IFB Job No. 02-14 Kapaa Homesteads 325' Tanks
Two 0.5 MG Tanks, Package B - Tanks Package
Appendix O: Geotechnical Engineering Exploration

GEOTECHNICAL ENGINEERING EXPLORATION
KAPAHI 1.0 MG STORAGE TANKS
WAIALUA-KAPAA WATER SYSTEM
KAPAA, KAUAI, HAWAII

SEPTEMBER 30, 2011

Prepared for

BELT COLLINS HAWAII LTD.

GEOLABS, INC.

Geotechnical Engineering and Drilling Services

W.O. 5988-00 & 10

GEOTECHNICAL ENGINEERING EXPLORATION
KAPAHI 1.0 MG STORAGE TANKS
WAIALUA-KAPAA WATER SYSTEM
KAPAA, KAUAI, HAWAII

W.O. 5988-00 & 10 SEPTEMBER 30, 2011

Prepared for

BELT COLLINS HAWAII LTD.

CLAYTON S. MIMURA
LICENSED PROFESSIONAL ENGINEER
No. 4176-C
HAWAII, U.S.A.

THIS WORK WAS PREPARED BY
ME OR UNDER MY SUPERVISION.

Clayton S. Mimura 4-30-12

SIGNATURE EXPIRATION DATE OF THE LICENSE

GEOLABS, INC.
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Hawaii • California

GEOLABS, INC.
Geotechnical Engineering and Drilling Services

September 30, 2011
W.O. 5988-00 & 10

Mr. Cary Kondo, P.E.
Belt Collins Hawaii Ltd.
2153 North King Street, Suite 200
Honolulu, HI 96819

Dear **Mr. Kondo:**

Geolabs, Inc. is pleased to submit our report entitled "Geotechnical Engineering Exploration, Kapahi 1.0 MG Storage Tanks, Waialua-Kapaa Water System, Kapaa, Kauai, Hawaii" prepared for the design of the water storage tanks project.

Our work was performed in general accordance with the scope of services outlined in our fee proposals of January 11, 2007 and October 26, 2010.

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

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

Very truly yours,

GEOLABS, INC.

Clayton S. Mimura

Clayton S. Mimura, P.E.
President

CSM:RP:as

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Hawaii • California

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KAPAA, KAUAI, HAWAII

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SUMMARY OF FINDINGS AND RECOMMENDATIONS

Our field exploration generally encountered medium soft to stiff residual soil underlain by medium stiff saprolite soil. The residual and saprolite soils generally contain clayey silts with some sand and gravel. The saprolite soil extends to the maximum depth explored of about 87 feet below the existing ground surface. We did not encounter groundwater in the borings at the time of our field exploration; however, the soil samples recovered exhibit high moisture content in general.

We understand that the new water tanks will be constructed near the existing grade. Based on the relatively soft soil conditions encountered at the project site, we explored various foundation options to support the future tanks including both shallow foundation system and deep foundations system.

A shallow foundation system for tanks situated near the existing grade was first evaluated. However, due to the heavy structural loads and compressible subsoils, excessive settlements on the order of 8 to 12 inches were calculated.

We evaluated the effect of removing up to 10 feet of soils below the tanks and replacing with structural fill to eliminate the settlements from this layer. However, due to the large size of the tanks (about 70 feet diameter), the zone of influence below the tanks is very large, therefore, significant settlements (on the order of 5 to 8 inches) will still occur from the soils below the 10-foot over-excavation level.

Another alternative evaluated consisted of lowering the tank finished grade to provide a buried tank. From a foundation design standpoint, this alternative would lessen the load on the subsoils by the weight of the soil removed, reducing the settlements. However, the calculated settlements still were excessive and the buried tanks would require costly retaining walls to maintain the lower grades around the tanks.

Other alternatives, including surcharging and ground stabilization/densification, were considered but did not appear cost effective and could adversely affect the adjacent neighboring properties.

Based on the above evaluations, a deep foundation system was recommended for the tank foundations. Two systems, drilled shafts and micropiles, are discussed in this report. Driven precast concrete piles were not considered due to possible adverse effects to adjacent properties from the pile driving vibrations, possible noise issued to the residential properties, and difficulties to transport precast pile sections to the site via the narrow and winding roads.

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

END OF SUMMARY OF FINDINGS AND RECOMMENDATIONS

SECTION 1. GENERAL

1.1 Introduction

This report presents the results of our geotechnical engineering exploration performed for the proposed Kapahi 1.0 MG Water storage tanks in the Kapaa area on the Island of Kauai, Hawaii. The project location and general vicinity are shown on the Project Location Map, Plate 1.

This report summarizes our findings and geotechnical engineering recommendations resulting from our field exploration, laboratory testing, and engineering analyses for the proposed storage tanks project. These recommendations are intended for the design of site grading, foundations, and pavements only. The findings and recommendations presented herein are subject to the limitations noted at the end of this report.

1.2 Project Considerations

The project site is at the south side of the Kapahi Road and Kawaihau Road intersection in the Kapaa area on the Island of Kauai, Hawaii. Based on the information provided, we understand that it is proposed to construct two new 0.5 MG water storage tanks for the Kapahi area at Elevation 306 to 307 feet Mean Sea Level (MSL). The new storage tanks will be north of the existing 0.2 MG Ornellas Tank at the project site on Kawaihau Road.

We understand the storage tanks will be reinforced concrete structures. A short access road and a service perimeter road are also planned for the project. In addition, the project includes the installation of a connecting pipeline to the Waialua-Kapaa Water System.

Based on the available site plan and the proposed finish floor elevation, we understand that the new storage tanks will be constructed near the existing grade at the proposed tank location. Therefore, we anticipate minimum fills/cuts, up to 1 foot, will be required to achieve the design grades.

1.3 Purpose and Scope

The purpose of our exploration was to obtain an overview of the surface and subsurface conditions to develop a generalized soil data set to formulate geotechnical engineering recommendations for the design of site grading, foundations, and pavements for the proposed *Kapahi 1.0 MG Storage Tanks* project. Our work was performed in general accordance with our fee proposals dated January 11, 2007 and October 26, 2010. The scope of work for this exploration included the following tasks and work efforts:

1. Mobilization and demobilization of a truck-mounted drill rig, and two operators from Honolulu to the project site and back.

2. Drilling and sampling of five borings to depths of about 26 to 87 feet below the existing ground surface. Also, collection of a bulk soil sample for analyses of the pavement support characteristics of the surface soils.

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

4. Laboratory testing of selected soil samples obtained from the field exploration as an aid in classifying the materials and evaluating their engineering properties.

5. Analyses of the field and laboratory data to formulate geotechnical engineering recommendations for the design of foundations, site grading, and pavements for the project.

6. Preparation of this report summarizing our work on the project and presenting our findings and recommendations.

7. Coordination of our overall work by our project engineer and principal engineer.

8. Quality assurance and client/design team consultation by our principal engineer.

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

10. Review of the project plans and specifications for general conformance with our recommendations presented herein.

Detailed descriptions of our field exploration and Logs of Borings are provided in Appendix A of this report. Results of the laboratory tests 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 renewed volcanic activity occurred 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.

The basalt rock of the Koloa Volcanic Series is generally characterized by thick lava flows of dense basaltic rock extruded from groups of vents aligned in north-south trends in various locales on the eastern half of the Island of Kauai. Associated with the Koloa Volcanic Series lava flows are some localized volcanic deposits consisting of pyroclastic materials (volcanic ash and cinders), which usually are encountered as surficial mantling deposits and accumulations surrounding the cinder cone vents. Rocks of the Koloa Volcanic Series cover most of the eastern half of the Island of Kauai, including the regional area surrounding the project site.

Based on a review of available geological mapping information, the project site appears to be inland on a gently sloping plateau region along the foot of the Makaleha Mountains as indicated on the Project Location Map, Plate 1. The project site is underlain by weathered soils and basaltic rock belonging to the Koloa Volcanic Series. Multiple meandering stream channels, which drain towards the ocean located easterly of the project site, incise the inland plateau region. In general, the near-surface soils consist of residual and saprolitic soils (completely weathered rock), derived from the deep in-situ weathering of the Koloa Volcanic Series igneous rocks. It grades to highly and moderately weathered basalt rock formation with increasing depth. Therefore, occasional hard basaltic rock boulders may be encountered embedded within the deeply weathered soils.

2.2 Site Description

The project site is near the intersection of Kapahi Road and Kawaihau Road in the Kapaa area on the eastern side of the Island of Kauai, Hawaii. The project site is an existing County of Kauai-Department of Water tank lot, which encompasses approximately 0.84 acres. The project location and general vicinity are shown on the Project Location Map, Plate 1.

Based on our field exploration, the current location for the new tanks is generally open and covered with grass. An existing square-shaped 0.2 MG Ornellas Tank is at the south side of the site. Based on the topographic map provided, the proposed new tank location gently slopes down towards the easterly direction. The existing ground surface elevations generally range between about +300 and +310 feet MSL. Details of the project site conditions are shown on the Site Plan, Plate 2.

2.3 Subsurface Conditions

Our field exploration consisted of drilling and sampling five borings, designated as Boring Nos. 1 through 5, at the proposed Kapahi 1.0 MG tanks site. The borings were drilled to depths of about 26 to 87 feet below the existing ground surface. We collected one bulk soil sample, designated as Bulk-1, for CBR analysis to assist with pavement design. The approximate boring and bulk soil sample collection point locations are also shown on the Site Plan, Plate 2.

Our field exploration generally encountered medium soft to stiff residual soils below the existing ground surface. The residual soils generally consist of clayey silt or silty clay with some sand and gravel. Medium stiff to stiff saprolite soils consisting of sandy silt and clayey silt with gravel were encountered below the residual soils. The saprolite soils grade from stiff to medium stiff at about 50 feet below the existing ground surface and extended to the maximum depth explored of about 87 feet. It should be noted that Boring No. 5 encountered a thin layer of basalt (~2.5 feet in thickness) at 82 feet below the ground surface. In general, the basalt is moderately fractured and weathered. It is common that hard/unweathered rock core exists within residual/saprolite soil. Therefore, it is not unusual to encounter basalt within the residual and saprolite soil at the project site.

We did not encounter groundwater in the borings during our field exploration; however, the soil samples recovered appeared to be fairly wet. It should be noted that groundwater levels may vary significantly depending on seasonal rainfall, time of year, and other factors.

Detailed descriptions of our field exploration methodology are presented in Appendix A of this report. Descriptions and graphic representation of the materials encountered in the borings are provided on the Logs of Borings in Appendix A. Laboratory tests were performed on selected samples, and the results are presented in Appendix B.

2.4 Seismic Design Considerations

Based on the International Building Code (2006 Edition), 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, soil profile type for seismic design, and the potential for liquefaction at the project site.

2.4.1 Earthquakes and Seismicity

In general, earthquakes 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 are so small that they can only 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 that are 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 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 has experienced several earthquakes greater than Magnitude 5. Therefore, moderate to strong earthquakes have occurred in the County of Maui. The effects of earthquakes occurring on the Islands of Hawaii and Maui may be felt on the Island of Oahu. For example, small landslides occurred on the Island of Oahu as a result of the Maui Earthquake of 1938 (M6.8). Some houses on the Island of Oahu were reportedly damaged as a result of the Lanai Earthquake of 1871 (M7+).

Seismic hazards on the Island of Kauai generally are considered to be low. Earthquakes with a magnitude greater than 5 have not been recorded on the Island of Kauai.

2.4.2 Soil Profile Type for Seismic Design

Our field exploration generally encountered stiff clayey silts extending to the maximum depth explored of approximately 87 feet below the existing ground surface. However, soft soils were also encountered at various borings and depths.

Based on the subsurface materials encountered 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 a “Stiff Soil Profile.” Therefore, we believe the seismic design of the building structures may be designed based on a Site Class “D” soil profile based on the International Building Code (Table No. 1613.5.2), 2006 Edition.

Based on Site Class “D,” the following seismic design parameters were estimated and may be used for seismic analysis of the project structures.

SEISMIC DESIGN PARAMETERS	
Parameter	**Value**
Mapped MCE Spectral Response Acceleration, S_S =	0.238g
Mapped MCE Spectral Response Acceleration, S_1 =	0.065g
Site Class =	“D”
Site Coefficient, F_a =	1.6
Site Coefficient, F_v =	2.4
Adjusted MCE Spectral Response Acceleration, S_{MS} =	0.381g
Adjusted MCE Spectral Response Acceleration, S_{M1} =	0.156g
Design Spectral Response Acceleration, S_{DS} =	0.254g
Design Spectral Response Acceleration, S_{D1} =	0.104g
Peak Bedrock Acceleration, PBA (Site Class B) =	0.064g
Peak Ground Acceleration, PGA (Site Class D) =	0.102g

Based on the subsurface conditions encountered, the phenomenon of soil liquefaction is not a design consideration for this project site. The risk for potential liquefaction is non-existent at this project site based on the subsurface conditions encountered (clayey soil profile with no evidence of loose granular soils) and the absence of groundwater within the depths of the borings (not encountered within boring depth).

END OF SITE CHARACTERIZATION

SECTION 3. DISCUSSION AND RECOMMENDATIONS

Our field exploration generally encountered medium soft to stiff residual and saprolite soils extending to the maximum depth explored of about 87 feet below the existing ground surface. We did not encounter groundwater in the borings at the time of our field exploration; however, groundwater levels may vary significantly depending on seasonal rainfall, time of year, and other factors.

We understand that the new water tanks will be constructed near the existing grade. Based on the relatively soft soil conditions encountered at the project site, we explored various foundation options to support the future tanks including both shallow foundation system and deep foundations system.

A shallow foundation system for tanks situated near the existing grade was first evaluated. However, due to the heavy structural loads and compressible subsoils, excessive settlements on the order of 8 to 12 inches were calculated.

We evaluated the effect of removing up to 10 feet of soils below the tanks and replacing with structural fill to eliminate the settlements from this layer. However, due to the large size of the tanks (about 70 feet diameter), the zone of influence below the tanks is very large, therefore, significant settlements (on the order of 5 to 8 inches) will still occur from the soils below the 10-foot over-excavation level.

Another alternative evaluated consisted of lowering the tank finished grade to provide a buried tank. From a foundation design standpoint, this alternative would lessen the load on the subsoils by the weight of the soil removed, reducing the settlements. However, the calculated settlements still were excessive and the buried tanks would require costly retaining walls to maintain the lower grades around the tanks.

Other alternatives, including surcharging and ground stabilization/densification, were considered but did not appear cost effective and could adversely affect the adjacent neighboring properties.

Based on the above evaluations, a deep foundation system was recommended for the tank foundations. Two systems, drilled shafts and micropiles, are discussed in this report. Driven precast concrete piles were not considered due to possible adverse effects to adjacent properties from the pile driving vibrations, possible noise issued to the residential properties, and difficulties to transport precast pile sections to the site via the narrow and winding roads.

Our laboratory tests indicate that the in-situ soil moisture contents range between about 30 and 60 percent. We anticipate that wet soils will be encountered during the trench excavation and subsequent backfilling operations at the project site. Because aeration of the high moisture soils may not be practical in the high rainfall environment of the site, compaction of the retaining wall, trench backfills, and road subgrade to the normal specified compaction requirement of 90 and 95 percent relative compaction is anticipated to be problematic. Therefore, we believe that the compaction requirements and pavement structural sections for the road should be modified and imported backfill will be required for retaining walls and trenches.

Our geotechnical engineering recommendations are discussed in the following sections.

3.1 Drilled Shaft Foundation

Based on the structural load information provided, one deep foundation solution involves constructing approximately 76 new drilled shafts to support the proposed tanks. A 12-foot by 12-foot grid was superimposed over the tank layout, and each node will be support by a single cast-in-place concrete drilled shaft. Based on the information provided by the project structural engineer, the future drilled shaft foundation system will be subjected to the following structural load demands.

DRILLED SHAFT FOUNDATION ALLOWABLE LOADING DEMAND		
Center (kips)	Center Column (kips)	Perimeter (kips)
205	300	263

Because of the nature of the water tank structures, the foundation uplift demand is minimum. Based on the above condition, the drilled shaft foundation is mainly designed against the vertical loads.

Due to the remote location of the project site, we anticipate that 3-foot diameter drilled shafts will be most feasible and economical comparing to other diameter shafts. The cast-in-place concrete drilled shafts would derive vertical support from both friction and end bearing between the concrete shaft and the surrounding soils. The following table summarizes our recommendation of drilled shaft properties.

3-FOOT DIAMETER DRILLED SHAFT FOUNDATION DESIGN				
Location	Allowable Compressive Load Capacity (kips)	Ultimate Compressive Load Capacity (kips)	Total Quantity For Each Water Tank	Minimum Shaft Length (feet)
Center	205	513	18	43
Center Column	300	750	4	56
Perimeter	263	751	16	56

The allowable compressive load capacity for the drilled shafts is for dead-plus-live loads. The compressive load capacity may be increased by one-third (1/3) when considering transient loads, such as wind or seismic forces. A factor of safety of 2.0 was used to derive the allowable compressive load capacity from ultimate compressive load capacity. It should be noted that the allowable compressive load capacity has been also reduced due to the close-spacing between the shafts. Therefore, the actual allowable capacity of single shaft will be higher than the value tabulated above.

3.1.1 Trial Shaft Program

A trial shaft program is normally required and highly recommended for drilled shaft foundation projects. Considering the project location and structural load capacities of the drilled shafts, we recommend undertaking a trial shaft program, including the performance of an instrumented load test, to fulfill the following objectives:

- To examine the adequacy of the methods and equipment proposed by the contractor to install the drilled shafts through the stiff residual/saprolite soils.
- To confirm or modify the estimated tip elevations of the drilled shafts.
- To assess the contractor’s method of placing and extracting the temporary casing for the drilled shaft.
- To assess the contractor’s method of tremmie concrete placement.

To achieve these objectives, we recommend that the trial shaft program consist of drilling a 3-foot diameter trial shaft extending to a depth of at least 75 feet below the existing ground surface. The location of the trial shaft should be near, but outside of, the tank foundations. After drilling the trial shaft, the trial shaft should be inspected to evaluate the contractor’s drilling capability. If accepted by the engineer, the trial shaft may be converted to a load test shaft for load testing purposes.

The load test shaft should be structurally reinforced and instrumented with embedment strain gauges for load testing. The embedment strain gauges should be placed starting from the bottom at an elevation of about 5 feet above the tip of the trial shaft and subsequently at 10-foot intervals.

Due to the high capacities recommended for the drilled shafts, a conventional load test would not be practical and would be costly to conduct. Therefore, a bi-directional axial load test should be conducted on the reinforced load test shaft using an expandable base load cell (Osterberg Load Cell). The expandable base load cell will need to be attached to the reinforcing cage prior to lowering the reinforcing cage in place. The drilled shaft load test should be performed in general accordance with the Quick Load Test Method of ASTM D 1143. In general, the load test shaft should be loaded at increments of about 100 to 200 kips, and up to the ultimate load capacity tabulated above. The load test should be held for a minimum of 12 hours at or near failure to evaluate the potential for creep effects. The load test shaft should then be loaded to failure to evaluate the ultimate side shear resistance of the trial shaft. Installation of the expandable base load cell and embedment strain gauges, performance of the bi-directional axial load test, and analyses of the load test data should be performed by a qualified professional experienced in these types of load testing procedures.

Considering the specialized nature of the trial shaft program, we recommend a Geolabs representative be present during the trial shaft and load testing program to evaluate the contractor's method of drilled shaft installation and to evaluate the subsurface materials encountered. In addition, Geolabs should observe the instrumented load test on the reinforced load test shaft. It should be noted that the drilled shaft design was developed from our analyses using the field exploration data. Therefore, observation of the drilled shaft installation operations by Geolabs is a vital part of the foundation design to confirm the design assumptions.

3.1.2 Foundation Settlement

Settlement of the drilled shaft foundation will result from elastic compression of the shaft and subgrade response of the foundation embedded in the soils encountered at the site. The total settlements of the drilled shafts are estimated to be on the order of 0.5 inch and the potential differential settlements between adjacent drilled shafts may be on the order of 0.25 inch. We believe that a significant portion of the settlement will be elastic and should occur as the loads are applied.

3.1.3 Drilled Shaft Construction Considerations

The performance of the shaft will significantly depend upon the contractor's method of construction and construction procedures. As a result, we recommend that a Geolabs representative observe the drilled shaft installation during construction. In our opinion, the following may have a significant impact on the effectiveness and cost of the drilled shaft foundations.

The load carrying capacity of the drilled shaft depends on both the friction between the shaft and the surrounding soil and the end bearing over the soil at the bottom of the shaft. Therefore, proper construction techniques are important. The contractor should exercise care in drilling or excavating the shaft hole and in placing concrete in the hole.

Our field exploration revealed that the project site is generally underlain by stiff residual soil and saprolite soil. It is very common that hard rock cores exist within this type of soils. The hardness and extent of rock core might vary significantly at different locations. Therefore, some difficult drilling conditions will likely be encountered at the project site and should be expected. In addition, although water table was not encountered during our field exploration at the project site, wet soils were recovered throughout all our borings, possibly due to perched water and seepage zones. It is possible that the excavated hole will accumulate water after it reaches the design tips.

Temporary casing of the drilled holes might be required during the drilled shaft installation to keep the drilled hole open and provide a safe working zone for field personnel. Temporary casing may be extended to a suitable depth determined by the contractor. The casing shall be continuous between the top and bottom elevations, and shall be advanced through the ground by twisting, driving or vibration before cleaning out the shaft.

A low-shrink concrete mix with high slump (6 to 8-inch slump range) should be used to provide close contact between the drilled shafts and the surrounding soils. Concrete should be placed in a suitable manner to reduce the potential for segregation of the aggregates from the concrete mix. In addition, concrete should be placed promptly after drilling (within 24 hours after drilling of the holes) to reduce the potential for caving-in the sides of the drilled holes. Based on the recommended shaft length, we recommended using tremmie method for concrete placement. It should be noted that the tip of the tremmie pipe should be kept at least 5 feet below the fresh grout surface to minimize grout contamination.

In addition, due to the remote location of the site, access for the traditional drilling equipment might be restricted. The drilled shaft subcontractor shall evaluate the site access and shall have the appropriate equipment and tools to drill through rock, if encountered.

It is imperative that a Geolabs representative is present at the project site to observe the drilling and installation of drilled shafts during construction. Geolabs observation of the drilled shaft installation operations is necessary to confirm the assumed subsurface conditions and should be designated a "Special Inspection" item in accordance with Section 1704 of the International Building Code (2006 Edition).

3.2 Micropile Foundation

Based on the current site conditions, micropile foundation might also be a viable option to support the propose tank structures. With a smaller working footprint, micropile system tends to be more flexible when construction working spaces are limited.

In general, a micropile foundation system consists of a small diameter (usually less than 12 inches), drilled and grouted, pile with reinforcing steel. The micropile foundation typically is constructed by drilling a borehole, placing reinforcing steel in the hole, and grouting the borehole. Micropiles are desirable because they can be installed readily in access restrictive environments with generally smaller equipment and in numerous soil types and ground conditions. In addition, installation of the micropiles generally causes minimal disturbance to the adjacent soils and the environment.

Based on the structural information provided, the proposed water tanks might be supported by 246 micropiles. A 6-foot by 6-foot grid was superimposed over the tank layout by the project structural engineer, and each node will be support by a single micropile. The structural load demands at each micropile location are tabulated below.

MICROPILE FOUNDATION ALLOWABLE LOADING DEMAND		
Center (kips)	**Center Column** (kips)	**Perimeter** (kips)
52	145	78

Based on our experience with similar projects and the structural load demands provided, we believe a micropile system with a minimum grout bulb diameter of 7 inches (minimum drill bit size) should be used to support the proposed tank structures. The micropiles would derive its vertical support primarily from skin friction between the grout bulb and the surrounding stiff residual/saprolitic materials. Based on the above assumptions and our engineering analyses, we recommend installing the micropiles as listed in the table below.

7-INCH DIAMETER MICROPILE FOUNDATION DESIGN				
Location	**Ultimate Compressive Load Capacity** (kips)	**Total Quantity For Each Water Tank**	**Minimum Micropile Length** (feet)	**Unbound Casing Length** (feet)
Center	130	97	54	10
Center Column	362*	8	67	10
Perimeter	195	18	72	10

*Supported by two micropiles at minimum 3.5 feet center-to-center distance

Based on the subsurface conditions at the project site, we recommend providing a permanent steel casing at the top of micropiles for a minimum length (unbound length) of 10 feet below the bottom of footing elevation. The permanent steel casing should have an outside diameter (OD) of about 7 inches (same as the grout bulb size), and the permanent steel casing should provide confinement to the micropile in the area where moment demand on the micropile is the greatest. It should be noted that the 10-foot unbound casing length does not include any casing length required by the structural engineer to tie into the tank footings.

The load-supporting capacity of micropiles is highly dependent on the installation procedures of the micropiles. Due to variations in the subsurface materials and the potential of hard rock core in the subsurface, we strongly recommend conducting a micropile static load test program to further evaluate and validate our assumptions in providing the above micropile recommendations for support of the new structural elements at the project site. Due to possible variation in the length of the micropiles, unit prices should be obtained during bidding for add-ons, shorter micropiles, etc.

3.2.1 Micropile Load Test Program

It should be noted that the load carrying capacity of the micropiles is highly dependent on the drilling procedures and the grouting methods employed by the contractor to install the micropile. Therefore, the load capacity of the micropile may vary considerably between different contractors and micropile foundation systems. In order to determine whether the contractor's methods of micropile installation are adequate and to determine the ultimate load capacity, we recommend that at least one pre-production compressive load test be performed on a sacrificial micropile at or near each new tank location. It should be noted that the location of sacrificial test pile and reaction piles installed for load test should not be coincident with future pile location.

In general, the purpose of the pre-production load test on a sacrificial micropile is to fulfill the following objectives:

- To examine the adequacy of the methods and equipment proposed by the contractor to install the micropiles to the depths required.
- To assess the contractor's method of drilling and grout injection.
- To confirm or modify the estimated tip elevations of the micropiles.

The pre-production load tests should be performed in accordance with ASTM D 1143 (Compressive Load Test). Based on experience with similar projects, we recommend conducting the load test no earlier than 7 days after completion of the micropile installation. Additional micropiles may be used for reaction during the compressive load testing of the pre-production load test micropile. The reaction micropiles should be installed to adequate depths (or as deep as the load test micropile) to provide the necessary reaction in uplift.

We recommend the load test piles have a minimum length of 72 feet, and the maximum test load be about 200 kips. We recommend the maximum test load for the compressive load test be held for a minimum of 4 to 8 hours depending on the recorded movements of the load test micropile. The pre-production load tests are an integral part of the design of the micropile foundation system. Therefore, we recommend conducting the pre-production load tests under the observation of a Geolabs representative.

3.2.2 Micropile Foundation Settlements

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

3.2.3 Construction Considerations

As mentioned earlier, our field exploration revealed that the project site is generally underlain by residual soil and saprolite soil. It is very common that hard rock cores exist within this type of soils. The hardness and extend of rock core might vary significantly at different locations and depths. Therefore, some difficult drilling conditions will likely be encountered at the project site and should be expected.

Installation of micropiles should be performed by a specialty subcontractor experienced in the construction of a micropile foundation system. Due to the specialized nature of the micropile foundation construction, observation of the micropile foundation installation system and testing of the micropiles should be designated a “Special Inspection” item. Therefore, it is necessary for a Geolabs representative to observe the micropile installation operations to confirm our design assumptions.

3.3 Site Grading

Based on the available site plan and current design concept, we anticipate minimum fills and cuts will be required to achieve the design grades.

At the on-set of earthwork, the areas within the contract grading limits should be cleared and grubbed thoroughly. Vegetation, debris, deleterious materials, and other unsuitable materials should be removed and disposed properly off-site. Soft and yielding areas encountered during clearing and grubbing below footing areas should be over-excavated to expose firm natural material and the resulting excavation should be backfilled with well-compacted fill. The excavated soft and/or organic soils should be properly disposed off-site or used in landscaping areas, if appropriate.

Where shrinkage cracks are noted after preparation of the subgrade, we recommend thoroughly moistening the soils to close the cracks and recompacting. 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.

Imported fill material should consist of non-expansive imported granular material, such as crushed coral, basalt or cinder sand. The material 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 material should have a laboratory CBR value of 20 or more and should have a maximum swell of less than 1 percent. Geolabs should test imported fill material for conformance with these recommendations prior to delivery to the project site for its intended use. Select borrow and aggregate base course should meet the requirement as specified in Sections 30 and 31 of the Standard Specifications for Public Works Construction, Department of Public Works, September 1986.

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 on-site soils. Select borrow, base course, and imported granular fill materials should be moisture-conditions to above the optimum moisture, placed in level lifts not exceeding 8 inches in loose thickness, and compacted to a minimum of 95 percent relative compaction. Relative compaction refer 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 D 1557 test procedures. Optimum moisture is the water content (percentage by dry weight) corresponding to the maximum dry density.

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

3.4 Utility Trench Backfill

We envision that new connecting pipe lines will be required for this project. In general, for support of the pipe lines, we recommend using granular bedding consisting of 6 inches of No. 3B Fine gravel (ASTM C 33, No. 67 gradation) under the pipe lines. The initial backfill up to about 12 inches above the pipes should consist of free-draining backfills, such as No. 3B Fine gravel, to reduce the potential for pipe damage from compaction backfill. It is critical to use a free draining granular material to reduce the potential for formation of voids below the haunches of pipes and to provide adequate support for the sides of the pipes. The use of on-site clayey soils as backfill immediately around water line pipes is not recommended.

As previously mentioned, we anticipate that there may be some difficulty in obtaining the normal specified compaction requirements for the trench backfill due to the wet soil conditions. Therefore, we believe that the upper portion of the trench backfill from the level 1-foot above the pipes to the finished subgrade may consist of the on-site soils compacted to a lesser degree. The backfill material should be moisture-conditioned to above the optimum moisture content, placed in level lifts not exceeding 8 inches in loose thickness, and compacted to a minimum of 85 percent relative compaction.

It should be noted that the reduced compaction requirement for the trench backfill materials would result in some additional trench backfill settlement. Therefore, we believe that the reduced compaction requirement for the trench backfill materials should be limited to a total trench depth of 10 feet or less to reduce the potential for appreciable ground subsidence due to the lower compaction trench backfill materials. For the deeper trenches, we believe that the compaction requirement for the lower portion of the trench backfill should be increased to 90 percent relative compaction. In order to achieve the higher compaction requirement, imported backfill materials may be required. Geolabs should be contacted to provide supplemental recommendations for this condition, if appropriate.

3.5 Corrosion Potential

Three sets of laboratory resistivity tests, including pH and Minimum Resistivity were conducted on selected soil samples from our field exploration. These tests were designed to evaluate the corrosivity of the underlying subsurface materials encountered. Based on the test results (refer to Plate B-10), the subsurface materials encountered exhibit low minimum resistivity values in the range of about 4,800 to 10,000 ohm-cm, which correspond to Corrosion Rating of 3 to 4 (Corrosive to Moderately Corrosive). Therefore, we recommend properly designing all metallic substructures in contact with the soils be protected against potential corrosion. A corrosion engineer should be consulted for detailed recommendations on corrosion protection.

As a minimum, we recommend providing a permanent steel casing in the upper 10 feet of the micropile to protect the micropile reinforcing steel bar from corrosion and also to provide adequate confinement to the upper portion of the micropile where the structural demand on the micropile is the greatest.

3.6 Pavements

We anticipate that the vehicle loading for the pavement areas will consist primarily of passenger vehicles and light utility trucks. We have made our preliminary pavement design assuming the pavement subgrade soil will be similar to the silty soils encountered during our field exploration and represented by the CBR test. Based on the above assumptions, we recommend using the following pavement sections for design purposes:

Asphaltic Concrete Pavement

2.0-Inch Asphaltic Concrete
6.0-Inch Aggregate Base Course (95 Percent Relative Compaction)
6.0-Inch Select Borrow Subbase (90 Percent Relative Compaction)
14.0-Inch Total Pavement Thickness on Compacted Subgrade

Portland Cement Concrete Pavement

6.0-Inch Portland Cement Concrete
6.0-Inch Select Borrow Subbase (90 Percent Relative Compaction)
12.0-Inch Total Pavement Thickness on Compacted Subgrade

Due to the high rainfall environment and the high in-situ moisture contents of the existing on-site soils, we believe that the common compaction requirement of 95 percent relative compaction for the subgrade soils may be impractical and should be reduced. Based on our laboratory test results and engineering analyses, the subgrade soils under the pavement areas should be proof-rolled to provide a subgrade with a minimum of 85 percent relative compaction. Based on the available compaction tests performed, we believe that the 85 percent relative compaction may be achieved with some difficulty.

Because of the reduced subgrade compaction requirement and the high in-situ moisture content, we believe that it would be difficult to obtain a compaction requirement of typical 95 percent for the first layer of the select borrow subbase placed over the pavement subgrades. Therefore, we recommend compacting the first layer of select borrow subbase placed above the pavement subgrades to a minimum of 90 percent relative compaction.

The aggregate base course and select borrow subbase should consist of crushed basaltic aggregate and should meet the requirement as specified in Sections 30 and 31 of the Standard Specifications for Public Works Construction, Department of Public Works, September 1986. CBR tests and/or field observations should be performed on the actual subgrade soils at the in-situ moisture content during construction to confirm that the above design section is adequate. In addition, the recommended pavement section assumes that good drainage will be provided for areas adjacent to the pavements.

Paved areas should be sloped, and drainage gradients should be maintained to carry surface water off the site. Surface water ponding should not be allowed on the site during or after construction.

3.7 Drainage

The finished grades outside the new storage tank structures should be sloped to shed water away from foundations and slabs and to reduce the potential for ponding. Excessive landscape watering near the foundations and slabs should also be avoided. These drainage requirements are essential for the proper performance of the above foundation recommendations since ponded water could cause subsurface soil saturation and subsequent heaving or loss of strength. The foundation excavations should be properly backfilled against the walls or slab footings immediately after setting of the concrete to reduce water infiltration. In addition, drainage swales should be provided as soon as possible and should be maintained to drain all surface run-off away from slabs and foundations.

3.8 Design Review

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

3.9 Construction Monitoring

It is recommended to retain Geolabs, Inc. to provide geotechnical engineering services during construction of the proposed project. The critical items of construction monitoring that require "Special Inspection" include observation of deep foundation construction and testing, subgrade preparation, fill placement, and compaction. A Geolabs representative should monitor other aspects of earthwork construction to observe compliance with the intent of the design concepts, specifications, and/or recommendations, and to expedite suggestions for design changes that may be required in the event that subsurface conditions differ from those anticipated at the time this report was prepared. The recommendations provided herein are contingent upon such observations.

If actual exposed subsurface conditions encountered during construction differ from those assumed or considered in this report, Geolabs should be contacted to review and/or revise the geotechnical engineering recommendations presented herein.

END OF DISCUSSION AND RECOMMENDATIONS

SECTION 4. LIMITATIONS

The analyses and recommendations submitted herein are based in part upon information obtained from the field borings. Variations of subsoil 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, Geolabs, Inc. should be contacted to re-evaluate the recommendations presented in this report.

The field boring locations were determined by taping from structures indicated on the Preliminary Site Plan provided by Belt Collins Hawaii Ltd. on April 2007. Elevations of the field borings were interpolated from the contour lines shown on the same plan. The field boring locations and elevations should be considered accurate only to the degree implied by the methods used.

The stratification lines shown on the graphic representations of the borings depict the approximate boundaries between the soil types, and as such, may denote a gradual transition. Water level data from the borings were measured at the times shown on the graphic representations and/or in the text of this report. These data have been reviewed and interpretations made in the formulation of this report. However, it must be noted that fluctuation may occur due to variation in rainfall, temperatures, and other factors.

This report has been prepared for the exclusive use of Belt Collins Hawaii Ltd. and other project consultants for specific application to the proposed *Kapahi 1.0 MG Storage Tanks* project in accordance with generally accepted geotechnical engineering principles and practices. No warranty is expressed or implied.

This report has been prepared solely for the purpose of assisting the design engineers and architect in the preliminary design evaluation of the proposed project. Therefore, this report may not contain sufficient data, or the proper information, to serve as the basis for preparation of construction cost estimates nor for bidding purposes. A contractor wishing to bid on this project should retain a competent geotechnical engineer to assist in the interpretation of this report and/or in the performance of additional site specific exploration for bid estimating purposes.

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

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

END OF LIMITATIONS

CLOSURE

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

-ΩΩΩΩΩΩΩΩ-

Respectfully submitted,

GEOLABS, INC.

By Clayton S. Mimura

Clayton S. Mimura, P.E.
President

CSM:RP:as

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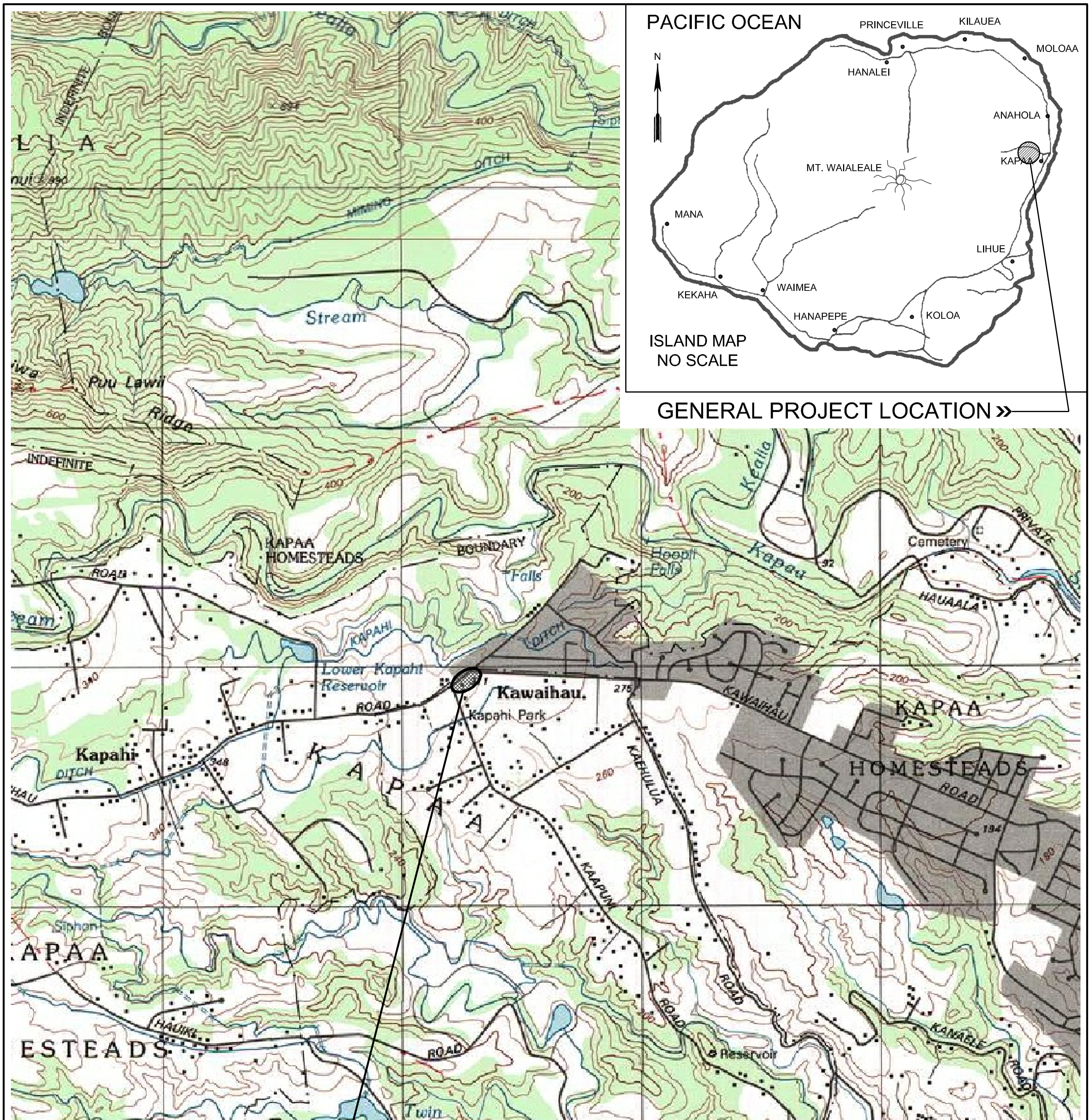
PLATES



PROJECT LOCATION MAP

KAPAHI 1.0 MG (313 FEET) STORAGE TANK
WAIALUA-KAPAA WATER SYSTEM
KAPAA, KAUAI, HAWAII

2000 1000 0 2000 FT.

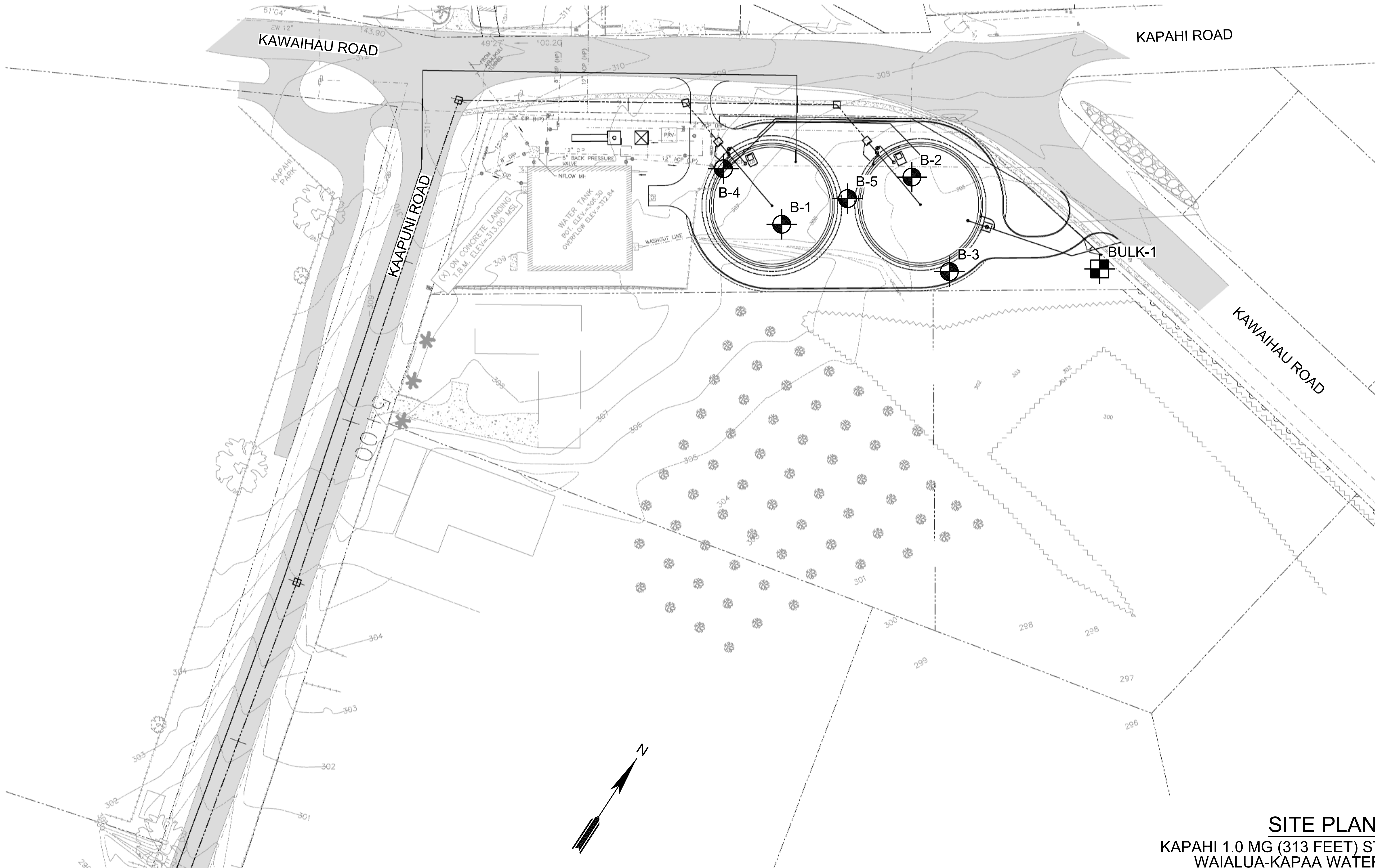
GRAPHIC SCALE

N

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

GEOLABS, INC. *Geotechnical Engineering*		
DATE APRIL 2011	DRAWN BY HYC	PLATE
SCALE 1" = 2,000'	W.O. 5988-00&10	1

CAD User: HENRY File Last Updated: April 15, 2011 4:24:31pm Plot Date: September 30, 2011 - 5:44:47pm
File: T:\Drafting-9904\Working\5988-00&10KapahiStorageTank\5988-00&10PLM.dwg\plm
Plotter: CutePDF Writer Plotstyle: GEOLABS-DC260-Modified-COLOR.ctb



SITE PLAN

KAPAHI 1.0 MG (313 FEET) STORAGE TANK
WAIALUA-KAPAA WATER SYSTEM
KAPAA, KAUAI, HAWAII

LEGEND:

- APPROXIMATE BORING LOCATION
- APPROXIMATE BULK SAMPLE LOCATION

REFERENCE:
TOPO RECEIVED FROM BELT COLLINS HAWAII, LTD ON AUGUST 10, 2010.

0 20 40 60 120 180 FT.
GRAPHIC SCALE

GEOLABS, INC.
Geotechnical Engineering

DATE	DRAWN BY	PLATE
APRIL 2011	HYC	
SCALE	W.O.	2
1" = 60'	5988-00&10	

APPENDIX A

APPENDIX A

Field Exploration

We explored the subsurface conditions at the project site by drilling and sampling five borings, designated as Boring Nos. 1 through 5, extending to depths of about 26 to 87 feet below the existing ground/pavement surface. The approximate boring locations are shown on the Site Plan, Plate 2. The borings were drilled using truck-mounted drill rigs equipped with continuous flight augers and coring tools.

Our geologists classified the materials encountered in the borings by visual and textural examination in the field in general accordance with ASTM D 2488, 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 D 2487, Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System), as shown on the Soil Log Legend, Plate A. Graphic representations of the materials encountered are presented on the Logs of Borings, Plates A-1 through A-5.

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

GEOLABS, INC. Geotechnical Engineering	Soil Log Legend

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)

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

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

<u>LEGEND</u>

	(2-INCH) O.D. STANDARD PENETRATION TEST	LL	LIQUID LIMIT (NP=NON-PLASTIC)
	(3-INCH) O.D. MODIFIED CALIFORNIA SAMPLE	PI	PLASTICITY INDEX (NP=NON-PLASTIC)
S	SHELBY TUBE SAMPLE	TV	TORVANE SHEAR (tsf)
G	GRAB SAMPLE	PEN	POCKET PENETROMETER (tsf)
	CORE SAMPLE	UC	UNCONFINED COMPRESSION (psi)
∇	WATER LEVEL OBSERVED IN BORING	UU	UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION (ksf)

Plate

A

LOG LEGEND FOR SOIL 5988-00.GPJ GEOLABS.GDT 9/30/11

GEOLABS, INC. Geotechnical Engineering	KAPAHI 1.0 MG (313 FEET) STORAGE TANK WAIALUA-KAPAA WATER SYSTEM KAPAA, KAUAI, HAWAII	Log of Boring 1

Approximate Ground Surface Elevation (feet): 306.5 *

Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)	Depth (feet)	USCS	Description
							0	MH	Reddish brown **CLAYEY SILT**, stiff, moist (residual soil)
	32	75			10	1.0			
LL=57 PI=22	34				6	1.5			grades to soft
Direct Shear	35	84			22	1.0	5	MH	Reddish brown **CLAYEY SILT** with little sand and gravel, stiff, moist (saprolite)
	39				7	1.0	10		grades to soft
Direct Shear	55	68			13	1.0	15		
								ML	Reddish brown **SANDY SILT** with traces of clay, stiff, moist to wet (saprolite)
	46				11	1.0	20		
	52	72			36	2.0	25		grades to multi-color mottled, very stiff
LL=62 PI=15	65				14	2.0	30	MH	Reddish brown **CLAYEY SILT** with traces of sand, stiff, wet (saprolite)
							35		

Date Started:	January 24, 2008	Water Level: ⊻	Not Encountered	Plate
Date Completed:	January 24, 2008			
Logged By:	Y. Chiba	Drill Rig:	CME-55	
Total Depth:	41 feet	Drilling Method:	4" Auger	A - 1.1
Work Order:	5988-00&10	Driving Energy:	140 lb. wt., 30 in. drop	

GEOLABS, INC. Geotechnical Engineering	KAPAHI 1.0 MG (313 FEET) STORAGE TANK WAIALUA-KAPAA WATER SYSTEM KAPAA, KAUAI, HAWAII	Log of Boring 1

(Continued from previous plate)

Laboratory			Field								
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)	Depth (feet)	Sample	Graphic	USCS	Description
Consol.	57	60			35	2.0				MH	grades to very stiff
	53				25	2.0	40				
							41				Boring terminated at 41 feet * Elevations estimated from Topo received from Belt Collins Hawaii, Ltd. on August 10, 2010.
							45				
							50				
							55				
							60				
							65				
							70				

Date Started:	January 24, 2008	Water Level: ▽	Not Encountered	Plate
Date Completed:	January 24, 2008			
Logged By:	Y. Chiba	Drill Rig:	CME-55	A - 1.2
Total Depth:	41 feet	Drilling Method:	4" Auger	
Work Order:	5988-00&10	Driving Energy:	140 lb. wt., 30 in. drop	

GEOLABS, INC.

Geotechnical Engineering

KAPAHI 1.0 MG (313 FEET) STORAGE TANK
WAIALUA-KAPAA WATER SYSTEM
KAPAA, KAUAI, HAWAII

Log of Boring

2

Approximate Ground Surface Elevation (feet): 305 *

Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)	Depth (feet)	USCS	Description
								CH	Brown **CLAY** with roots, stiff, damp to moist (residual soil)
	45	78			15	2.0			
	40				15	2.0			
	41	78			20	1.5	5	MH	Reddish brown **CLAYEY SILT** with little sand and gravel, very stiff, damp to moist (residual soil)
									grades to stiff
	47				10	1.0	10		
	54	71			15	2.0	15	ML	Reddish brown **SANDY SILT** with little clay, stiff, wet (saprolite)
	56				14	2.0	20		grades with more clay
	54	69			23	2.5	25		grades with more sand, very stiff
								MH	Reddish brown **CLAYEY SILT** with little fine sand, very stiff, wet (saprolite)
	64				8	2.0	30		grades to medium stiff
							35		

Date Started:	January 25, 2008
Date Completed:	January 25, 2008
Logged By:	Y. Chiba
Total Depth:	41 feet
Work Order:	5988-00&10

Water Level: ⊻	Not Encountered
Drill Rig:	CME-55
Drilling Method:	4" Auger
Driving Energy:	140 lb. wt., 30 in. drop

Plate

A - 2.1

GEOLABS, INC. Geotechnical Engineering	KAPAHI 1.0 MG (313 FEET) STORAGE TANK WAIALUA-KAPAA WATER SYSTEM KAPAA, KAUAI, HAWAII	Log of Boring 2

Laboratory: Other Tests	Laboratory: Moisture Content (%)	Laboratory: Dry Density (pcf)	Field: Core Recovery (%)	Field: RQD (%)	Field: Penetration Resistance (blows/foot)	Field: Pocket Pen. (tsf)	Depth (feet)	Sample	Graphic	USCS	Description (Continued from previous plate)
	62	48			22	2.5				MH	grades to very stiff
											grades to hard
	53				41	3.5	40				
											Boring terminated at 41 feet
							45				
							50				
							55				
							60				
							65				
							70				

Date Started:	January 25, 2008	Water Level: ∇	Not Encountered
Date Completed:	January 25, 2008		
Logged By:	Y. Chiba	Drill Rig:	CME-55
Total Depth:	41 feet	Drilling Method:	4" Auger
Work Order:	5988-00&10	Driving Energy:	140 lb. wt., 30 in. drop

Plate A - 2.2

BORING_LOG 5988-00.GPJ GEOLABS.GDT 9/30/11

GEOLABS, INC. Geotechnical Engineering	KAPAHI 1.0 MG (313 FEET) STORAGE TANK WAIALUA-KAPAA WATER SYSTEM KAPAA, KAUAI, HAWAII	Log of Boring 3

Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)	Depth (feet)	USCS	Description
									Approximate Ground Surface Elevation (feet): 304 *
							0	MH	Reddish brown **CLAYEY SILT** with little sand, stiff, damp to moist (residual soil)
	43	78			16	2.0			
	39				14	2.5			
								CH	Reddish brown **SILTY CLAY** with little sand, very stiff, damp to moist (residual soil)
	42	84			28		5		
	44				9	1.0	10		grades with some extremely weathered gravel (basaltic), stiff
								ML	Reddish brown **SANDY SILT** with little clay, very stiff, moist (saprolite)
	43	75			22	1.5	15		
	56				8	1.0	20		grades to medium stiff
	62	58			22	1.0	25		grades to very stiff
								MH	Reddish brown **CLAYEY SILT** with little sand, stiff, moist to wet (saprolite)
	61				8	1.0	30		grades to medium stiff
									grades to wet
							35		

Date Started:	January 25, 2008	Water Level: ∇	Not Encountered	Plate
Date Completed:	January 25, 2008			
Logged By:	Y. Chiba	Drill Rig:	CME-55	A - 3.1
Total Depth:	41 feet	Drilling Method:	4" Auger	
Work Order:	5988-00&10	Driving Energy:	140 lb. wt., 30 in. drop	

GEOLABS, INC. Geotechnical Engineering	KAPAHI 1.0 MG (313 FEET) STORAGE TANK WAIALUA-KAPAA WATER SYSTEM KAPAA, KAUAI, HAWAII	Log of Boring 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	(Continued from previous plate) Description
Consol.	52	61			39	2.0				MH	grades to very stiff
	56				18	2.0	40				
							41				Boring terminated at 41 feet
							45				
							50				
							55				
							60				
							65				
							70				

Date Started:	January 25, 2008	Water Level: ⊻	Not Encountered	Plate
Date Completed:	January 25, 2008			
Logged By:	Y. Chiba	Drill Rig:	CME-55	
Total Depth:	41 feet	Drilling Method:	4" Auger	A - 3.2
Work Order:	5988-00&10	Driving Energy:	140 lb. wt., 30 in. drop	

GEOLABS, INC.

Geotechnical Engineering

KAPAHI 1.0 MG (313 FEET) STORAGE TANK
WAIALUA-KAPAA WATER SYSTEM
KAPAA, KAUAI, HAWAII

Log of Boring

4

Approximate Ground Surface Elevation (feet): 308 *

Description

Other Tests (Laboratory)	Moisture Content (%) (Laboratory)	Dry Density (pcf) (Laboratory)	Core Recovery (%) (Field)	RQD (%) (Field)	Penetration Resistance (blows/foot) (Field)	Pocket Pen. (tsf) (Field)	Depth (feet)	USCS	Description
							0	MH	Reddish brown **CLAYEY SILT**, medium stiff, moist (fill)
	38	80			11	1.5			
	39				13	1.0	5	MH	Orangish brown **CLAYEY SILT** with traces of gravel (basaltic), stiff, moist (residual soil)
					10	1.3	10		
							13	ML	Reddish brown **CLAYEY SILT** with some gravel (basaltic), medium stiff to stiff, moist (saprolite)
	54				9	1.3	15		
LL=NP PI=NP Consol.	60	66			18	1.0	20		grades with fine sand
	67				5	0.8	25		grades to soft
							26		Boring terminated at 26 feet

Date Started:	March 9, 2011	Water Level: ⊻	Not Encountered
Date Completed:	March 9, 2011		
Logged By:	S. Latronic	Drill Rig:	CME-55
Total Depth:	26 feet	Drilling Method:	4" Casing & PQ Coring
Work Order:	5988-00&10	Driving Energy:	140 lb. wt., 30 in. drop

Plate

A - 4

GEOLABS, INC.

Geotechnical Engineering

KAPAHI 1.0 MG (313 FEET) STORAGE TANK
WAIALUA-KAPAA WATER SYSTEM
KAPAA, KAUAI, HAWAII

Log of Boring

5

Approximate Ground Surface Elevation (feet): 305.5 *

Laboratory: Other Tests	Laboratory: Moisture Content (%)	Laboratory: Dry Density (pcf)	Field: Core Recovery (%)	Field: RQD (%)	Field: Penetration Resistance (blows/foot)	Field: Pocket Pen. (tsf)	Depth (feet)	USCS	Description
								MH	Dark brown **CLAYEY SILT** with some gravel and traces of organic matters, stiff, moist (fill)
								MH	Orangish brown **CLAYEY SILT**, stiff to very stiff, moist (residual soil)
	40	76			25	3.0			
	41	74			19	3.0	5		grades to stiff
	37				14		10		
									grades with traces of gravel (basaltic)
	47	75			24	3.8	15		
								ML/ MH	Brown **CLAYEY SILT** with sand (basaltic), medium stiff, very moist (saprolite)
	59				6	1.0	20		grades to soft
					18	1.0	25		
	63				6	1.3	30		
							35	ML	

Date Started:	March 7, 2011	Water Level: ∇	Not Encountered
Date Completed:	March 9, 2011		
Logged By:	S. Latronic	Drill Rig:	CME-55
Total Depth:	87 feet	Drilling Method:	PQ Coring
Work Order:	5988-00&10	Driving Energy:	140 lb. wt., 30 in. drop

Plate

A - 5.1

GEOLABS, INC. Geotechnical Engineering	KAPAHI 1.0 MG (313 FEET) STORAGE TANK WAIALUA-KAPAA WATER SYSTEM KAPAA, KAUAI, HAWAII	Log of Boring 5

(Continued from previous plate)

Laboratory: Other Tests	Laboratory: Moisture Content (%)	Laboratory: Dry Density (pcf)	Field: Core Recovery (%)	Field: RQD (%)	Field: Penetration Resistance (blows/foot)	Field: Pocket Pen. (tsf)	Depth (feet)	USCS	Description
	54	72			50	3.5		ML	Grayish brown **CLAYEY SILT** with sand and gravel (basaltic), very stiff, moist (saprolite)
							40		
	37	91			65/5"	3.5			
							45		
	52	73			37/6" +50/4"	4.0			
			67						
							50	MH	Reddish brown **CLAYEY SILT** with sand and traces of gravel (basaltic), stiff, moist (residual soil)
	62				11	1.0			
			71						
							55		
	59	70			23	1.5			
							60	MH	Reddish brown **CLAYEY SILT** with sand and gravel (basaltic), stiff, moist (saprolite)
	63	66			28	2.5			
							65		
LL=53 PI=12 Consol.	68	62			16	1.3			
							70		

Date Started:	March 7, 2011	Water Level: ∇	Not Encountered
Date Completed:	March 9, 2011		
Logged By:	S. Latronic	Drill Rig:	CME-55
Total Depth:	87 feet	Drilling Method:	PQ Coring
Work Order:	5988-00&10	Driving Energy:	140 lb. wt., 30 in. drop

Plate

A - 5.2

GEOLABS, INC. Geotechnical Engineering	KAPAHI 1.0 MG (313 FEET) STORAGE TANK WAIALUA-KAPAA WATER SYSTEM KAPAA, KAUAI, HAWAII	Log of Boring **5**

(Continued from previous plate)

Other Tests (Laboratory)	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%) (Field)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)	Depth (feet)	USCS	Description
	57	66			21	2.0		MH	
					8	0.8	75	ML	Brown **CLAYEY SILT** with sand and traces of gravel (basaltic), soft to medium stiff, very moist (saprolite)
	55	75			56	2.5	80	ML	Grayish brown **CLAYEY SILT** with gravel (basaltic), very stiff, moist (saprolite)
			64	17					Brownish gray **BASALT**, moderately fractured, moderately to highly weathered, soft to medium hard
	72	60			11	1.0	85	ML	Grayish brown **CLAYEY SILT** with sand and traces of gravel (basaltic), stiff, moist (saprolite)
									Boring terminated at 87 feet
							90		
							95		
							100		
							105		

Date Started:	March 7, 2011	Water Level: ⊻	Not Encountered
Date Completed:	March 9, 2011		
Logged By:	S. Latronic	Drill Rig:	CME-55
Total Depth:	87 feet	Drilling Method:	PQ Coring
Work Order:	5988-00&10	Driving Energy:	140 lb. wt., 30 in. drop

Plate

A - 5.3

APPENDIX B

APPENDIX B

Laboratory Tests

Moisture Content (ASTM D 2216) and Unit Weight determinations (ASTM D 2937) 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.

Four Atterberg Limits test (ASTM D 4318) was performed on a selected soil sample to evaluate the liquid and plastic limits and to aid in soil classification. The test results are presented on the Logs of Borings at the appropriate sample depths. Graphic representations of the test results are provided on Plate B-1.

Two Direct Shear tests (ASTM D 3080) were performed on selected samples to evaluate the shear strength characteristics of the materials tested. Test results are presented on Plates B-2 and B-3.

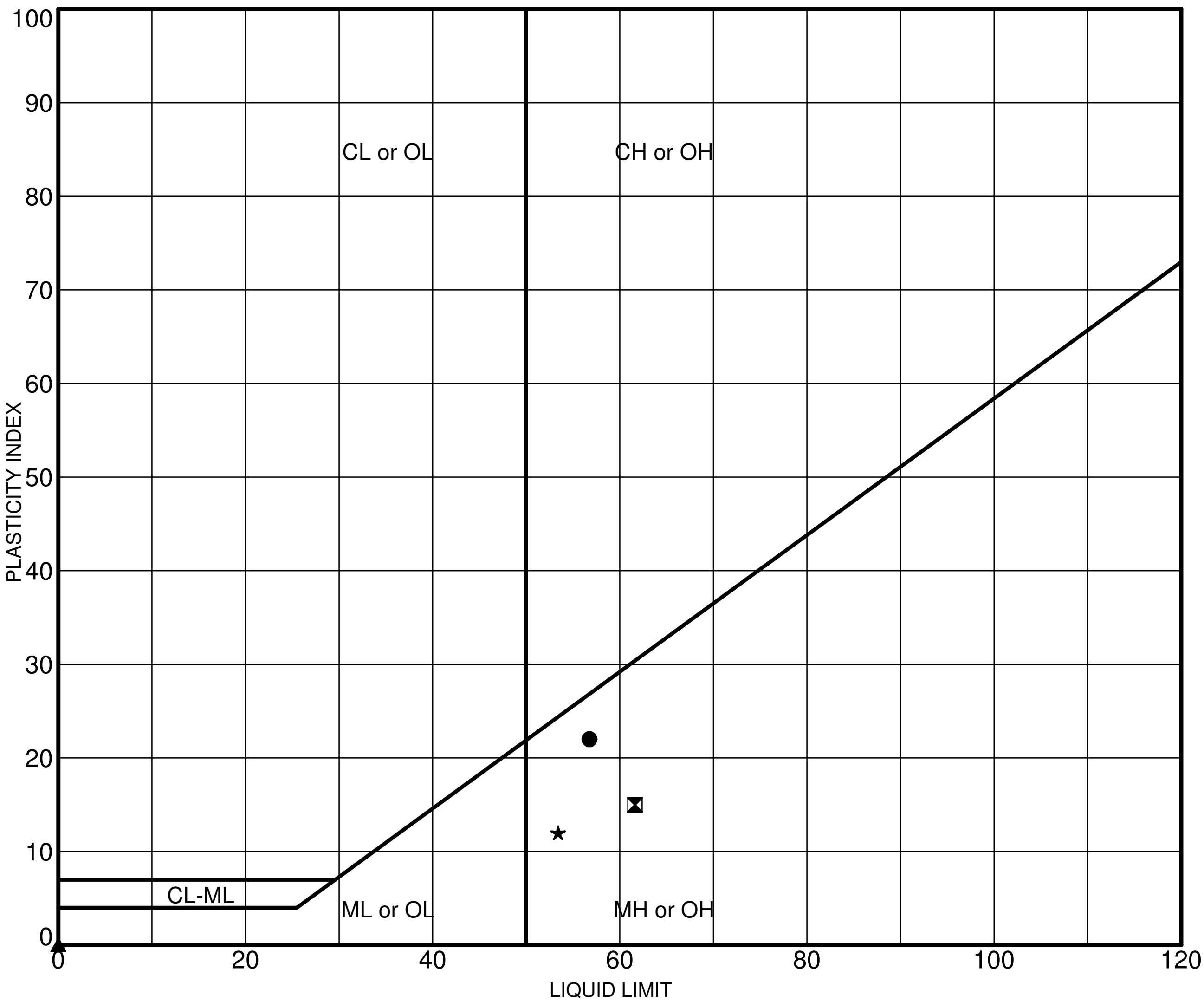
Four Consolidation tests (ASTM D 2435) were performed on selected soil samples to evaluate the compressibility characteristics of the soils. The tests were performed on the relatively undisturbed on-site soils. The test results and consolidation curves are presented on Plates B-4 through B-7.

One Modified Proctor compaction test (ASTM D 1557) was performed on a bulk sample of the near-surface soils to determine the dry density and moisture content relationship. Test results are presented on Plate B-8.

One California Bearing Ratio (CBR) test (ASTM D 1883) was performed on a bulk sample of the near-surface soils to evaluate the strength characteristics for pavement subgrade support. CBR test results are presented on Plate B-9.

Three sets of corrosion tests, including pH (AASHTO T 289) and Minimum Resistivity (ASTM G 57), were performed on selected soil samples obtained from our field exploration. The test results are summarized on Plate B-10.

Two one-inch Ring Swell tests (ASTM D 4546) were performed on selected soil samples to evaluate the swelling potential of the in-situ soils. The test results are summarized on Plate B-11.



	Sample	Depth (ft)	LL	PL	PI	Description
●	B-1	2.5-4.0	57	35	22	Reddish brown clayey silt (MH)
⊠	B-1	29.5-31.0	62	47	15	Reddish brown clayey silt (MH) w/ traces of sand
▲	B-4	19.5-21.0	NP	NP	NP	Reddish brown clayey silt (Non-Plastic) with fine sand
★	B-5	65.5-67.0	53	41	12	Reddish brown clayey silt with sand (MH)

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GEOTECHNICAL ENGINEERING

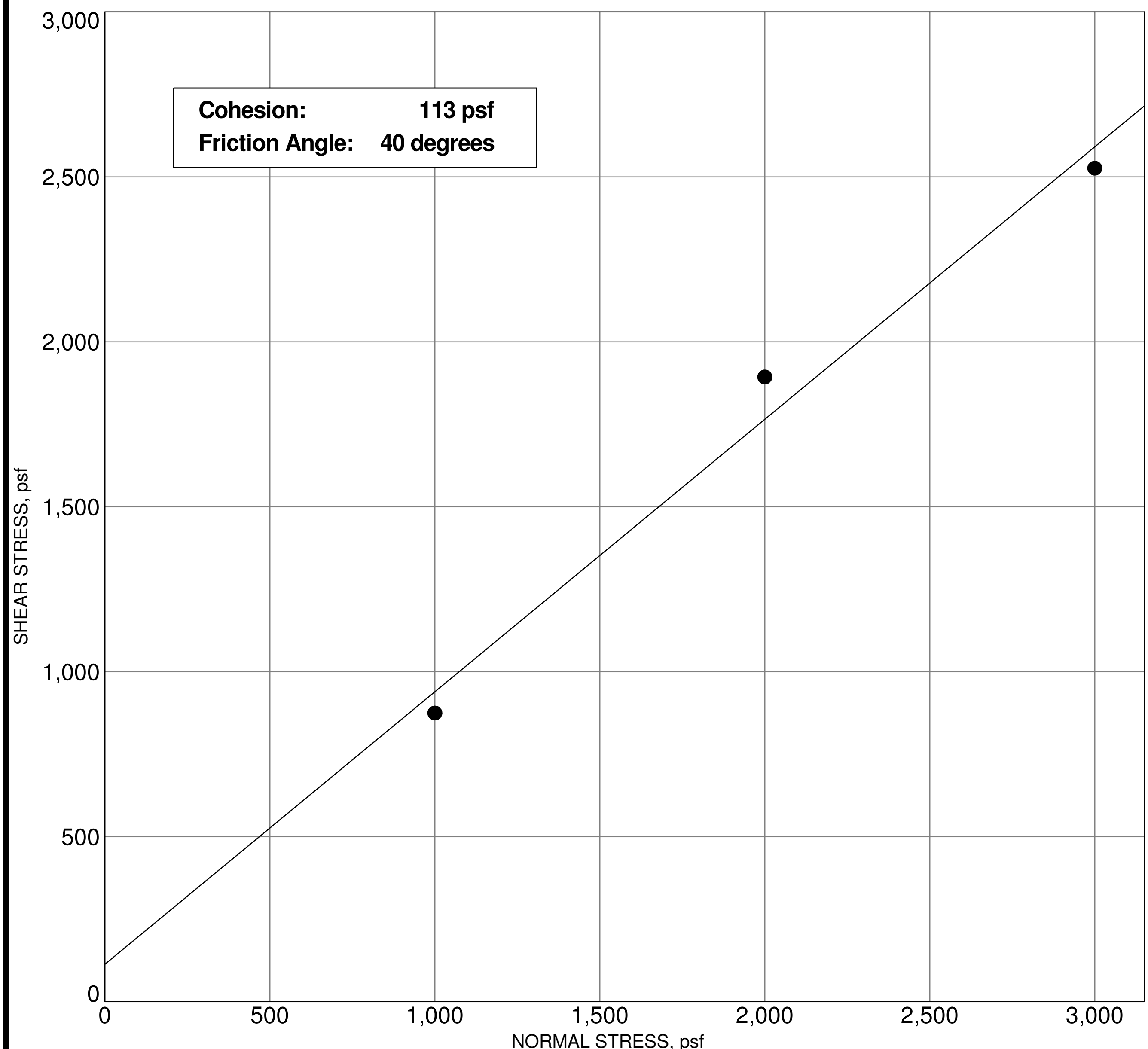
W.O. 5988-00&10

ATTERBERG LIMITS TEST RESULTS - ASTM D 4318

KAPAHI 1.0 MG (313 FEET) STORAGE TANK
WAIALUA-KAPAA WATER SYSTEM
KAPAA, KAUAI, HAWAII

Plate
B - 1

G_ATTERBERG 5988-00.GPJ GEOLABS.GDT 9/30/11



		Sample #1	Sample #2	Sample #3
INITIAL	Moisture Content, %	38.2	28.9	38.5
	Dry Density, pcf	82.9	88.3	79.3
	Height, inches	1.00	1.00	1.00
FINAL	Moisture Content, %	46.8	43.0	47.4
	Dry Density, pcf	81.1	88.8	80.4
	Height, inches	1.022	0.994	0.986
	Diameter, inches	2.42	2.42	2.42
	Deformation Rate, inch/minute	0.0024	0.0024	0.0022
	Normal Stress, psf	**1000**	**2000**	**3000**
	Peak Shear Stress, psf	**875**	**1893**	**2527**
	Shear Displacement, inches	0.21	0.20	0.25

Sample: B-1
Depth: 4.5 - 6.0 feet
Description: Reddish brown clayey silt with little sand and gravel

G DIRECT SHEAR 5988-00.GPJ GEOLABS.GDT 9/30/11

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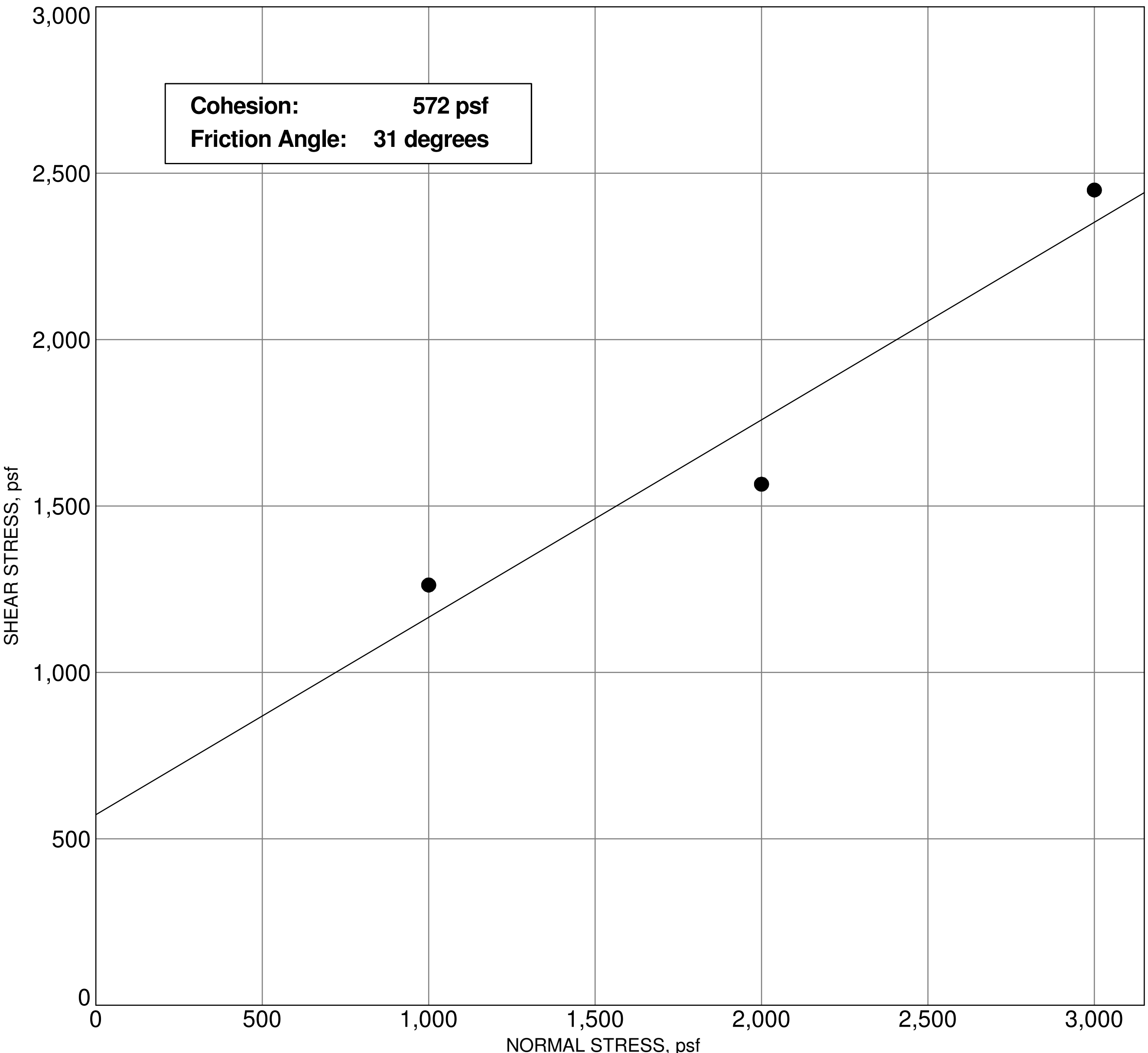
W.O. 5988-00&10

DIRECT SHEAR TEST - ASTM D 3080

KAPAHI 1.0 MG (313 FEET) STORAGE TANK
WAIALUA-KAPAA WATER SYSTEM
KAPAA, KAUAI, HAWAII

Plate
B - 2



			Sample #1	Sample #2	Sample #3
INITIAL	Moisture Content, %		55.6	57.9	54.1
	Dry Density, pcf		65.8	64.1	68.2
	Height, inches		1.00	1.00	1.00
FINAL	Moisture Content, %		63.6	63.2	57.2
	Dry Density, pcf		65.3	65.8	70.5
	Height, inches		1.009	0.975	0.968
	Diameter, inches		2.42	2.42	2.42
	Deformation Rate, inch/minute		0.0020	0.0020	0.0019
	Normal Stress, psf		**1000**	**2000**	**3000**
	Peak Shear Stress, psf		**1263**	**1566**	**2449**
	Shear Displacement, inches		0.18	0.30	0.29

Sample: B-1
Depth: 14.5 - 16.0 feet
Description: Reddish brown clayey silt with little sand

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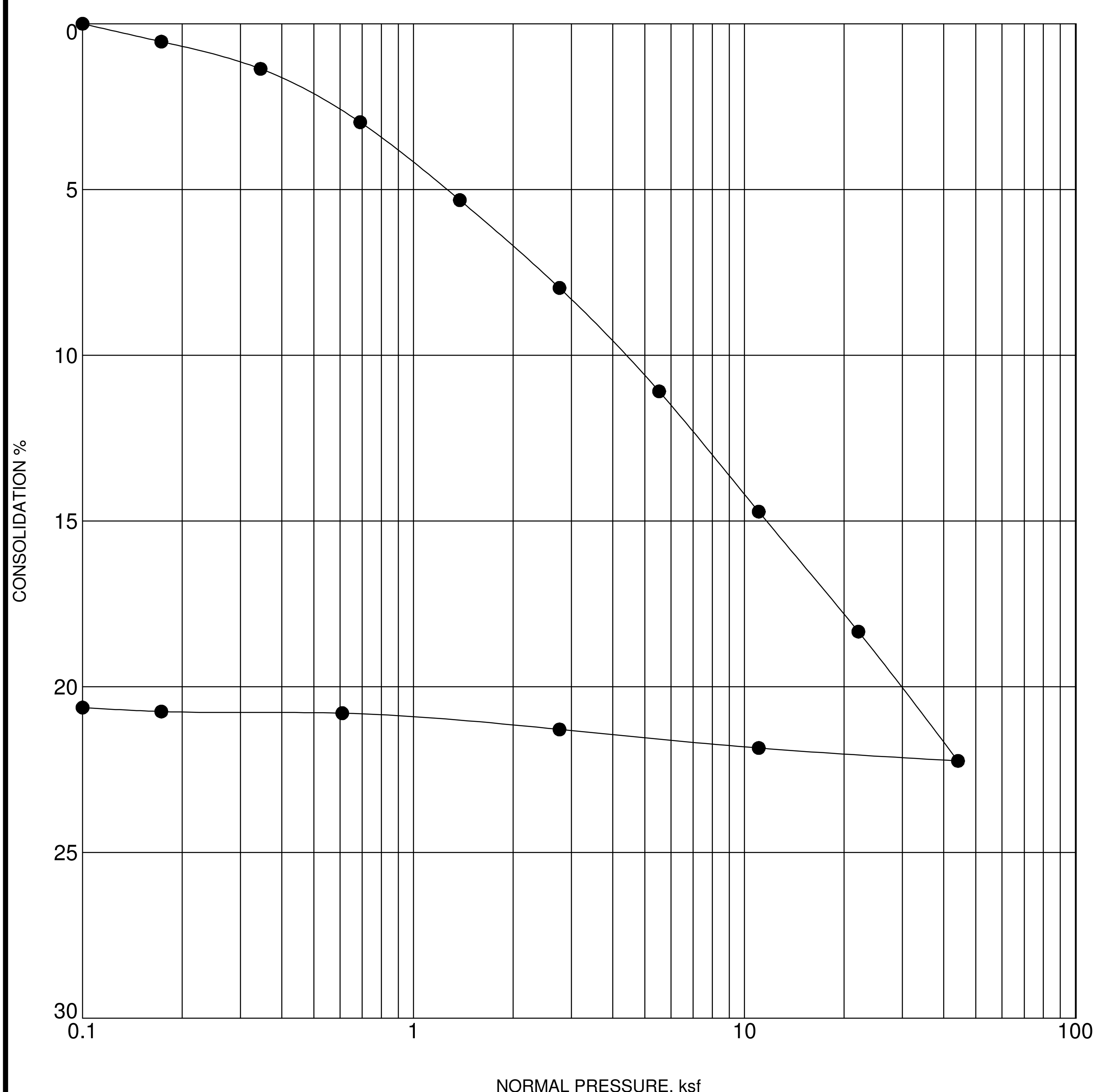
W.O. 5988-00&10

DIRECT SHEAR TEST - ASTM D 3080

KAPAHI 1.0 MG (313 FEET) STORAGE TANK
WAIALUA-KAPAA WATER SYSTEM
KAPAA, KAUAI, HAWAII

Plate
B - 3



Sample: B-1
Depth: 34.5 - 36.0 feet
Description: Reddish brown silty clay with traces of sand

Liquid Limit = N/A   Plasticity Index = N/A

	Initial	Final
Water Content, %	54.6	42.5
Dry Density, pcf:	71.3	89.8
Void Ratio	2.244	1.575
Degree of Saturation, %	90.1	100.0
Sample Height, inches	1.0000	0.7849

G_CONSOL 5988-00.GPJ GEOLABS.GDT 9/30/11

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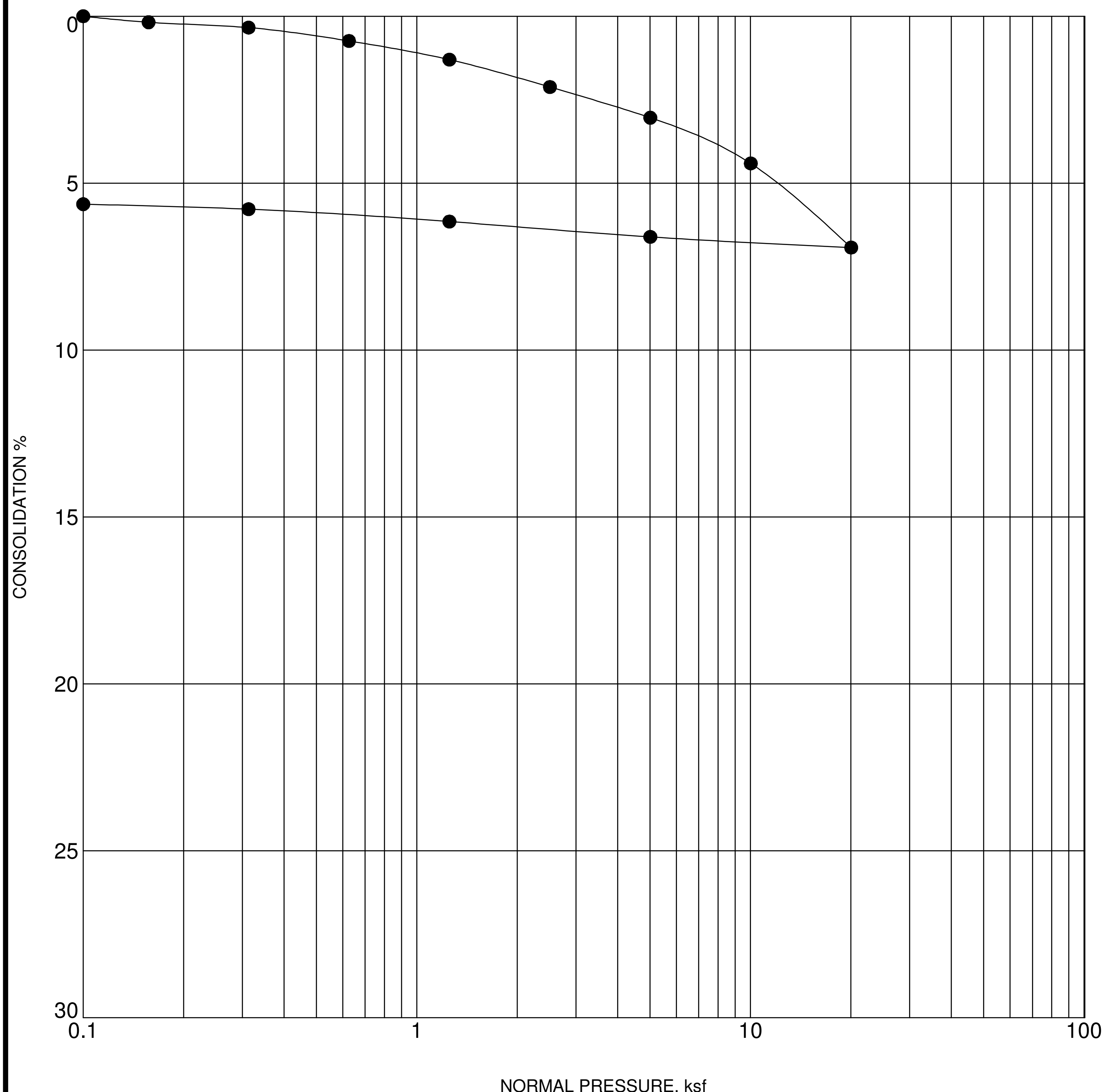
CONSOLIDATION TEST - ASTM D 2435

KAPAHI 1.0 MG (313 FEET) STORAGE TANK
WAIALUA-KAPAA WATER SYSTEM
KAPAA, KAUAI, HAWAII

Plate
B - 4



Sample: B-3
Depth: 34.5 - 36.0 feet
Description: Brown clayey fine sand with some highly decomposed rock

Liquid Limit = N/A   Plasticity Index = N/A

	Initial	Final
Water Content, %	53.0	51.0
Dry Density, pcf:	70.9	75.1
Void Ratio	1.743	1.589
Degree of Saturation, %	94.7	100.0
Sample Height, inches	1.0000	0.9376

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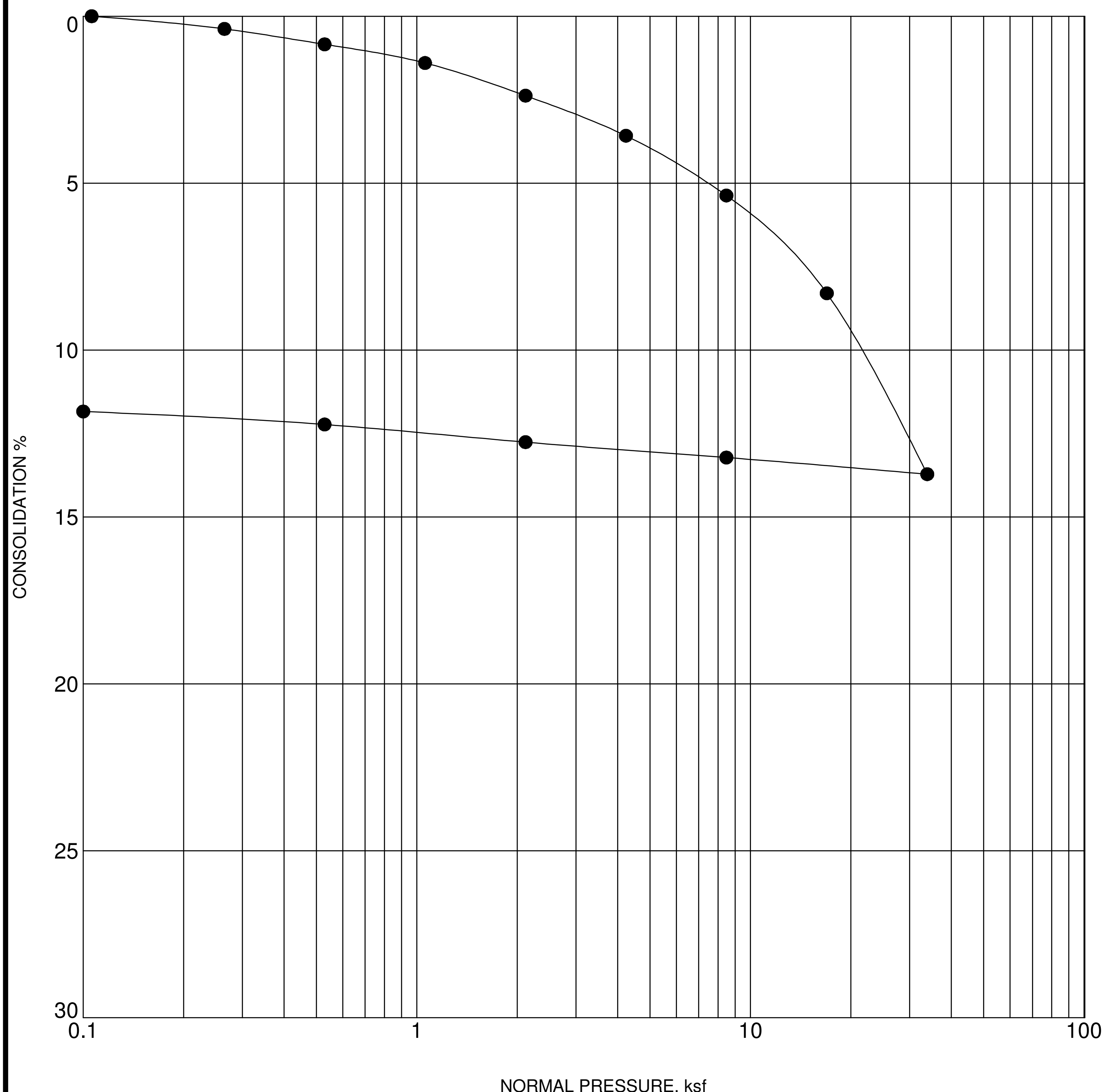
CONSOLIDATION TEST - ASTM D 2435

KAPAHI 1.0 MG (313 FEET) STORAGE TANK
WAIALUA-KAPAA WATER SYSTEM
KAPAA, KAUAI, HAWAII

Plate
B - 5

G_CONSOL 5988-00.GPJ GEOLABS.GDT 9/30/11



	Initial	Final
Water Content, %	57.8	52.7
Dry Density, pcf:	63.2	63.2
Void Ratio	1.144	1.144
Degree of Saturation, %	109.7	100.0
Sample Height, inches	1.0000	0.8823

Sample: B-4
Depth: 19.5 - 21.0 feet
Description: Reddish brown clayey silt (Non-Plastic) with fine sand

Liquid Limit = 0
Plasticity Index = 0

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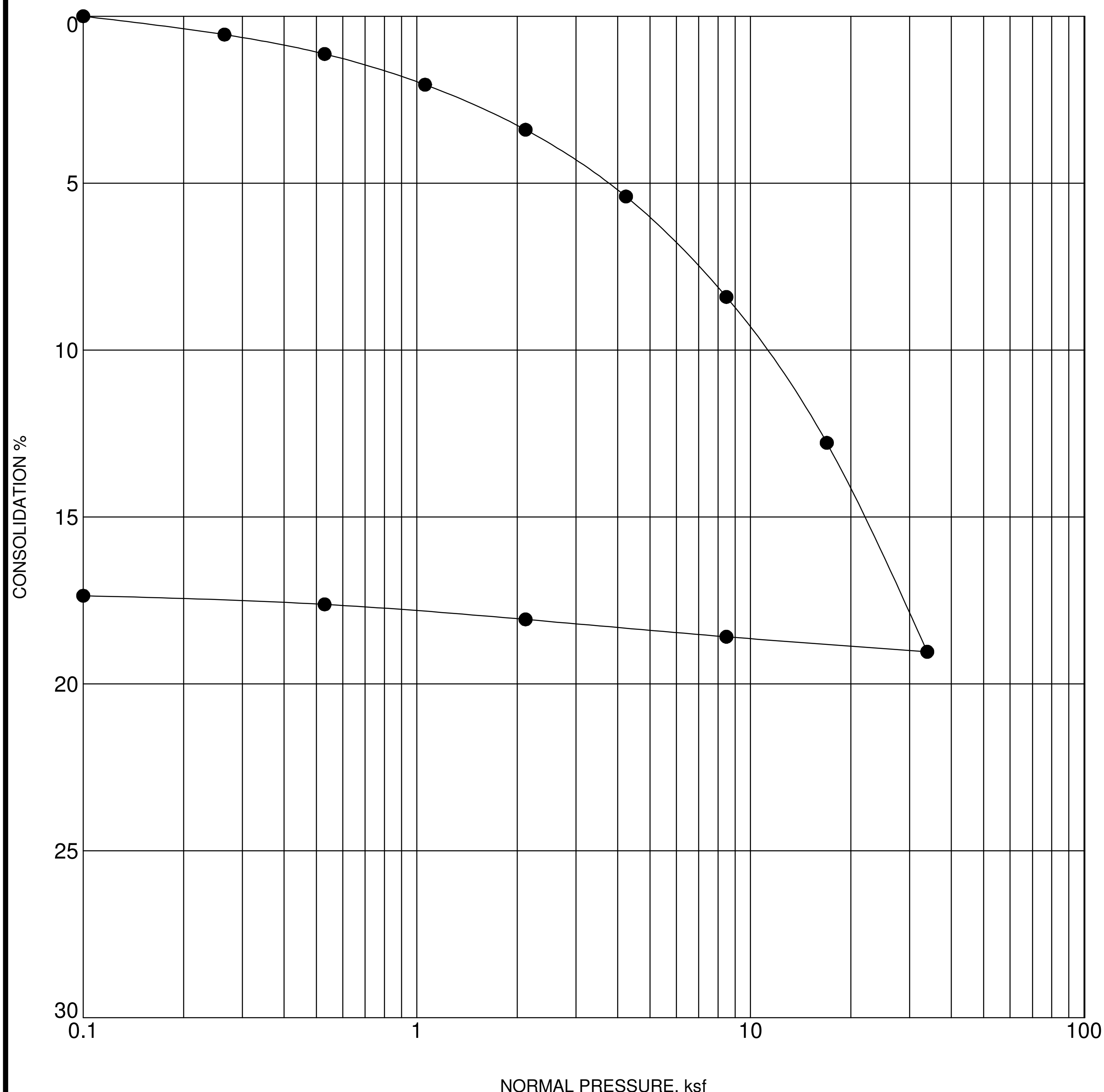
CONSOLIDATION TEST - ASTM D 2435

KAPAHI 1.0 MG (313 FEET) STORAGE TANK
WAIALUA-KAPAA WATER SYSTEM
KAPAA, KAUAI, HAWAII

Plate
B - 6

G_CONSOL 5988-00.GPJ GEOLABS.GDT 9/30/11



Sample: B-5
Depth: 65.5 - 67.0 feet
Description: Reddish brown clayey silt with sand (MH)

Liquid Limit = 53 Plasticity Index = 12

	Initial	Final
Water Content, %	68.9	52.0
Dry Density, pcf:	63.2	76.5
Void Ratio	2.339	1.759
Degree of Saturation, %	99.7	100.0
Sample Height, inches	1.0000	0.8206

G_CONSOL 5988-00.GPJ GEOLABS.GDT 9/30/11

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CONSOLIDATION TEST - ASTM D 2435

KAPAHI 1.0 MG (313 FEET) STORAGE TANK
WAIALUA-KAPAA WATER SYSTEM
KAPAA, KAUAI, HAWAII

Plate
B - 7

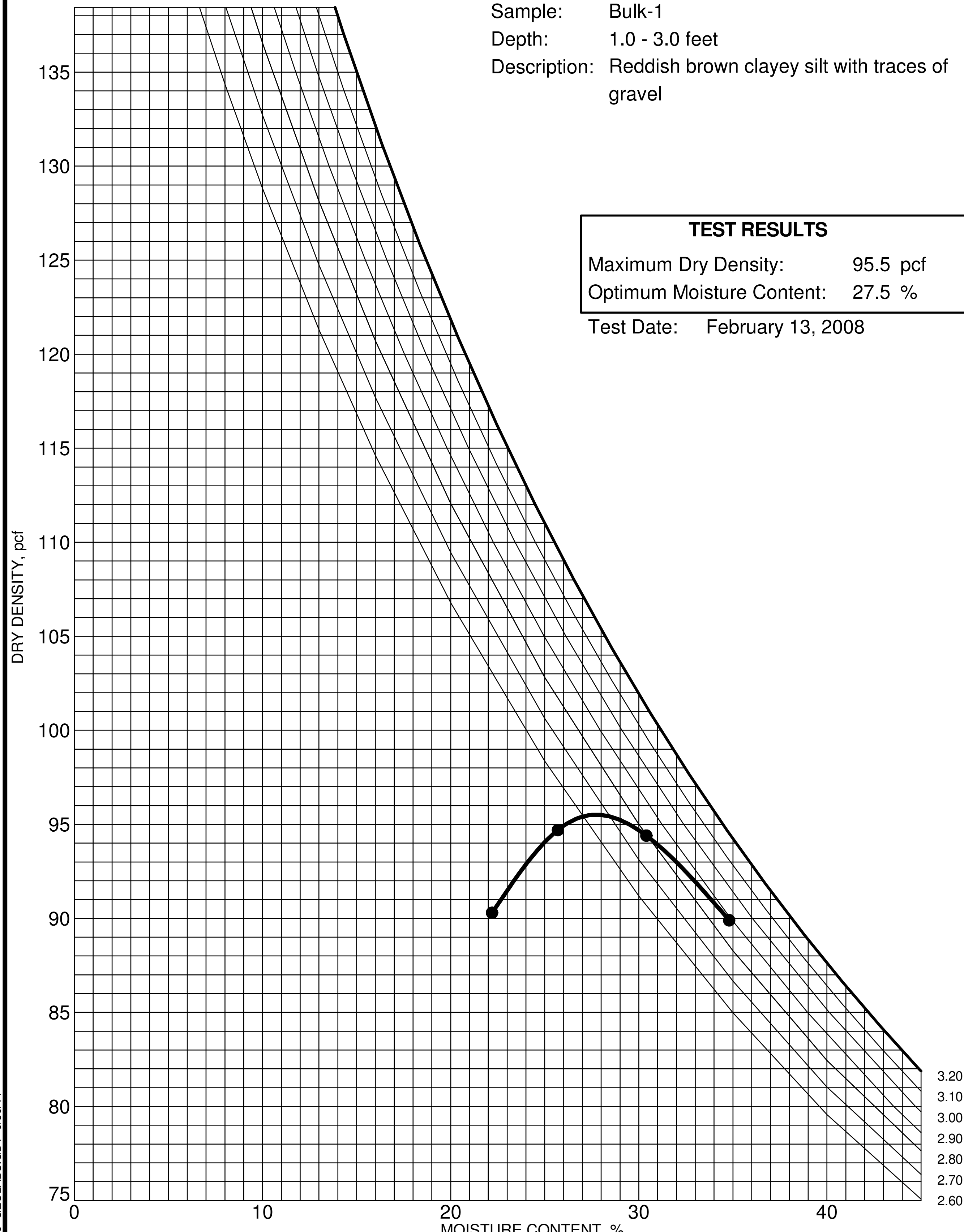
Sample: Bulk-1

Depth: 1.0 - 3.0 feet

Description: Reddish brown clayey silt with traces of gravel

TEST RESULTS	
Maximum Dry Density:	95.5 pcf
Optimum Moisture Content:	27.5 %

Test Date: February 13, 2008



G_COMPACTION 5988-00.GPJ GEOLABS.GDT 9/30/11

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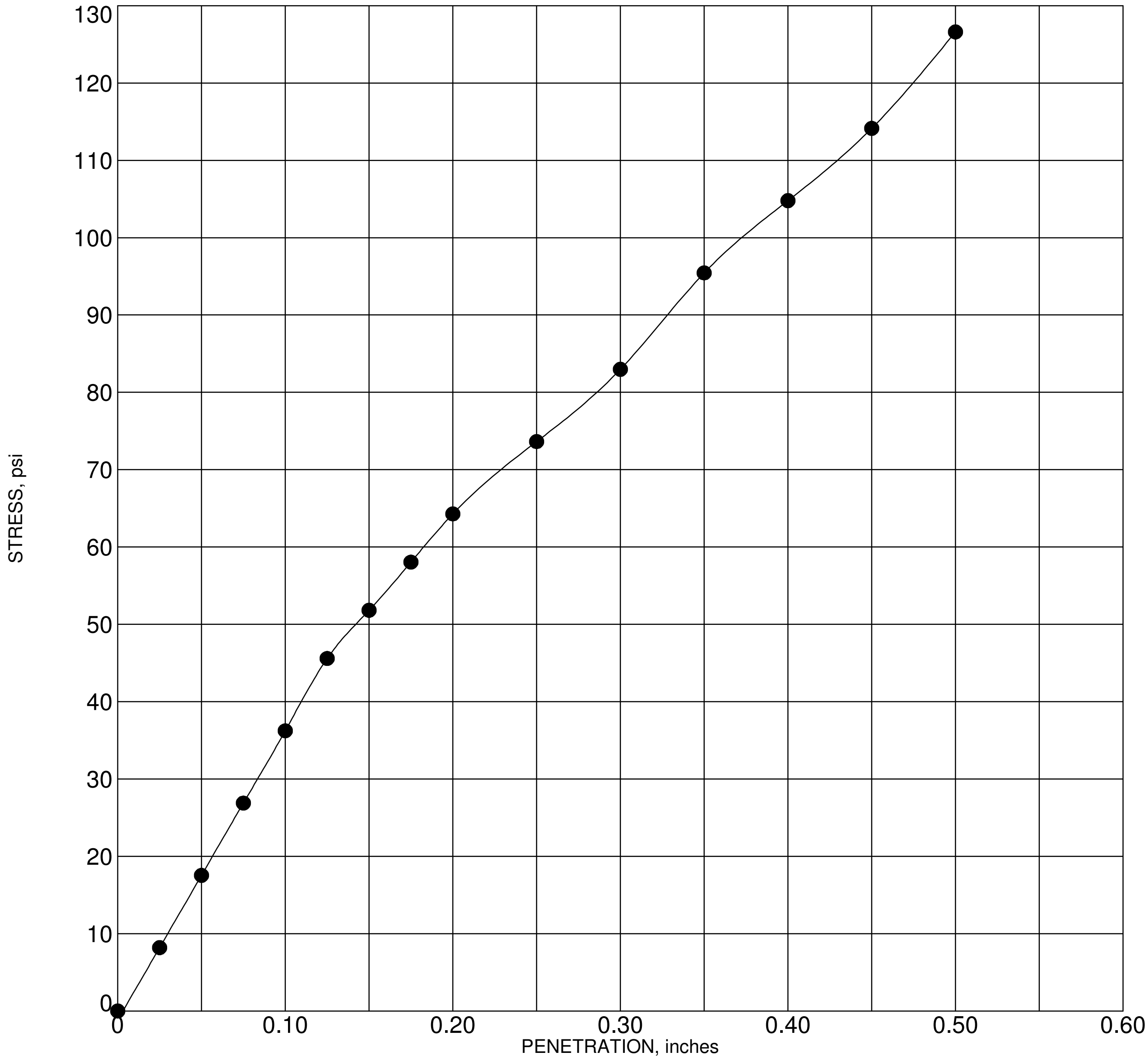
MOISTURE-DENSITY RELATIONSHIP - ASTM D 1557 A

KAPAHI 1.0 MG (313 FEET) STORAGE TANK
WAIALUA-KAPAA WATER SYSTEM
KAPAA, KAUAI, HAWAII

Plate B - 8



Corr. CBR @ 0.1"	3.7
Swell (%)	0.02

Sample: Bulk-1
Depth: 1.0 - 3.0 feet
Description: Reddish brown clayey silt with traces of gravel

Molding Dry Density (pcf)	88.2	Hammer Wt. (lbs)	10
Molding Moisture (%)	36.2	Hammer Drop (inches)	18
Days Soaked	5	No. of Blows	56
Aggregate	3/4 inch minus	No. of Layers	5

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CALIFORNIA BEARING RATIO - ASTM D 1883

KAPAHI 1.0 MG (313 FEET) STORAGE TANK
WAIALUA-KAPAA WATER SYSTEM
KAPAA, KAUAI, HAWAII

Plate
B - 9

G_CBR 5988-00.GPJ GEOLABS.GDT 9/30/11

Location	Depth	pH Value	Minimum Resistivity	Chloride Content	Sulfate Content
	(feet)		(ohm-cm)	(mg/kg)	(mg/kg)
B-2	9.5 - 11.0	6.08	10,000	-	-
B-3	9.5 - 11.0	5.91	8,400	-	-
B-5	20.5 - 22.1	5.33	4,800	-	-

TEST METHODS

pH Value	ASTM G51
Minimum Resistivity	ASTM G57
Chloride Content	EPA 300.0
Sulfate Content	EPA 300.0

ND: Not Detected Within Reporting Limits

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W.O. 5988-00&10

SUMMARY OF CORROSIVITY TESTS

KAPAHI 1.0 MG (313 FEET) STORAGE TANK
WAIALUA-KAPAA WATER SYSTEM
KAPAA, KAUAI, HAWAII

Plate
B - 10

G SUMMARY OF CORROSIVITY TESTS 5988-00.GPJ GEOLABS.GDT 9/30/11

Location	Depth (feet)	Soil Description	Dry Density (pcf)	Moisture Contents Initial (%)	Moisture Contents Air-Dried (%)	Moisture Contents Final (%)	Ring Swell (%)
B-1**	9.5 - 11.0	Reddish brown clayey silt	83.3	39	31	40	0.4
B-3**	9.5 - 11.0	Reddish brown silty clay	74.5	47	40	47	0.3

NOTE: Samples tested were either relatively undisturbed or remolded in 2.4-inch diameter by 1-inch high rings. They were air-dried overnight and then saturated for 24 hours under a surcharge pressure of 55 psf.

* Relatively Undisturbed

** Remolded

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W.O. 5988-00&10

SUMMARY OF RING SWELL TESTS

KAPAHI 1.0 MG (313 FEET) STORAGE TANK
WAIALUA-KAPAA WATER SYSTEM
KAPAA, KAUAI, HAWAII