)	102.9	Hammer Wt. (lbs)	10
Molding Moisture (%)	21.1	Hammer Drop (inches)	18
Days Soaked	4	No. of Blows	56
Aggregate	3/4 inch minus	No. of Layers	5

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CALIFORNIA BEARING RATIO - ASTM D1883

CENTRAL MAUI TRANSIT HUB
KAHULUI, MAUI, HAWAII

Plate
B - 9

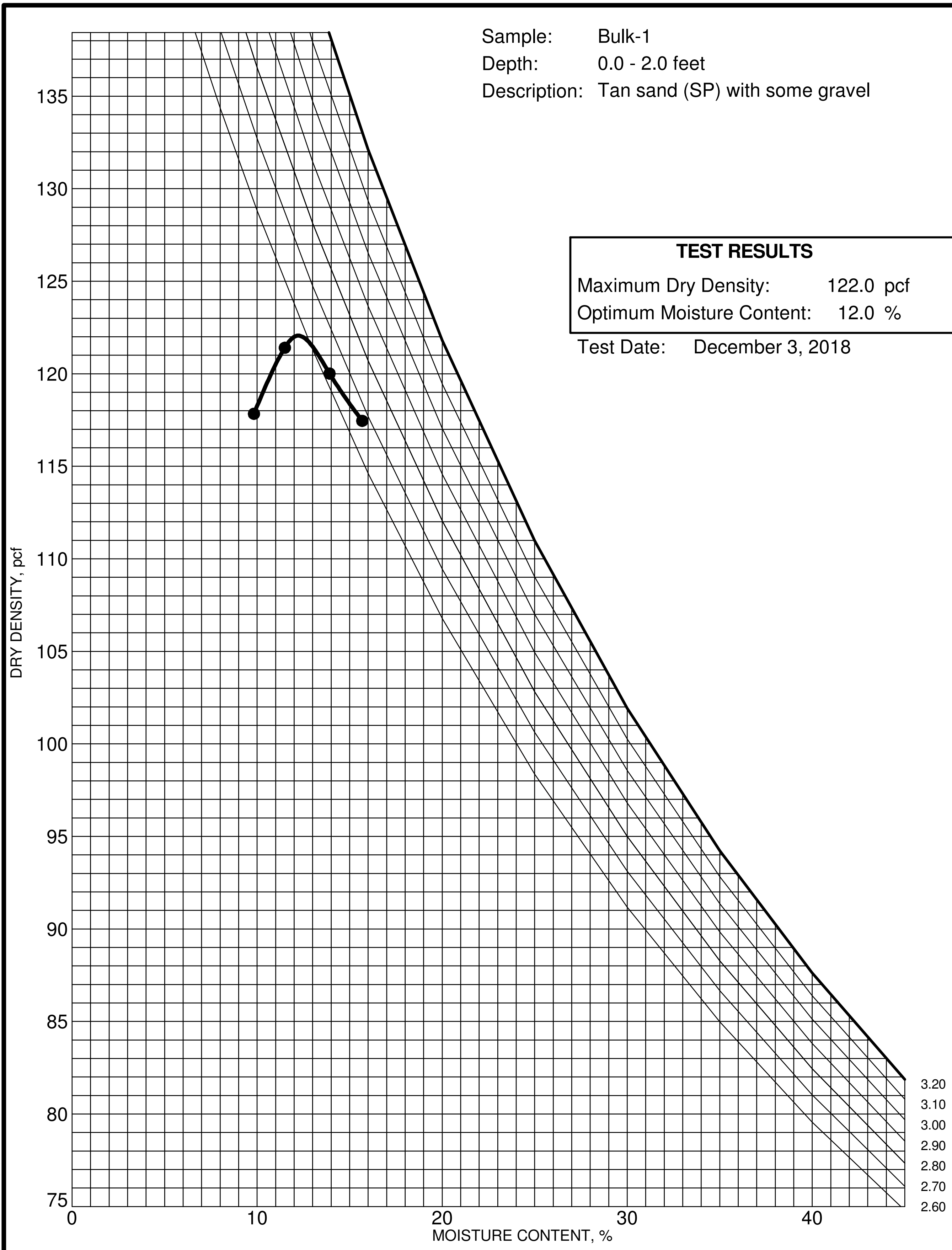
G_CBR 7798-00.GPJ GEOLABS.GDT 7/2/19

Sample: Bulk-1
Depth: 0.0 - 2.0 feet
Description: Tan sand (SP) with some gravel

TEST RESULTS

Maximum Dry Density: 122.0 pcf
Optimum Moisture Content: 12.0 %

Test Date: December 3, 2018



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MOISTURE-DENSITY RELATIONSHIP - ASTM D1557 A

CENTRAL MAUI TRANSIT HUB
KAHULUI, MAUI, HAWAII

Plate
B - 10

G_COMPACTION 7798-00.GPJ GEOLABS.GDT 7/2/19