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July 21, 2020
W.O. 14-3925

Mr. Michael Okamoto
R.M. Towill Corporation
2024 N. King Street, Suite 200
Honolulu, Hawaii 96819

Dear Mr. Okamoto:

Re: Addendum 1 to Geotechnical Investigation Report
Farrington Highway
Makaha Bridge No. 3 and No. 3A Replacement
Makaha, Oahu, Hawaii

We understand that based on the 500-year event, the contraction scour depth at Bridge 3A will be 19.28 feet and the abutment scour will be about 10.2 feet. Riprap countermeasures will be provided to mitigate the abutment scour; however, the drilled shafts supporting the bridge structure will need to design for the potential contraction scour.

Based on a contraction scour depth of 19.28 feet with the corresponding elevation of -18.28, we recommend that the drilled shaft at Bridge 3A abutments be increased to 108 feet in length for 330 kips at strength limit state.

For drilled shafts supporting the wingwalls at Bridge No. 3 and No. 3A, 3-foot diameter drilled shafts with a length of 72 feet may be designed to support an axial load of 210 kips at strength limit state.

We appreciate this opportunity to be of service. Should you have any questions concerning this addendum, please feel free to call on us.

Respectfully submitted,

HIRATA & ASSOCIATES, INC.

Con C. Truong, Project Engineer

CON C. TRUONG
LICENSED PROFESSIONAL ENGINEER
No. 9019-C
HAWAII, U.S.A.

This work was prepared by me
or under my supervision.
Expiration Date of License:
April 30, 2022

GEOTECHNICAL INVESTIGATION
FARRINGTON HIGHWAY
MAKAHA BRIDGE NO. 3 AND
NO. 3A REPLACEMENT
MAKAHA, OAHU, HAWAII

for

R. M. TOWILL CORPORATION

HIRATA & ASSOCIATES, INC.
W.O. 04-3925
July 2, 2020

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Geotechnical
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July 2, 2020
W.O. 04-3925

Mr. Michael Okamoto
R.M. Towill Corporation
2024 N. King Street, Suite 200
Honolulu, Hawaii 96819

Dear Mr. Okamoto:

Our report, "Geotechnical Investigation, Farrington Highway, Makaha Bridge No. 3 and No. 3A Replacement, Makaha, Oahu, Hawaii," dated July 2, 2020, our Work Order 04-3925 is enclosed. This investigation was conducted in general conformance with the scope of services presented in our proposal dated May 2, 2003.

Due to the heavy structural loads expected, drilled shafts are recommended for support of the proposed replacement bridges. The drilled shafts will derive most of their vertical load bearing capacity from friction between the shaft and the surrounding soil and coral/coral rubblestone. Based on the structural loads provided, drilled shaft lengths on the order of 80 and 90 feet will be required for Bridge No. 3 and a drilled shaft length on the order of 100 feet will be required for Bridge No. 3A. Additional recommendations for drilled shaft foundations, as well as geotechnical recommendations for the design of new pavement, the temporary detour road, and site grading are presented in this report.

We appreciate this opportunity to be of service. Should you have any questions concerning this report, please feel free to call on us.

Very truly yours,

HIRATA & ASSOCIATES, INC.

Jennifer H. Yamaguchi

Jennifer H. Yamaguchi President

JHY:CCT

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GEOTECHNICAL INVESTIGATION
FARRINGTON HIGHWAY
MAKAHA BRIDGE NO. 3 AND NO. 3A REPLACEMENT
MAKAHA, OAHU, HAWAII

INTRODUCTION

This report presents the results of our geotechnical investigation performed for the proposed Makaha Bridge No. 3 and No. 3A replacement project in Makaha, Oahu. Our scope of services for this study included the following:

- A visual reconnaissance of the site and its vicinity to observe existing conditions which may affect the project. The general location of the project site is shown on the enclosed Location Map, Plate A2.1.

- A review of available in-house soils information pertinent to the site and the proposed project.

- Drilling and sampling ten exploratory borings to depths ranging from about 23.5 to 110.5 feet. A description of our field investigation is summarized on Plates A1.1 and A1.2. The soils encountered are described on the Boring Logs, Plates A4.1 through A4.22. The approximate exploratory boring locations are shown on the enclosed Boring Location Plan, Plate A2.2.

- Laboratory testing of selected soil samples. Testing procedures are presented in Appendix B, Description of Laboratory Testing, Plates B1.1 and B1.2. Test results are presented in the Description of Laboratory Testing, on the Boring Logs (Plates A4.1 through A4.22), Consolidation Test reports (Plates B2.1 through B2.3), Direct Shear Test reports (Plates B3.1 through B3.7), Gradation Curves (Plates B4.1 through B4.4), Modified Proctor Curve (Plate B5.1), and R-Value Test reports (Plates B6.1 and B6.2).

- Engineering analyses of the field and laboratory data.

- Preparation of this report presenting geotechnical recommendations for design of the new bridges, temporary detour road, new highway pavement, and the relocated overhead and underground utilities.

PROJECT CONSIDERATIONS

The project will include replacing two existing timber bridge structures along Farrington Highway that span over Makaha Stream and West Makaha Stream.

Subsequence to our fieldwork and during the design stage, the layout of the replacement bridges were revised twice. Based on the new information obtained in relation to a flood event which occurred in late 2008, the most recent layout indicates that Bridge No. 3 will be a two-span concrete structure about 101 feet in length, while replacement Bridge No. 3A will be a single span concrete structure about 70 feet in length. Both bridges will have a minimum curb-to-curb width of about 47.5 feet. Vertical loads at the Bridge No. 3 abutments and center pier will be on the order of 1,709 and 1,900 kips, respectively, at strength limit states. Vertical loads at the Bridge No. 3A abutments will be on the order of 2,253 kips at strength limit states.

The abutments and center pier for Bridge No. 3 will each have 7 drilled shafts in a single row. The bottom of the footings are expected at approximate elevation -0.5. The abutments of Bridge No. 3A will also have 7 drilled shafts in a single row, with bottom of the footings at approximate elevation -4.

Bridge No. 3 will be protected from scour by a concrete lining at the channel bottom and grouted riprap at the side slopes. Bridge No. 3A will only have riprap protection at the abutments. Anticipated contraction scour is about 10.43 feet at the abutments.

A temporary detour road and stream crossing will need to route traffic and pedestrians around the site during construction. The detour road will extend along the Makai (western) side of the bridges. Finish grades for the temporary detour road were not available at the time of this report. However, based on the preliminary alignment, the detour road finish grades are expected to generally match the existing ground elevations. A temporary, prefabricated steel bridge is being considered for the temporary crossing of West Makaha Stream, while temporary culverts will be used for crossing Makaha Stream during the construction period. Temporary sheetpile walls may also be required to protect the detour road at the stream crossings.

The project will also include relocating overhead and underground utilities within the project limits.

SITE CONDITIONS

The project site is located along Farrington Highway, near its intersection with Kili Drive in Makaha, Hawaii. Existing Bridge No. 3 is situated approximately 150 feet south of Kili Drive, while Bridge No. 3A is located about 200 feet north of Kili Drive. Makaha Beach Park borders the site on the west.

The proposed replacement bridges will be located at the existing bridge locations. The existing bridges consist of wooden bridge decks supported on concrete abutment footings and center piers. Bridge No. 3 is about 50 feet in length and 25 feet wide. Bridge No. 3A is about 75 feet long and 25 feet wide. At the time of our field work, the stream bed below the bridges were partially dry and partially under about 1 to 2 feet of stagnant water. The dry stream bed exposed clayey silt, fine sand, and coral/coral rubblestone outcrops.

Except for the remnants of railroad abutments and supporting piers at the stream crossings, the site for the detour road is generally vacant of structures and is covered by a sparse growth of vegetation and kiawe trees.

SOIL CONDITIONS

Boring locations were selected based on the initial bridge layout. Borings B1 through B5, and B10, drilled along Farrington Highway for the replacement bridges encountered surface soils consisting of mottled brown to brown silty clay and clayey silt. The soils were in a stiff condition and generally mixed with sand, gravel, and cobbles. Boulders were also encountered within the surface soil layer in several borings.

Except for borings B3 and B10, the surface soil either transitioned to a soft to very soft condition at depths below groundwater level, or was underlain by a layer of very loose silty sand at depths ranging from about 6 to 11 feet. Underlying the very soft and loose soils were layers of clayey silt, sand, gravel, and coral rubblestone down to the maximum depths drilled. Coral rubblestone is defined as a mixture of partially cemented sand, silt, and gravel-sized coral fragments. The sand and coralline gravel (coral detritus) layers varied from a medium dense to dense condition with occasional loose pockets. The clayey silt stratum varied from a medium stiff to stiff condition with occasional very soft pockets, and the coral rubblestone was in a dense to medium hard condition.

Borings B3 and B10 encountered tan coral at depths of about 8 feet. The coral was in a medium hard to hard condition extending down to the maximum depths drilled.

Borings B6, B7, B8, and B9, drilled along the detour road alignment, encountered surface soil consisting of mottled tan to brown sand and silty sand. The sand and silty sand were in a medium dense condition and ranged from about 7 to 10 feet in thickness. Cobbles and boulders were also encountered in the sand and silty sand layers in borings B6, B8, and B9. Underlying the surface soils were layers of loose to medium dense silty sand and silty gravel, and dense to medium hard coral rubblestone extending down to the maximum depths drilled.

Groundwater was encountered in all our borings at depths ranging from about 8.7 to 11.6 feet below existing ground, corresponding to approximate elevations +3.8 to -0.6. The depth to groundwater can be expected to vary with tidal fluctuations and seasonal rainfall.

CONCLUSIONS AND RECOMMENDATIONS

Based on the most recent layout, borings were not drilled in the area of the proposed northern abutment (abutment no. 2) of Bridge No. 3. Although boring B2 was drilled to the south of the revised abutment location, and borings B3 and B10 were drilled to the north, the results of these borings indicated that there is a fairly significant discontinuity of the subsurface soil conditions between the borings to the north and south of the abutment.

Boring B2 encountered loose to medium dense silty sand from a depth of about 6 feet to about 43 feet. Underlying the loose to medium dense silty sand was a layer of very dense coralline gravel which gradually transitioned to a medium dense condition at deeper depths. In comparison, borings B3 and B10 encountered medium hard to hard coral at a depth of about 8 feet extending down to the maximum depths drilled. Although an additional boring at the location of the abutment was suggested, it was declined due to time constrains. As a result, subsurface conditions for design will need to be interpolated based on the borings drilled to the north and south of the abutment, in particular for lateral capacity analysis of the foundations. We recommend that prior to construction of the bridge foundations, a test boring be taken at the abutment location to confirm the design assumptions.

Based on the subsurface soil conditions encountered in our test borings and the heavy structural loads expected, drilled shaft foundations are recommended for support of the replacement bridges. The drilled shafts are intended to derive most of their load bearing capacity from friction between the shaft and the surrounding soil and coral/coral rubblestone.

Bridge Foundations

Recommendations are presented based on the use of 3-foot diameter drilled shafts. The drilled shafts should be spaced a minimum 2.5 shaft diameters apart, measured from center to center.

Axial Load Capacities

Drilled shafts will derive most of their load bearing capacity from friction between the shaft and the surrounding soils.

Based on borings B1 and B2, the upper 6 to 9 feet of silty sand immediately below the planned bottom of the Bridge No. 3 south abutment (abutment no. 1) and center pier footings was in a very loose condition. Analyses indicated that the very loose silty sand layer may be susceptible to liquefaction in an earthquake of magnitude 6 or higher. As a result, drilled shafts supporting the south abutment and center pier of Bridge No. 3 will need to account for potential downdrag on the drilled shafts in the event of an earthquake.

Based on the scour analyses, drilled shafts supporting Bridge No. 3A will need to account for potential lost of frictional resistance from the upper soil layer due to scouring.

The following are recommended axial capacities for the various design conditions. In determining the axial capacity of drilled shafts supporting the south abutment of Bridge No. 3, a soil profile similar to boring B2 was conservatively assumed.

Bridge No. 3					
		Strength Limit State		Extreme Event Limit State	
	Drilled Shaft Length	Compression	Uplift	Compression	Uplift
Abutments	80 ft	260 kips	170 kips	475 kips	380 kips
Pier	90 ft	300 kips	195 kips	540 kips	430 kips

Bridge No. 3A					
		Strength Limit State		Extreme Event Limit State	
	Drilled Shaft Length	Compression	Uplift	Compression	Uplift
Abutments	100 ft	330 kips	210 kips	660 kips	520 kips

The weight of the shaft was not included in determining the total uplift capacity of the drilled shaft. The project structural engineer should verify the structural capacity of the shaft member in tension.

Lateral Load Capacities

Lateral capacities of the drilled shafts will depend on the stiffness of the surrounding soil, the stiffness of the drilled shaft, the boundary condition at the top of the drilled shafts, and the acceptable horizontal displacement of the shafts. Results of lateral load analyses based on load combinations provided by the project structural engineer are presented on Plates C1.1 through C1.3. Since soil conditions at the north abutment of Bridge No. 3 is based on interpolation from adjacent borings, upper and lower bound limits of potential drilled shaft deflections, moments, and rotation were also presented for drilled shafts supporting the abutment.

Drilled Shaft Construction

Excavations for the drilled shafts can be expected to extend through very loose to medium dense silty sand and coralline gravel, medium stiff to stiff clayey silt, as well as very dense to medium hard coral rubblestone/coralline gravel and hard coral. As a result, relatively difficult drilling condition can be expected.

Rock drilling and coring equipment, as well as tools necessary for removal of the cored material, may be required for drilled shaft excavations extending into the very dense to medium hard coral rubblestone/coralline gravel and hard coral layers.

Due to the granular nature and very loose to medium dense condition of the coralline silty sand and gravel, potential significant sloughing of the sidewalls of the drilled shaft excavation can be expected. To reduce the potential of caving of the drilled shaft sidewalls, the use of deep temporary casing and/or drilling slurry will be required. To facilitate advancement of the casings through the various soil layers that vary from soft/loose to stiff/dense to hard, the temporary casings should be equipped with cutting teeth and installed by rotating or oscillating methods. Care should be exercised during removal of the temporary casing to reduce the potential for necking of the drilled shaft during concrete placement.

The bottom of the drilled hole should be cleaned prior to placement of concrete. The concrete should be placed as soon as practical upon completion of the drilled shaft excavations (within 24 hours), in order to prevent potential caving-in of the drilled shaft sidewalls. Concrete should be tremied through a pipe discharging below the surface of fresh concrete. Each drilled shaft should be poured in one continuous lift. Construction of cold joints should not be allowed.

Construction of adjacent drilled shafts within three drilled shaft diameters should not commence until 24 hours after the concrete placement.

Test Shafts

We recommend that at least one sacrificial 36-inch diameter trial shaft be constructed to determine the acceptability of the Contractor's equipment and procedures for drilled shaft construction. The test shaft should be extended to the same depth as the production shafts. Once the test shaft is accepted, the same type of equipment and procedures demonstrated in the test shaft program should be used for construction of production shafts.

Load testing is also recommended to confirm the capability of the drilled shafts to support the design loads. At least one static proof load test is recommended at each bridge site. The trial test shafts may be used for conducting the proof load test. Due to the relatively high load bearing capacity of the drilled shaft, load testing by conventional load test methods may not be practical. Bi-directional axial load tests utilizing the Osterberg Load Cell is therefore recommended. The test should be performed in general accordance with the Quick Load Test Method of ASTM D 1143. The load test shaft should be subjected to at least 120 percent of the recommended load bearing capacity for Extreme Event Limit State. The test shaft should not be used for the bridge structures after the proof load test.

Integrity Testing

Crosshole Sonic Logging (CSL) tests should be performed on all production drilled shafts as part of the quality control for drilled shaft construction. The downhole CSL method is a non-destructive integrity test that is based on the propagation of sound waves through concrete to assess the homogeneity of the drilled shafts, and to determine the location of anomalies, if any, in the concrete. The test should be performed in general accordance with ASTM D 6760.

To facilitate the CSL testing, access tubes should be embedded into the drilled shaft to allow the CSL probes, designed for receiving and transmitting ultrasonic waves, to enter the shaft. For the 36-inch diameter drilled shafts, we recommend a minimum of 3 equally spaced (120 degrees apart) and parallel access tubes per drilled shaft. The access tubes should consist of standard steel pipe with a minimum inside diameter of 2 inches extending from the bottom of the drilled shaft reinforcing cage to at least 3 feet above the top of the drilled shaft. The couplings and bottom cap of the access tubes should be watertight. The joints constructed along the full length of the access tubes should not hinder the passage of the CSL probes. The tubes should be filled with potable water as soon as possible but no later than 4 hours after concrete placement. We also recommend that the top of the tubes be covered with removable caps to keep out debris which may obstruct the free passage of the CSL probes.

The CSL testing should be performed after the concrete of the drilled shaft has cured for at least 4 days. However, in order to reduce the potential for undesirable loss of ultrasonic energy due to de-bonding between the access tube and the surrounding concrete, we recommend that CSL tests be performed no later than 14 days after the concrete placement. The access tubes should be filled with grout of the same strength as the drilled shaft after completion of the CSL tests.

In the event anomalies are detected by CSL testing, coring of the drilled shaft may be required to further evaluate the integrity of the concrete in the drilled shaft.

Foundation Settlement

Settlement on the order of 1/2 to 3/4 inch were computed for the drilled shafts at Service Limit States. Differential settlement is expected to be less than half of the total settlement. Much of the settlement is expected to occur during construction, upon the initial application of loads.

Seismic Design

Based on our borings advanced for this study and our knowledge of the deep soil conditions in the area, the soil profile at the site may be classified as a Site Class E. Based on 2008 design criteria provided by the State of Hawaii - Department of Transportation, Highway Division, the project site will need to be designed based on a seismic acceleration coefficient of 0.18g.

Lateral Design

Lateral capacities of drilled shafts are presented in the *Bridge Foundations* sections of this report. Resistance to lateral loading may also be provided by passive earth pressure acting on the abutment and pier footings/drilled shaft cap and buried portions of spread footing foundations. Passive earth pressure above groundwater may be computed as an equivalent fluid having densities of 260 and 520 pounds per cubic foot for Strength Limit State and Extreme Event Limit State, respectively. Below groundwater, an equivalent fluid having densities of 135 and 270 pounds per cubic foot may be assumed for Strength Limit State and Extreme Event Limit State, respectively. Unless covered by pavement or concrete slabs, the upper 12 inches of soil should not be considered in computing lateral resistance.

The following equivalent fluid pressures may be used for static active earth pressure considerations:

	Non-restrained Condition (pcf)	Restrained/At-rest Condition (pcf)
Normal Conditions Above Groundwater	36	57
Saturated Conditions or Below Groundwater	80	90

The recommended earth pressures assume that Type A Structural Backfill Material (Hawaii Standard Specifications for Road, Bridge, and Public Works Construction, Section 703.20) or granular structural fill specified in this report will be used to backfill above water level behind the retaining structures.

For dynamic lateral earth pressure considerations, a dynamic lateral force of $5.5H^2$ pounds per lineal foot of wall length may be used for conditions where walls are free to translate up to 2 to 3 inches or rotate. For walls that are restricted to lesser movement of less than 0.5 inches, a dynamic lateral earth force of $16H^2$ is recommended. H is the height of retained soil or backfill in feet. The dynamic lateral force may be assumed to act through the mid-height of the wall. The dynamic lateral earth forces are in addition to the static earth pressures.

An abutment backfill stiffness of 4 ksf per inch of deflection may be assumed for resistance of lateral loads in the longitudinal direction during seismic event. To reduce the potential for shear failure in the abutment fill, lateral deflection of the abutment soil should be limited to no more than 1.25 inches.

To prevent buildup of hydrostatic pressures, retaining structures above water level should be well-drained. Standard practice consists of placing a minimum 12-inch thick layer of free-draining gravel at the back of the wall. The gravel should extend from the base of the wall, around subdrains and/or weepholes, and up to within 12 inches of finish grade.

Alternatively, prefabricated drainage geocomposites, such as Miradrain or J-drain, may be used in lieu of the free-draining gravel. As with the free-draining gravel, the drainage geocomposites should be placed at the back of the wall, be connected with the weepholes and/or subdrains (in accordance with manufacturers specifications), and extend to within 12 inches of finish grade. For freestanding walls, the drainage system should be covered by at least 12 inches of compacted, low permeability soil, such as the onsite clayey silt.

Abutment End Walls

We understand that the abutment end walls may be about 10 to 20 feet in length and that the bottom of the wall will step up higher further behind the abutment. Due to the relative loose condition of the silty sand near the abutment footing elevations, we recommend that the abutment end walls be designed as a simple supported structure, resting on the abutment footing at one end and supported by spread footings at the opposite end, as the bottom of the wall steps up higher in elevation. Spread footings situated at elevations of +5 and above may be designed with a bearing value of 3,000 pounds per square foot under Service Limit State, 5,400 pounds per square foot under Strength Limit State, and 9,000 pounds per square foot under Extreme Event Limit State.

Alternatively, the end walls may also be supported on drilled shafts.

For lateral earth pressure considerations, recommendations presented in the *Lateral Design* sections of this report may be used for design of the end walls.

Bridge Approach Slabs

If approach slabs behind the bridge abutments are required, we recommend that the slabs be at least 15 feet in length. The slabs should be underlain by at least 6 inches of aggregate base course. The base course and subgrade should be compacted to a minimum 95 percent compaction as determined by AASHTO T-180 (ASTM D 1557). The approach slabs are expected to be founded on the stiff/dense surface soils. A bearing value of 3,000 pounds per square foot under Service Limit State, 5,400 pounds per square foot under Strength Limit State, and 9,000 pounds per square foot under Extreme Event Limit State may be assumed for design of the approach slabs.

Pipe Support, Trench Excavation and Backfill

Based on our exploratory borings, we believe that utility trench excavations into the surface silty clay, clayey silt, sand, silty sand, and dense coral rubblestone can generally be accomplished using conventional excavating equipment. However, confined excavations into medium hard to hard sections of coral rubblestone and coral will probably require pneumatic equipment.

In open trench excavations above groundwater, the onsite soils are expected to stand for temporary conditions at slopes of 1H:1V or flatter. However, localized sloughing should be expected where pockets of granular material or wet soil are encountered. Due to the granular nature of the onsite sand and silty sand, we anticipate that excavations below groundwater will require shoring. Dewatering will likely be necessary for placement of the utilities and backfill below groundwater. The Contractor's dewatering plan should address the potential effect of dewatering on adjacent structures.

Conventional crushed rock cradles and pipe bedding may be used for the support of underground utility lines. Should the invert of the pipe extend below groundwater level and expose the loose silty sand, the trench bottom should be overexcavated to a maximum depth of 24 inches and replaced with crushed rock. Prior to placing the crushed rock, geotextile fabric should be placed at the bottom of trench excavation, and should envelope the crushed rock and the pipe bedding material.

The pipe bedding material should also be placed on the sides of the pipe and up to a minimum 12 inches above the utility line or 12 inches above groundwater. Trench backfill above the pipe bedding material (12 inches above the pipe) may consist of onsite soils. Backfill should be placed in 8-inch loose lifts and compacted to a minimum 90 percent compaction as determined by AASHTO T-180 (ASTM D 1557).

In roadway areas, the upper 12 inches of backfill should be compacted to a minimum 95 percent compaction as determined by AASHTO T-180 (ASTM D 1557).

Pavement Design

Pavement recommendations for reconstruction of the roadway will be provided in a separate Pavement Justification Report for this project.

For the temporary detour road, a pavement section consisting of 3 inches of asphaltic concrete over 8 inches of aggregate base course is recommended. The subgrade and aggregate base course should be compacted to a minimum of 95 percent compaction as determined by AASHTO T-180 (ASTM D 1557). The recommended pavement section assumes that the detour road will be in use for about 10 months.

Temporary Detour Road

Foundations for Temporary Prefabricated Steel Bridge

Conventional spread footings founded on the stiff/dense surface soils may be used to support the prefabricated steel bridge. A bearing value of 3,000 pounds per square foot under Service Limit State, 5,400 pounds per square foot under Strength Limit State, and 9,000 pounds per square foot under Extreme Event Limit State may be used for design of the footings.

Foundations should be embedded at least 18 inches below finish adjacent grade. In addition, the footings should be embedded such that a minimum horizontal distance of 6 feet is maintained between the bottom edge of footing and slope face.

The bottom of all footing excavations should be cleaned of loose material and, where applicable, thoroughly tamped prior to placement of reinforcing steel and concrete.
Resistance to lateral loading may be provided by friction acting at the base of foundations and by passive earth pressure acting on the buried portions of foundations. Coefficients of friction of 0.46, and 0.58 may be used with the dead load forces to compute the friction acting at the base of foundations for Strength Limit State and Extreme Event Limit State, respectively.

Passive earth pressure for level ground conditions may be computed as an equivalent fluid having densities of 260 and 520 pounds per cubic foot for Strength Limit State and Extreme Event Limit State, respectively. For sloping ground conditions, an equivalent fluid having densities of 100 and 200 pounds per cubic foot for Strength Limit State and Extreme Event Limit State, respectively, may be assumed.

Temporary Culverts

Prior to placement of the culverts, all soft and loose soil at the stream bed should be removed down to a maximum depth of 36 inches and replaced with crushed rock. Prior to placing the crushed rock, geotextile fabric should be placed at the bottom of trench excavation, and should envelope the crushed rock material.

Temporary Sheetpiles

Temporary sheetpiles may be required to protect the detour road at the stream crossings. Borings drilled along the detour road alignment encountered boulders within the surface soil layer and dense to medium hard coral/coral rubblestone at depths ranging from about 8 to 12 feet below ground surface. As a result, predrilling will be required at all sheetpile locations to facilitate the sheetpile installation. The following earth pressures may be used for design of the sheetpile retaining walls.

	Above Water Level (pcf)	Below Water Level (pcf)
Active Earth Pressure	36	80
Passive Earth Pressure (Extreme Event Limit State)	520	290
Passive Earth Pressure (Strength Limit State)	260	145

For areal surcharges, a uniform pressure equal to 33 percent of the vertical surcharge pressure acting over the entire height of wall may be assumed in design. Additional analyses during design may be required to evaluate the surcharge effects of other loading conditions.

Site Grading

Site Preparation - The project site should be cleared of all vegetation, concrete slabs and footings, AC pavement, and other deleterious material. In areas requiring fill placement, the existing ground should first be scarified to a depth of six inches, moisture conditioned to about 2 percent above optimum moisture content, and compacted to a minimum 90 percent compaction as determined by AASHTO T-180 (ASTM D 1557).

Onsite Fill Material - The silty clay, clayey silt, sand, and silty sand may be reused as compacted fill and backfill, provided all rock and coral fragments larger than three inches in maximum dimension are removed. In addition, the silty clay and clayey silt soil should be moisture conditioned to about 2 percent above optimum moisture content during recompaction.

Imported Fill Material - Imported structural fill should be well-graded, non-expansive granular material. Specifications for imported structural fill should indicate a maximum particle size of 3 inches, and state that between 8 and 20 percent of soil by weight shall pass the #200 sieve. In addition, the plasticity index (P.I.) of that portion of the soil passing the #40 sieve shall not be greater than 10. Imported fill should also have a minimum CBR value of 20 and a CBR expansion potential no greater than 1.0 percent when tested in accordance with AASHTO T-193 (ASTM D 1883).

Backfill placed behind the abutment and retaining wall structures should consist of Structural Backfill Material A as indicated in Section 703.20 of the Hawaii Standard Specifications or imported granular structural fill as specified above.

Compaction - All fill placement should be in accordance with the Hawaii Standard Specifications for Road, Bridge, and Public Works Construction. Fill placed in areas which slope steeper than 5H:1V should be continually benched as the fill is brought up in lifts.

Structural Excavations - Based on our exploratory borings, we believe that excavations into the surface soils can be accomplished with conventional excavating equipment. Confined excavations into medium hard to hard sections of coral rubblestone and coral will probably require pneumatic equipment.

Temporary cuts into the existing surface soils should be stable at slope gradients of 1H:1V or flatter. Due to the granular nature of the onsite silty sand and gravel, excavations extending below groundwater level may require shoring. It should be the Contractor's responsibility to conform to all OSHA safety standards for excavations.

Construction Dewatering - Based on a bottom of abutment and pier footing/drilled shaft cap elevation of -4, temporary dewatering may be required for construction of the drilled shaft caps. Depending on the invert elevations of the underground utility lines, temporary dewatering may also be required for construction of the utility lines. Dewatering for construction is the responsibility of the contractor, and the selection and methods of dewatering should be left up to the contractor. However, the dewatering method selected should be designed to have minimal impact on the groundwater level surrounding the project site, and the contractor should address the potential for settlement of adjacent structures in his dewatering program.

The contractor should be made aware that the sand, silty sand, and coral materials anticipated at the bottom of excavations are permeable. If sand and silty sand are exposed at the bottom of excavations, care should also be exercised in the dewatering operations to prevent "quick" conditions (sand boil) or softening at the bottom of the excavations.

ADDITIONAL SERVICES

We recommend that we perform a general review of the final design plans and specifications. This will allow us to verify that the foundation design and earthwork recommendations have been properly interpreted and implemented in the design plans and construction specifications.

For continuity, we recommend that we be retained during construction to (1) observe the construction of drilled shafts, including all drilling and concrete placement operations, as well as proof load testing, (2) observe footing excavations prior to placement of reinforcing steel and concrete, (3) review and/or perform laboratory testing on import borrow to determine its acceptability for use in compacted fills, (4) observe structural fill placement and perform compaction testing, and (5) provide geotechnical consultation as required. Our services during construction will allow us to verify that our recommendations are properly interpreted and included in construction, and if necessary, to make modifications to those recommendations, thereby reducing construction delays in the event subsurface conditions differ from those anticipated.

LIMITATIONS

The boring logs indicate the approximate subsurface soil conditions encountered only at those times and locations where our borings were made, and may not represent conditions at other times and locations.

This report was prepared specifically for R. M. Towill Corporation and their sub-consultants for design of the proposed replacement of Makaha Bridges No. 3 and No. 3A in Makaha, Oahu. The boring logs, laboratory test results, and recommendations presented in this report are for design purposes only, and are not intended for use in developing cost estimates by the contractor.

During construction, should subsurface conditions differ from those encountered in our borings, we should be advised immediately in order to re-evaluate our recommendations, and to revise or verify them in writing before proceeding with construction.

Our recommendations and conclusions are based upon the site materials observed, the preliminary design information made available, the data obtained from our site exploration, our engineering analyses, and our experience and engineering judgement. The conclusions and recommendations in this report are professional opinions which we have strived to develop in a manner consistent with that level of care, skill, and competence ordinarily exercised by members of the profession in good standing, currently practicing under similar conditions in the same locality. We will be responsible for those recommendations and conclusions, but will not be responsible for the interpretation by others of the information developed. No warranty is made regarding the services performed, either express or implied.

Respectfully submitted,

HIRATA & ASSOCIATES, INC.

Con Truong, Project Engineer

CON C. TRUONG
LICENSED PROFESSIONAL ENGINEER
No. 9019-C
HAWAII, U.S.A.

This work was prepared by me or under my supervision
Expiration Date of License: April 30, 2022

APPENDIX A

FIELD INVESTIGATION

DESCRIPTION OF FIELD INVESTIGATION

GENERAL

The site was explored on July 15 and 16, 2004 and between May 24 and June 23, 2005, by performing a visual site reconnaissance and drilling 10 exploratory test borings to depths ranging from about 23.5 to 110.5 feet, with Mobile B40-L12 and Mobile B40-L22 truck-mounted drill rigs.

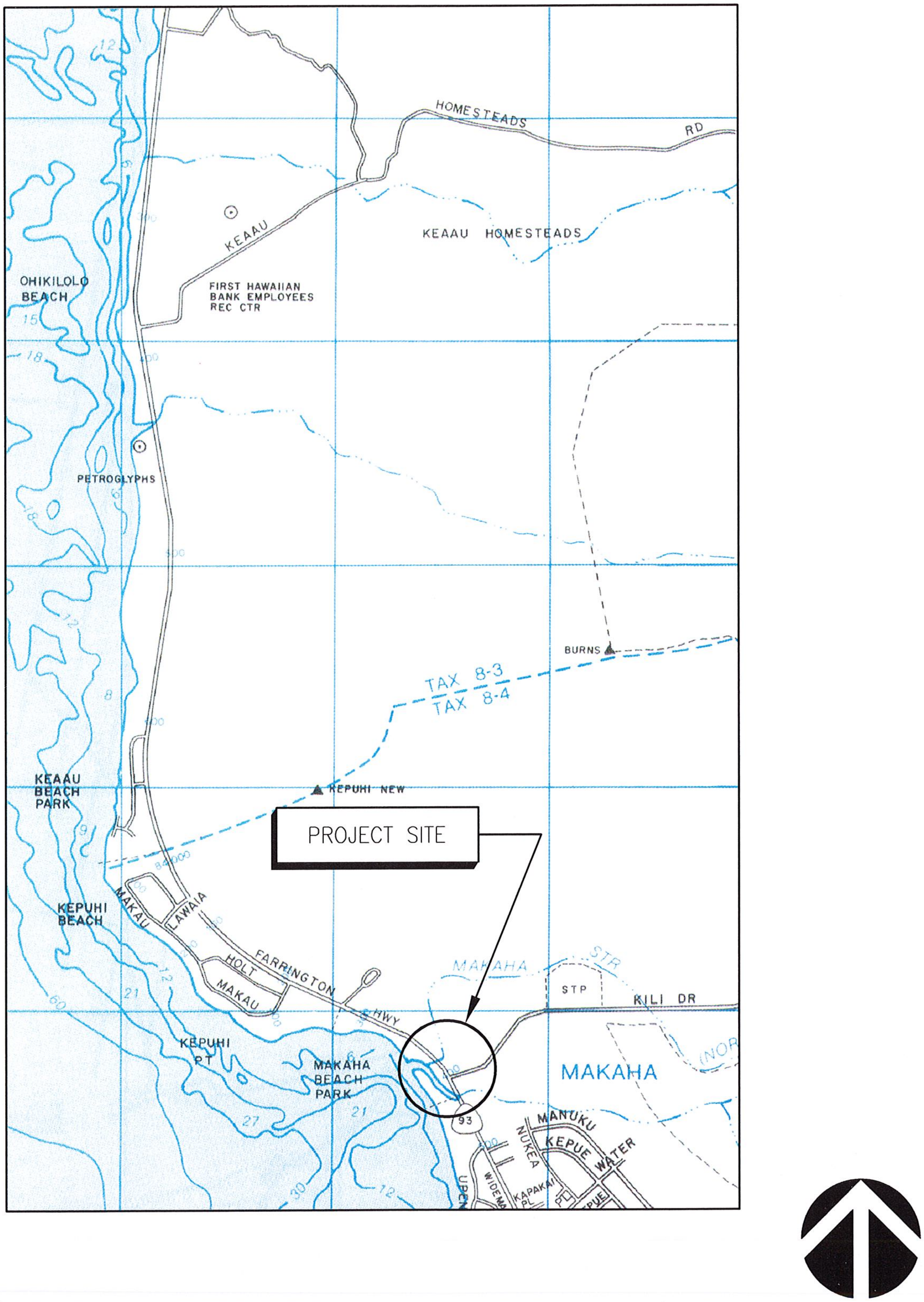
During drilling operations, the soils were continuously logged by our field engineers and classified by visual examination in accordance with the Unified Soil Classification System. The boring logs indicate the depths at which the soils or their characteristics change, although the change could actually be gradual. If the change occurred between sample locations, the depth was interpreted based on field observations. Classifications and sampling intervals are shown on the boring logs. A Boring Log Legend is presented on Plate A3.1, and the Unified Soil Classification System is shown on Plate A3.2. The soils encountered are logged on Plates A4.1 through A4.22.

Boring locations were located in the field by measuring/taping offsets from existing site features shown on the plans. The boring locations shown on Plate A2.2 are therefore approximate, in accordance with the field methods used. Ground surface elevations at boring locations were estimated using a topographic survey map provided by R. M. Towill Corporation.

SOIL SAMPLING

Representative soil samples and core samples of coral were recovered from the borings for selected laboratory testing and analyses. Representative samples were recovered by driving a 3-inch O.D. split tube sampler a total of 18 inches with a 140-pound hammer dropped from a height of 30 inches. The number of blows required to drive the sampler the final 12 inches are recorded at the appropriate depths on the boring logs, unless noted otherwise.

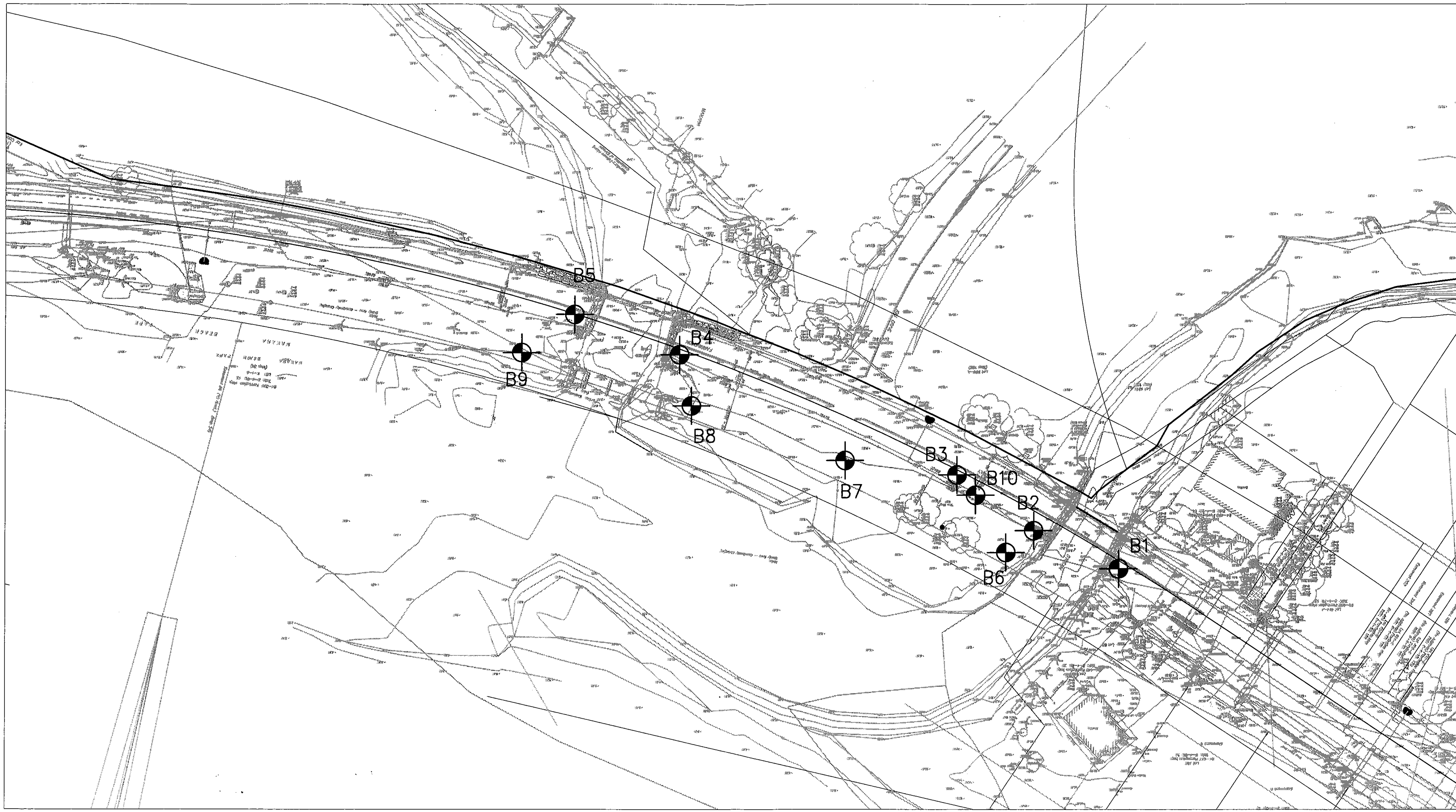
Core samples were obtained by drilling with an NX core barrel having an inside diameter of 2.1 inches. The depths and recovery percentages for each core run are shown on the enclosed Boring Logs.



Reference: Bryan's Sectional Maps, 2003 Edition
(Copyright J.R. Clere, used with permission)

W.O. 04-3925	Farrington Highway, Makaha Bridge No. 3 and No. 3A Replacement
Hirata & Associates, Inc.	LOCATION MAP

Plate A2.1



LEGEND:

Approximate location of borings

Reference: Site Plans provided by R. M. Towill Corporation on August 1, 2008.

GRAPHIC SCALE:
0 40 80 160FT.
1" = 80'

W.O. 04-3925	Farrington Highway, Makaha Bridge No. 3 and No. 3A Replacement
Hirata & Associates, Inc.	BORING LOCATION PLAN Plate A2.2

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
COARSE GRAINED SOILS (More than 50% of the material is LARGER than No. 200 sieve size.)	GRAVELS (More than 50% of coarse fraction is LARGER than the No. 4 sieve size.)	CLEAN GRAVELS (Little or no fines.)	GW	Well graded gravels, gravel-sand mixtures, little or no fines.
			GP	Poorly graded gravels or gravel-sand mixtures, little or no fines.
		GRAVELS WITH FINES (Appreciable amt. of fines.)	GM	Silty gravels, gravel-sand-silt mixtures.
			GC	Clayey gravels, gravel-sand-clay mixtures.
	SANDS (More than 50% of coarse fraction is SMALLER than the No. 4 sieve size.)	CLEAN SANDS (Little or no fines.)	SW	Well graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands or gravelly sands, little or no fines.
		SANDS WITH FINES (Appreciable amt. of fines.)	SM	Silty sands, sand-silt mixtures.
			SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS (More than 50% of the material is SMALLER than No. 200 sieve size.)	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 GREATER than 50.)		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS			PT	Peat and other highly organic soils.
				FRESH TO MODERATELY WEATHERED BASALT
				VOLCANIC TUFF / HIGHLY TO COMPLETELY WEATHERED BASALT
				CORAL

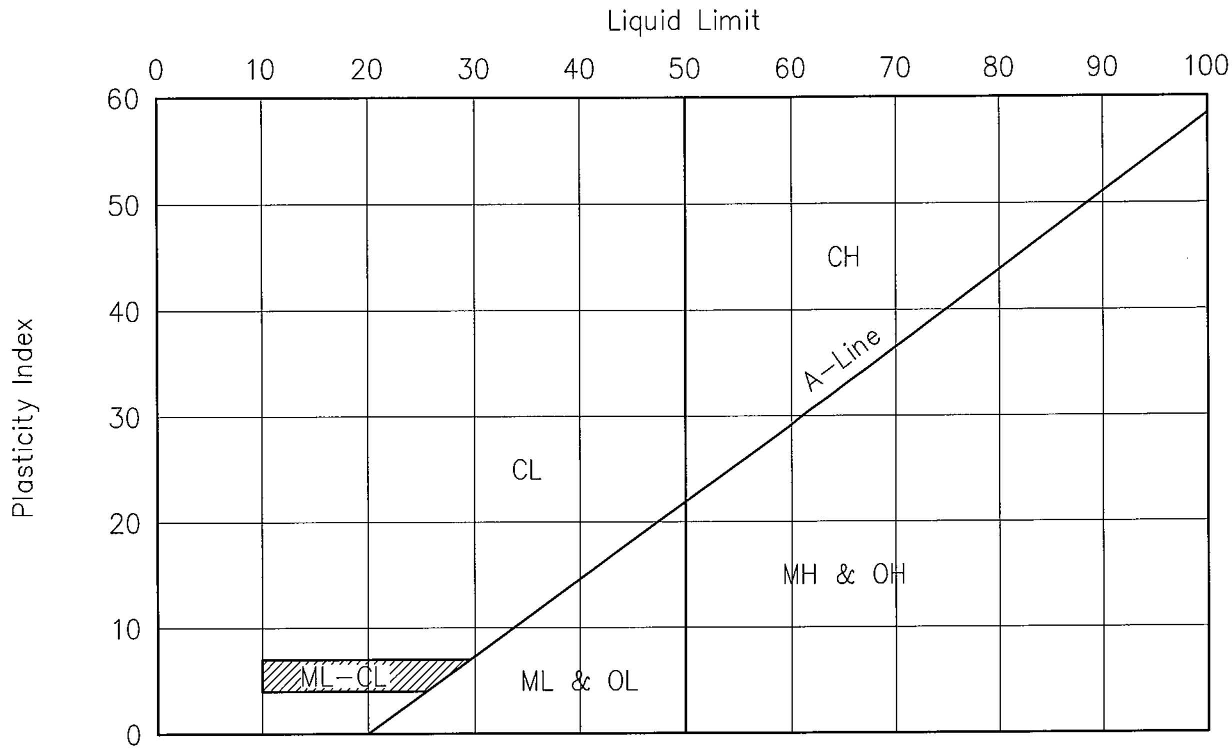
SAMPLE DEFINITION

2" O.D. Standard Split Spoon Sampler	Shelby Tube	RQD Rock Quality Designation
3" O.D. Split Tube Sampler	NX / 4" Coring	Water Level

W.O. 04-3925	Farrington Highway, Makaha Bridge No. 3 and No. 3A Replacement
Hirata & Associates, Inc.	BORING LOG LEGEND

Plate A3.1

PLASTICITY CHART



GRADATION CHART

COMPONENT DEFINITIONS BY GRADATION	
COMPONENT	SIZE RANGE
Boulders	Above 12 in.
Cobbles	3 in. to 12 in.
Gravel	3 in. to No. 4 (4.76 mm)
Coarse gravel	3 in. to 3/4 in.
Fine gravel	3/4 in. to No. 4 (4.76 mm)
Sand	No. 4 (4.76 mm) to No. 200 (0.074 mm)
Coarse sand	No. 4 (4.76 mm) to No. 10 (2.0 mm)
Medium sand	No. 10 (2.0 mm) to No. 40 (0.42 mm)
Fine sand	No. 40 (0.42 mm) to No. 200 (0.074 mm)
Silt and clay	Smaller than No. 200 (0.074 mm)

W.O. 04-3925	Farrington Highway, Makaha Bridge No. 3 and No. 3A Replacement
Hirata & Associates, Inc.	UNIFIED SOIL CLASSIFICATION SYSTEM Plate A3.2

HIRATA & ASSOCIATES, INC.

BORING LOG

W.O. 04-3925

BORING NO. B1 DRIVING WT. 140 lb. START DATE 5/24/05

SURFACE ELEV. 12.5±* DROP 30 in. END DATE 5/25/05

DEPTH	GRAPH	SAMPLE	BLOWS PER FOOT	DRY DENSITY (PCF)	MOIST. CONT. (%)	DESCRIPTION
0						Silty CLAY (CL) – Mottled brown, moist, stiff, with sand and gravel. Covered by 5 inches of asphaltic concrete over 7 inches of base material.
			20	93	29	
			14	85	27	
5			9	91	25	Medium stiff from 5 feet, with increase in sand content.
						Silty GRAVEL (GM) – Brown, medium dense, with sand and cobbles.
10			29	92	15	
						Silty SAND (SM) – Tan, very loose, with coral fragments.
15			6	66	41	
						(Begin wash–boring at 16 feet.)
20			4	62	42	
						Clayey SILT (MH) – Mottled dark brown, medium stiff.
			6/6"		45	
25			20	79	26	Silty SAND (SM) – Tan, medium dense, with coral fragments.
30			20	82	33	

HIRATA & ASSOCIATES, INC.

BORING LOG

W.O. 04-3925

BORING NO. B1 (continued) DRIVING WT. 140 lb. START DATE 5/24/05

SURFACE ELEV. 12.5± DROP 30 in. END DATE 5/25/05

DEPTH	GRAPH	SAMPLE	BLOWS PER FOOT	DRY DENSITY (PCF)	MOIST. CONT. (%)	DESCRIPTION
30						
35			29	97	21	
40			18	77	36	Gray color, with shell fragments at 39 feet.
45			26	95	20	Brown clayey silt at 44 feet.
50			56	92	15	Silty Coralline GRAVEL (GM) – Tan, dense, with sand.
55			85		18	
60			94	91	18	

HIRATA & ASSOCIATES, INC.

BORING LOG

W.O. 04-3925

BORING NO. B1 (continued) DRIVING WT. 140 lb. START DATE 5/24/05
SURFACE ELEV. 12.5± DROP 30 in. END DATE 5/25/05

DEPTH	GRAPH	SAMPLE	BLOWS PER FOOT	DRY DENSITY (PCF)	MOIST. CONT. (%)	DESCRIPTION
60						
64		☐	31	106	19	Medium dense at 64 feet.
65						
66						Coralline GRAVEL (GP-GM) – Brownish tan, medium dense, with silt and sand.
69		☐	29	86	29	
70						
74		☐	24		23	
75						
79		☐	12	79	28	Tan color at 79 feet, loose.
80						
83						Brown clayey silt seam at 83 feet.
84		☐	27	99	15	
85						
89		☐	34	116	12	End boring at 90.5 feet. Groundwater at 8.7 feet on 5/26/05. * Elevation based on topographic survey map provided by R. M. Towill Corporation.
90						

Plate A4.3

HIRATA & ASSOCIATES, INC.

BORING LOG

W.O. 04-3925

BORING NO. B2 DRIVING WT. 140 lb. START DATE 7/15/04

SURFACE ELEV. 12± DROP 30 in. END DATE 7/16/04

DEPTH	GRAPH	SAMPLE	BLOWS PER FOOT	DRY DENSITY (PCF)	MOIST. CONT. (%)	DESCRIPTION
0						Silty CLAY (CL) – Brown, moist, very stiff, with sand, gravel, and cobbles. Cobbles and boulder from 0.5 to 2 feet.
		☐	56	89	25	
		☐	96	105	19	Increase in gravel content at 4 feet.
5						
						Silty SAND (SM) – Yellowish tan, slightly moist, very loose, with coral fragments.
		☐	8	85	11	
10 ▽						
		☐	4	65	56	Increase in silt content from 13 to 18 feet.
15						
		☐	10	90	30	Loose from 18 feet. (Begin wash–boring from 18 feet.)
20						
		☐	15	81	42	Clayey at 23 feet.
25						
						Silty Coralline GRAVEL (GM) – Tan, dense, with sand.
		☐	74	119	12	
30						

HIRATA & ASSOCIATES, INC.

BORING LOG

W.O. 04-3925

BORING NO. B2 (continued) DRIVING WT. 140 lb. START DATE 7/15/04

SURFACE ELEV. 12± DROP 30 in. END DATE 7/16/04

DEPTH	GRAPH	SAMPLE	BLOWS PER FOOT	DRY DENSITY (PCF)	MOIST. CONT. (%)	DESCRIPTION
30						Clayey SAND (SC) – Grayish brown, loose, with coral fragments.
			15	69	49	
35						
			10	71	48	
40						
			48/6"	108	18	Silty Coralline GRAVEL (GM) – Tan, very dense, with sand.
45						
			62/6"	94	27	
50						Begin NX coring at 50 feet. 33% Recovery from 50 to 53 feet.
						End NX coring at 53 feet.
			26	No Recovery		Medium dense from 53 feet.
55						
			32	110	18	
60						

HIRATA & ASSOCIATES, INC.

BORING LOG

W.O. 04-3925

BORING NO. B2 (continued) DRIVING WT. 140 lb. START DATE 7/15/04
SURFACE ELEV. 12± DROP 30 in. END DATE 7/16/04

DEPTH	GRAPH	SAMPLE	BLOWS PER FOOT	DRY DENSITY (PCF)	MOIST. CONT. (%)	DESCRIPTION
60						
		☐	34	109	21	
65						
		☐	43	88	18	
70						
		☐	23	102	24	
75						
		☐	58	88	26	Dense at 78 to 83 feet.
80						
		☐	47	103	21	
85						
		☐	34	75	14	
90						

HIRATA & ASSOCIATES, INC.

BORING LOG

W.O. 04-3925

BORING NO. B2 (continued) DRIVING WT. 140 lb. START DATE 7/15/04

SURFACE ELEV. 12± DROP 30 in. END DATE 7/16/04

DEPTH	GRAPH	SAMPLE	BLOWS PER FOOT	DRY DENSITY (PCF)	MOIST. CONT. (%)	DESCRIPTION
90						
93		☐	26/6" 50/3"	100	27	Dense to very dense at 93 feet.
95						
98		☐	53	104	24	Clayey at 98 feet.
100						End boring at 99.5 feet.
						Groundwater at 10.2 feet on 7/16/04.
105						
110						
115						
120						

Plate A4.7

HIRATA & ASSOCIATES, INC.

BORING LOG

W.O. 04-3925

BORING NO. B3 | DRIVING WT. 140 lb. | START DATE 5/26/05

SURFACE ELEV. 13± | DROP 30 in. | END DATE 6/2/05

DEPTH	BLOWS PER FOOT	DRY DENSITY (PCF)	MOIST. CONT. (%)	DESCRIPTION
0				Clayey SILT (MH) – Dark brown, moist, stiff, with sand, gravel, and cobbles. Covered by 2 inches of asphaltic concrete over 2 inches of base material.
	27	103	19	
	33	95	21	
				Boulder at 4 feet.
5	52	99	10	Silty SAND (SM) – Mottled tan, slightly moist, medium dense to dense.
				Cobbles at 7 feet.
				CORAL – Mottled tan and gray, medium hard, fragmented.
	40/3"			
10				
	20/No Penetration			
15				
	50/3"			(Begin wash-boring at 19 feet) Begin NX coring at 19 feet. 100% Recovery from 19 to 24 feet.
20				
				70% Recovery from 24 to 29 feet.
25				
				Clayey SILT (MH) – Brown, stiff.
30				

HIRATA & ASSOCIATES, INC.

BORING LOG

W.O. 04-3925

BORING NO. B3 (continued) DRIVING WT. 140 lb. START DATE 5/26/05

SURFACE ELEV. 13± DROP 30 in. END DATE 6/2/05

DEPTH	GRAPH	SAMPLE	BLOWS PER FOOT	DRY DENSITY (PCF)	MOIST. CONT. (%)	DESCRIPTION
30						95% Recovery from 29 to 34 feet.
						CORAL RUBBLESTONE – Mottled tan, medium hard, highly fragmented, with layers of cemented sand.
						45% Recovery from 34 to 39 feet.
35						Cemented sand at 36 to 39 feet.
						Medium dense from 38 feet. 0% Recovery from 39 to 44 feet.
40						
			30/2" 10/No Penetration			End NX coring at 44 feet. Dense to medium hard from 44 feet.
45						
			20/No Penetration			
50						
			20/No Penetration			
55						Begin NX coring at 54 feet. 50% Recovery from 54 to 59 feet.
60						

HIRATA & ASSOCIATES, INC.

BORING LOG

W.O. 04-3925

BORING NO. B3 (continued) DRIVING WT. 140 lb. START DATE 5/26/05
SURFACE ELEV. 13± DROP 30 in. END DATE 6/2/05

DEPTH	GRAPH	SAMPLE	BLOWS PER FOOT	DRY DENSITY (PCF)	MOIST. CONT. (%)	DESCRIPTION
60						50% Recovery from 59 to 64 feet.
			53/6"	124	13	End NX coring at 64 feet.
65						
			20/No Penetration			Begin NX coring at 69 feet. 38% Recovery from 69 to 74 feet.
70						
						End NX coring at 74 feet.
75						
			37	83	17	Medium dense at 79 feet.
80						
			20/No Penetration			
						End boring at 84 feet.
85						Groundwater at 10 feet on 6/2/05.
90						

Plate A4.10

HIRATA & ASSOCIATES, INC.

BORING LOG

W.O. 04-3925

BORING NO. B4 DRIVING WT. 140 lb. START DATE 6/3/05

SURFACE ELEV. 12± DROP 30 in. END DATE 6/8/05

DEPTH	GRAPH	SAMPLE	BLOWS PER FOOT	DRY DENSITY (PCF)	MOIST. CONT. (%)	DESCRIPTION
0						Clayey SILT (MH) – Brown, moist, stiff, with sand, gravel, and cobbles. Covered by 3 inches of asphaltic concrete over 6 inches of base material.
			23	68	18	
			5	86	28	Soft at 3 feet.
5			26	105	21	
						Boulder at 7 feet.
10			12	83	35	Medium stiff from 9 feet.
15			45/6"	94	26	Silty SAND (SM) – Mottled gray, dense, slightly cemented. (Begin wash-boring at 14 feet.)
						CORAL RUBBLESTONE – Mottled tan and gray, medium hard, with layers of cemented sand.
20			30/2"			Begin NX coring at 19 feet. 63% Recovery from 19 to 24 feet.
25						40% Recovery from 24 to 29 feet. Dense to very dense from 24 feet.
30						Clayey SILT (MH) – Brown, stiff.

HIRATA & ASSOCIATES, INC.

BORING LOG

W.O. 04-3925

BORING NO. B4 (continued) DRIVING WT. 140 lb. START DATE 6/3/05

SURFACE ELEV. 12± DROP 30 in. END DATE 6/8/05

DEPTH	GRAPH	SAMPLE	BLOWS PER FOOT	DRY DENSITY (PCF)	MOIST. CONT. (%)	DESCRIPTION
30						40% Recovery from 29 to 34 feet.
			10	84	36	End NX coring at 34 feet. Medium stiff from 34 feet.
35						
			5	52	35	Soft at 39 feet.
40						
						Silty SAND (SM) – Mottled tan and gray, medium dense.
			25		20	
45						
			24	101	16	Dark gray clayey silt at 49 feet.
50						
			39	97	21	
55						
						Clayey SAND (SC) – Brown, medium dense, with weathered rock fragments.
			26	88	36	
60						

Plate A4.12

HIRATA & ASSOCIATES, INC.

BORING LOG

W.O. 04-3925

BORING NO. B4 (continued) DRIVING WT. 140 lb. START DATE 6/3/05
SURFACE ELEV. 12± DROP 30 in. END DATE 6/8/05

DEPTH	GRAPH	SAMPLE	BLOWS PER FOOT	DRY DENSITY (PCF)	MOIST. CONT. (%)	DESCRIPTION
60						
						Begin NX coring at 61.5 feet. 20% Recovery from 61.5 to 64 feet. Brown clayey silt from 62 to 64 feet. End NX coring at 64 feet.
65			27	88	27	
70			30	77	41	
75			47	66	59	Clayey SILT (MH) – Brown, stiff, with sand and weathered rock fragments.
80			63		35	
85			88	109	27	
90			49	85	39	End boring at 90.5 feet. Groundwater at 10.1 feet on 6/8/05.

Plate A4.13

HIRATA & ASSOCIATES, INC.

BORING LOG

W.O. 04-3925

BORING NO. B5 DRIVING WT. 140 lb. START DATE 6/8/05

SURFACE ELEV. 12.5± DROP 30 in. END DATE 6/21/05

DEPTH	GRAPH	SAMPLE	BLOWS PER FOOT	DRY DENSITY (PCF)	MOIST. CONT. (%)	DESCRIPTION
0						Clayey SILT (MH) – Brown, moist, stiff, with sand and gravel. Covered by 3 inches of asphaltic concrete over 13 inches of base material.
			27	104	15	
			10	88	32	Medium stiff from 3 feet.
5			12	96	21	
						Soft from 7 feet.
			3	86	37	
10						(Begin wash-boring at 10 feet.)
						CORAL RUBBLESTONE – Mottled tan and gray, medium hard.
15						
			10/2"			
20						SAND (SP-SM) – Mottled gray, medium dense, with silt. Begin NX coring at 19 feet. 5% Recovery from 19 to 24 feet.
						End NX coring at 24 feet.
25			12	96	28	Loose at 24 feet.
30			19		23	

Plate A4.14

HIRATA & ASSOCIATES, INC.

BORING LOG

W.O. 04-3925

BORING NO. B5 (continued) DRIVING WT. 140 lb. START DATE 6/8/05

SURFACE ELEV. 12.5± DROP 30 in. END DATE 6/21/05

DEPTH	GRAPH	SAMPLE	BLOWS PER FOOT	DRY DENSITY (PCF)	MOIST. CONT. (%)	DESCRIPTION
30						
35			25	102	24	
40			7	79	40	Very loose, with coral fragments at 39 feet.
45			50/4"			CORAL RUBBLESTONE – Mottled tan and gray, dense to medium hard, with layers of cemented sand. Begin NX coring at 46 feet. 93% Recovery from 46 to 51 feet.
50						46% Recovery from 51 to 55 feet.
55						71% Recovery from 55 to 59 feet.
60			41	96	23	

HIRATA & ASSOCIATES, INC.

BORING LOG

W.O. 04-3925

BORING NO. B5 (continued) DRIVING WT. 140 lb. START DATE 6/8/05
SURFACE ELEV. 12.5± DROP 30 in. END DATE 6/21/05

DEPTH	GRAPH	SAMPLE	BLOWS PER FOOT	DRY DENSITY (PCF)	MOIST. CONT. (%)	DESCRIPTION
60						Silty SAND (SM) – Mottled tan and gray, medium dense, with coral fragments.
65		☐	29	85	35	Clayey SILT (MH) – Brown, stiff, with sand and gravel.
70		☐	56/6"	No Recovery		
75		☐	31	79	39	Silty Coralline GRAVEL (GM) – Tan, medium dense, with sand.
80		☐	15	75	48	Loose from 79 feet.
						Clayey SILT (MH) – Brown, medium stiff.
85		☐	Weight of the rods	68	48	Very soft at 84 feet.
90		☐	24	68	58	Medium stiff at 89 feet.

HIRATA & ASSOCIATES, INC.

BORING LOG

W.O. 04-3925

BORING NO. B5 (continued) DRIVING WT. 140 lb. START DATE 6/8/05

SURFACE ELEV. 12.5± DROP 30 in. END DATE 6/21/05

DEPTH	GRAPH	SAMPLE	BLOWS PER FOOT	DRY DENSITY (PCF)	MOIST. CONT. (%)	DESCRIPTION
90						
		☐	20	61	68	
95						
						Grading with gravel and cobbles from 96 feet.
100						
						Silty Coralline GRAVEL (GM) – Tan, medium dense to dense, with sand.
		☐	40	98	23	
105						
		☐	77	104	24	
110						End boring at 110.5 feet. Groundwater at 9.8 feet on 6/15/05.
115						
120						

Plate A4.17

HIRATA & ASSOCIATES, INC.

BORING LOG

W.O. 04-3925

BORING NO. B6 DRIVING WT. 140 lb. START DATE 6/22/05
SURFACE ELEV. 11± DROP 30 in. END DATE 6/23/05

DEPTH	GRAPH	SAMPLE	BLOWS PER FOOT	DRY DENSITY (PCF)	MOIST. CONT. (%)	DESCRIPTION
0						Silty SAND (SM) – Brown, moist, medium dense, with clayey silt.
			26	92	13	
			25/9"	71	13	Cobbles at 2.5 to 3.5 feet.
			10/No Penetration			
5			16	123	8	
						(Begin wash–boring at 8 feet)
			10/No Penetration			CORAL RUBBLESTONE – Mottled tan, slightly moist, dense to very dense.
10						Silty SAND (SM) – Mottled gray, medium dense, with gravel and coral fragments. Begin NX coring at 9 feet. 10% Recovery from 9 to 14 feet.
						End NX coring at 14 feet.
			14	95	24	Loose at 14 feet.
15						
						Silty Coralline GRAVEL (GM) – Tan, loose to medium dense, with sand.
20			12	78	32	Brown clayey silt pocket at 19.5 feet.
			17	79	39	
25						End boring at 25.5 feet.
						Groundwater at 9.8 feet on 6/23/05.
30						

HIRATA & ASSOCIATES, INC.

BORING LOG

W.O. 04-3925

BORING NO. B7 | DRIVING WT. 140 lb. | START DATE 6/14/05

SURFACE ELEV. 11± | DROP 30 in. | END DATE 6/14/05

DEPTH	GRAPH	SAMPLE	BLOWS PER FOOT	DRY DENSITY (PCF)	MOIST. CONT. (%)	DESCRIPTION
0						SAND (SP) – Mottled brown, slightly moist, loose to medium dense, with coral fragments.
			23	79	4	
			18		3	
5			23	101	4	
						CORAL RUBBLESTONE – Mottled tan, slightly moist, dense to medium hard, with layers of cemented sand.
			45/6"	112	5	
10						
▽						
			10/No Penetration			
15						
			88	110	15	Cemented sand at 19 feet.
20						
			114	119	14	
25						End boring at 25 feet. Groundwater at 11.6 feet on 6/14/05.
30						

Plate A4.19

HIRATA & ASSOCIATES, INC.

BORING LOG

W.O. 04-3925

BORING NO. B8 | DRIVING WT. 140 lb. | START DATE 6/22/05

SURFACE ELEV. 10± | DROP 30 in. | END DATE 6/23/05

DEPTH	GRAPH	SAMPLE	BLOWS PER FOOT	DRY DENSITY (PCF)	MOIST. CONT. (%)	DESCRIPTION
0						SAND (SP) – Tannish brown, slightly moist, medium dense, with silt, cobbles, and boulders. Cobbles and boulders from 1 to 3 feet.
5			28	87	11	(Begin wash boring at 5 feet.)
						Silty SAND (SM) – Mottled gray, medium dense, with gravel.
10			27	96	22	
						CORAL RUBBLESTONE – Mottled tan, medium hard to hard, fragmented, with layers of cemented sand.
15			10/No Penetration			Begin NX coring at 14 feet. 100% Recovery from 14 to 19 feet.
20						65% Recovery from 19 to 24 feet.
25						End boring at 24 feet.
						Groundwater at 8 feet on 6/23/05.
30						

HIRATA & ASSOCIATES, INC.

BORING LOG

W.O. 04-3925

BORING NO. B9 | DRIVING WT. 140 lb. | START DATE 6/21/05

SURFACE ELEV. 11± | DROP 30 in. | END DATE 6/21/05

DEPTH	BLOWS PER FOOT	DRY DENSITY (PCF)	MOIST. CONT. (%)	DESCRIPTION
0				Silty SAND (SM) – Mottled tan, slightly moist, medium dense, with cobbles and boulders. Boulder at 1 to 3 feet.
	32	79	10	
5	13	102	2	Decrease in silt content, loose at 5 feet.
				(Begin wash-boring at 7 feet.)
	15/6" 50/5"	111	18	
10				CORAL RUBBLESTONE – Mottled tan, medium hard to hard, fragmented, with layers of cemented sand.
15				Begin NX coring at 14 feet. 100% Recovery from 14 to 17 feet.
				50% Recovery from 17 to 22 feet.
20				Silty SAND (SM) – Dark gray, loose.
	11	79	42	
25				End boring at 23.5 feet. Groundwater at 8.8 feet on 6/21/05.
30				

HIRATA & ASSOCIATES, INC.

BORING LOG

W.O. 04-3925

BORING NO. B10 | DRIVING WT. 140 lb. | START DATE 6/22/05

SURFACE ELEV. 12± | DROP 30 in. | END DATE 6/23/05

DEPTH	GRAPH	SAMPLE	BLOWS PER FOOT	DRY DENSITY (PCF)	MOIST. CONT. (%)	DESCRIPTION
0						Clayey SILT (MH) – Brown, moist, stiff, with sand and gravel.
			42	96	24	
			78	90	13	Increase in sand content at 3 feet.
5			67	117	2	SAND (SP) – Tan, slightly moist, dense, with coral fragments. Cobbles from 6 to 8 feet.
						CORAL – Tan, medium hard to hard, fragmented.
10						Begin NX coring at 9 feet. 88% Recovery from 9 to 14 feet.
15						97% Recovery from 14 to 19 feet.
20						92% Recovery from 19 to 24 feet.
25						End boring at 24 feet. Groundwater at 10.1 feet on 6/23/05.
30						

Plate A4.22

APPENDIX B

LABORATORY TESTING

DESCRIPTION OF LABORATORY TESTING

CLASSIFICATION

Field classification was verified in the laboratory in accordance with the Unified Soil Classification System. Laboratory classification was determined by visual examination and Atterberg Limit tests performed in general accordance with ASTM D 4318. Results of Atterberg Limit tests are presented below. The final classifications are shown at the appropriate locations on the Boring Logs, Plates A4.1through A4.22.

Sample	Liquid Limit	Plasticity Index (PI)
B1 at 3 feet	28	9

MOISTURE-DENSITY

Representative samples were tested for insitu moisture content and dry unit weight. The dry unit weight was determined in pounds per cubic foot while the moisture content was determined as a percentage of dry weight. Samples were obtained using a 3-inch O.D. split tube sampler. Test results are shown at the appropriate depths on the Boring Logs, Plates A4.1 through A4.22.

CONSOLIDATION

Consolidation tests were performed on representative soil samples, 2.42 inches in diameter and 1 inch high. Porous stones were placed in contact with the top and bottom of the test sample to permit addition and release of pore fluid. Loads were then applied in several increments in a geometric progression, and the resulting deformations recorded at selected time intervals. Test results are plotted on the Consolidation Test Reports, Plates B2.1 through B2.3.

SHEAR TESTS

Shear tests were performed on representative soil samples using the Direct Shear Machine which is of the strain control type. Each sample was sheared under varying confining loads in order to determine the Coulomb shear strength parameters, cohesion and angle of internal friction. Test results are presented on Plates B3.1 through B3.7.

SIEVE ANALYSES

Sieve analysis tests were conducted on selected soil samples to determine the distribution of particle sizes in the soil. The tests were conducted in general accordance with ASTM D 422. Test results are presented on Plates B4.1 through B4.4.

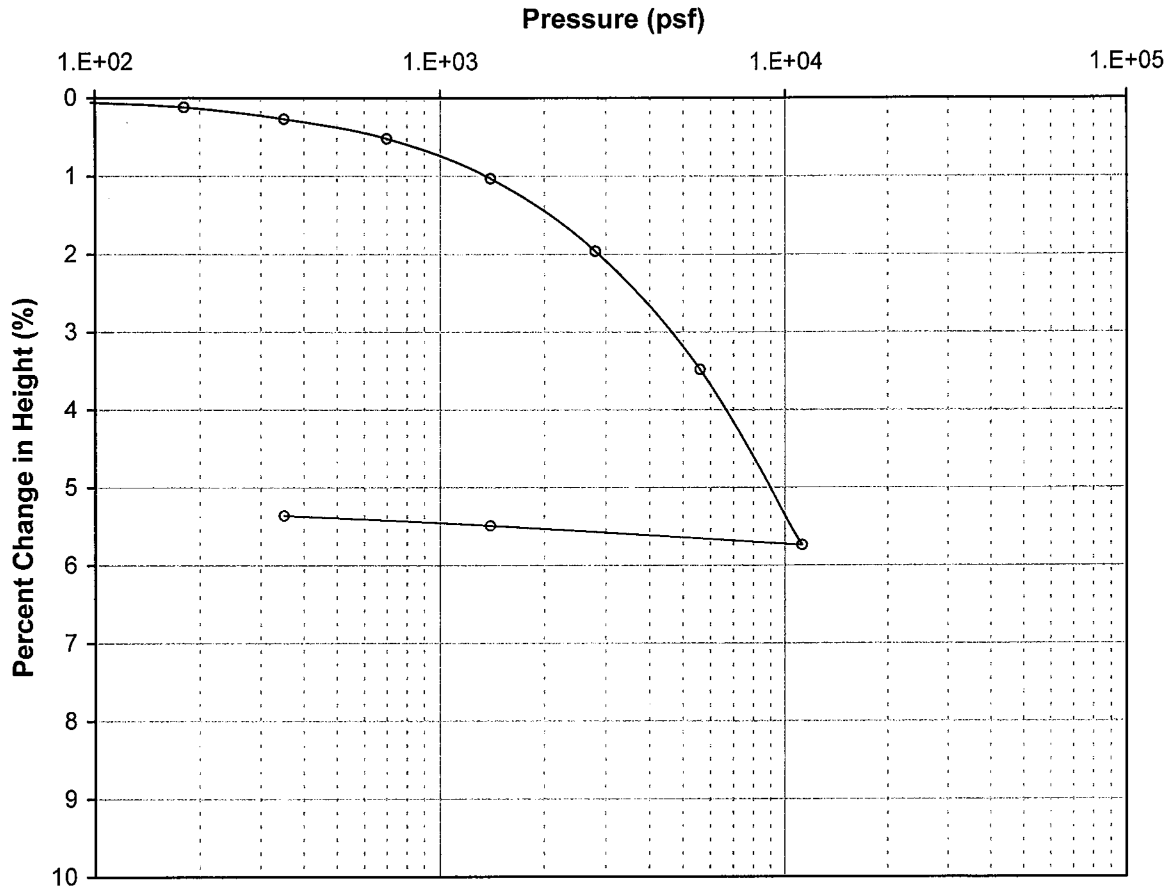
PROCTOR TESTS

A Modified Proctor test was performed on a bag sample of near surface soil obtained from boring B9. The test was performed in general accordance with ASTM D 1557 and results are shown on Plate B5.1.

R-VALUE TESTS

Two R-Value tests were performed on bulk samples of near surface soil. The tests were performed by Krazan Testing Laboratory in Clovis, California, in general accordance with ASTM D 2844. Test results are shown on Plates B6.1 and B6.2.

Consolidation Test Results



<u>Sample Description</u>

Boring No.: B1 Depth (ft): 39

Soil Description: Tan silty sand

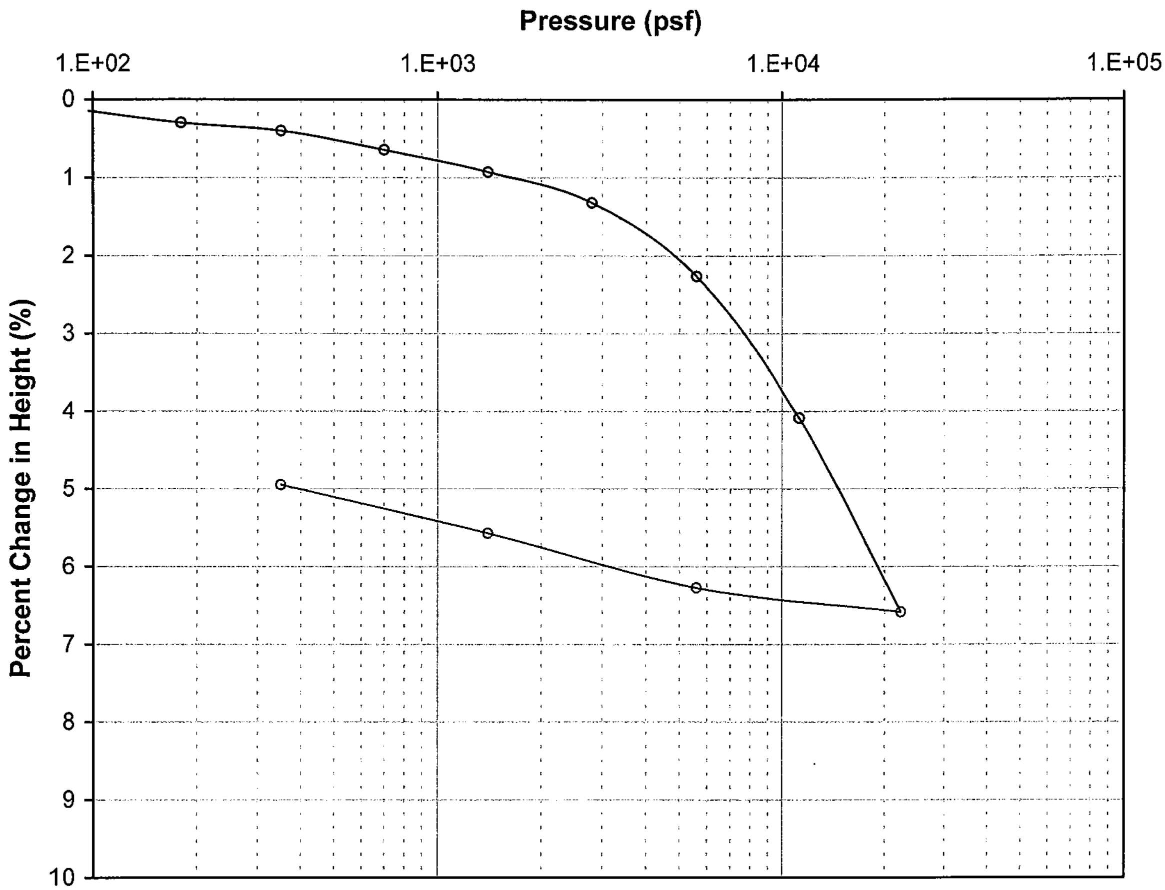
	Moisture Content (%)	Dry Density (pcf)
Initial	35.9	76.6
Final	30.7	80.9

Remark: 10/28/05

W.O. 04-3925	Farrington Highway, Makaha Bridge No. 3 and No. 3A Replacement
Hirata & Associates, Inc.	**CONSOLIDATION TEST**

Plate B2.1

Consolidation Test Results



Sample Description

Boring No.: B4 Depth (ft): 89

Soil Description: Brown clayey silt

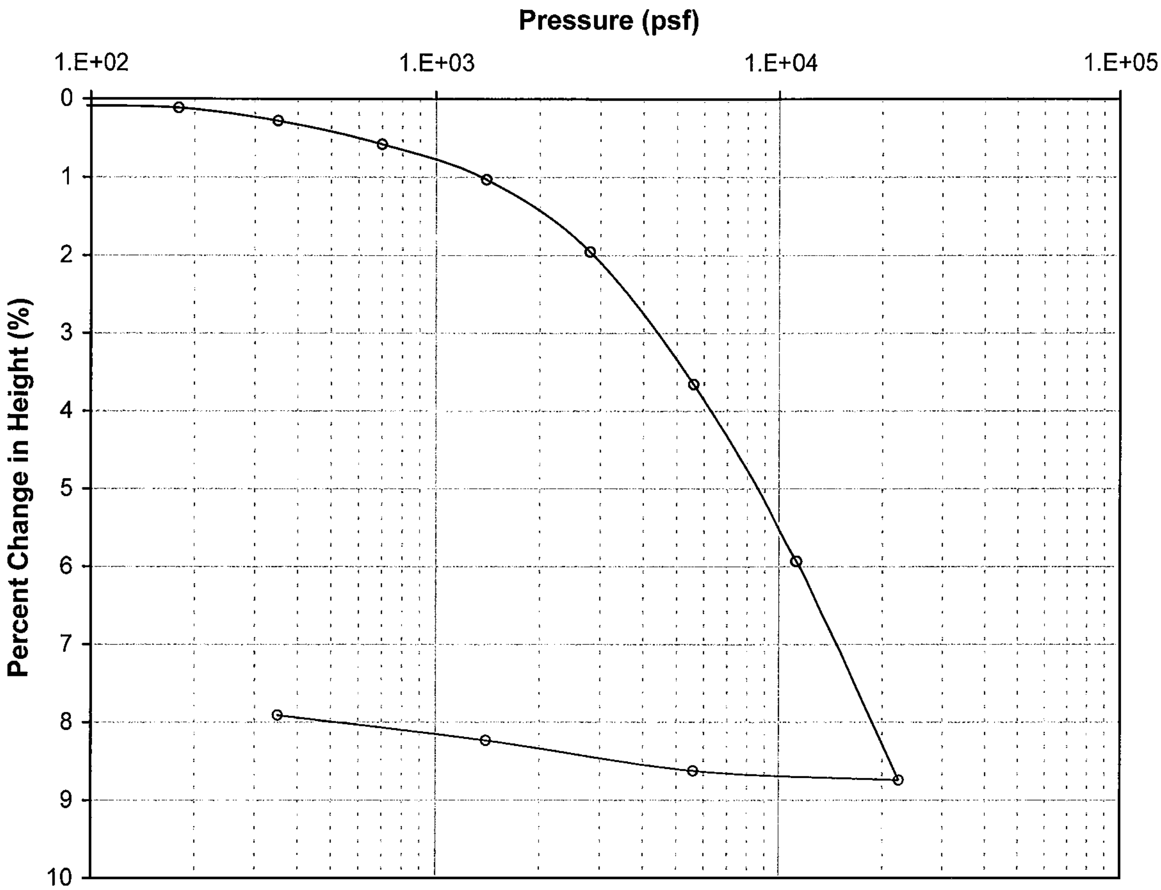
	Moisture Content (%)	Dry Density (pcf)
Initial	39.3	84.5
Final	37.5	88.0

Remark: 10/22/2005

W.O. 04-3925	Farrington Highway, Makaha Bridge No. 3 and No. 3A Replacement
Hirata & Associates, Inc.	CONSOLIDATION TEST

Plate B2.2

Consolidation Test Results



Sample Description

Boring No.: B5 Depth (ft): 64

Soil Description: Brown clayey silt

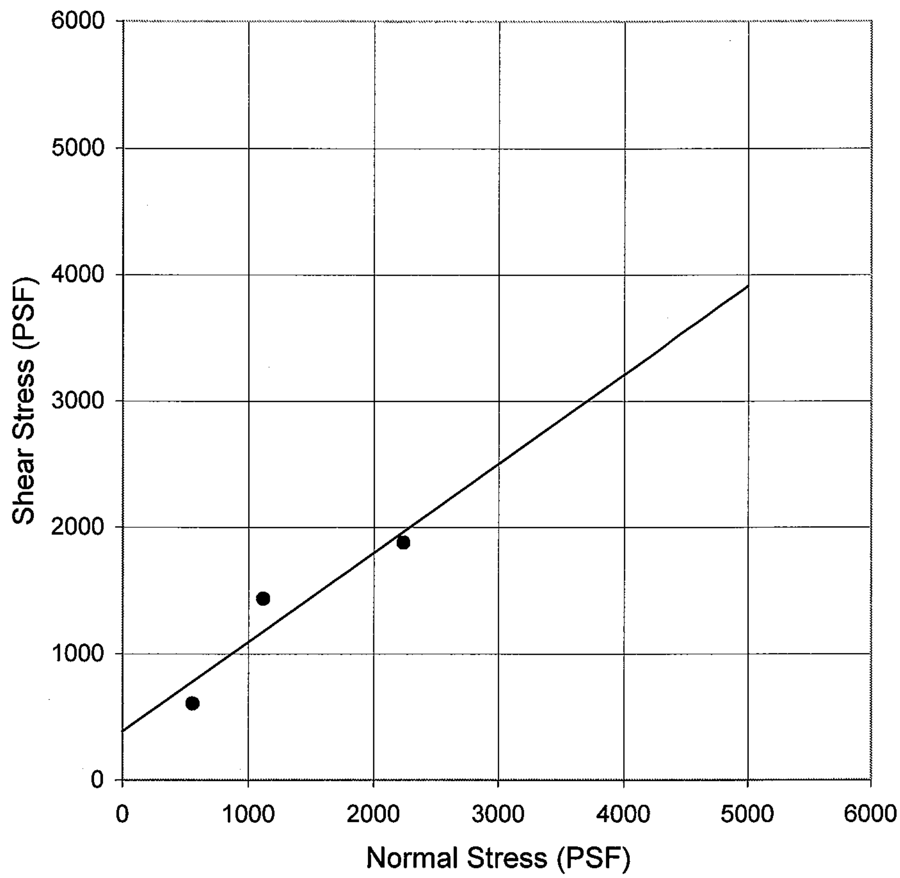
	Moisture Content (%)	Dry Density (pcf)
Initial	34.9	84.5
Final	31.1	91.8

Remark: 10/26/05

W.O. 04-3925	Farrington Highway, Makaha Bridge No. 3 and No. 3A Replacement
Hirata & Associates, Inc.	CONSOLIDATION TEST

Plate B2.3

Direct Shear Test Results



Sample Description

Boring No.: B1 Depth (ft): 29

Soil Description: Tan silty sand

Strength Intercept (C): 385.0 PSF (Peak Strength)

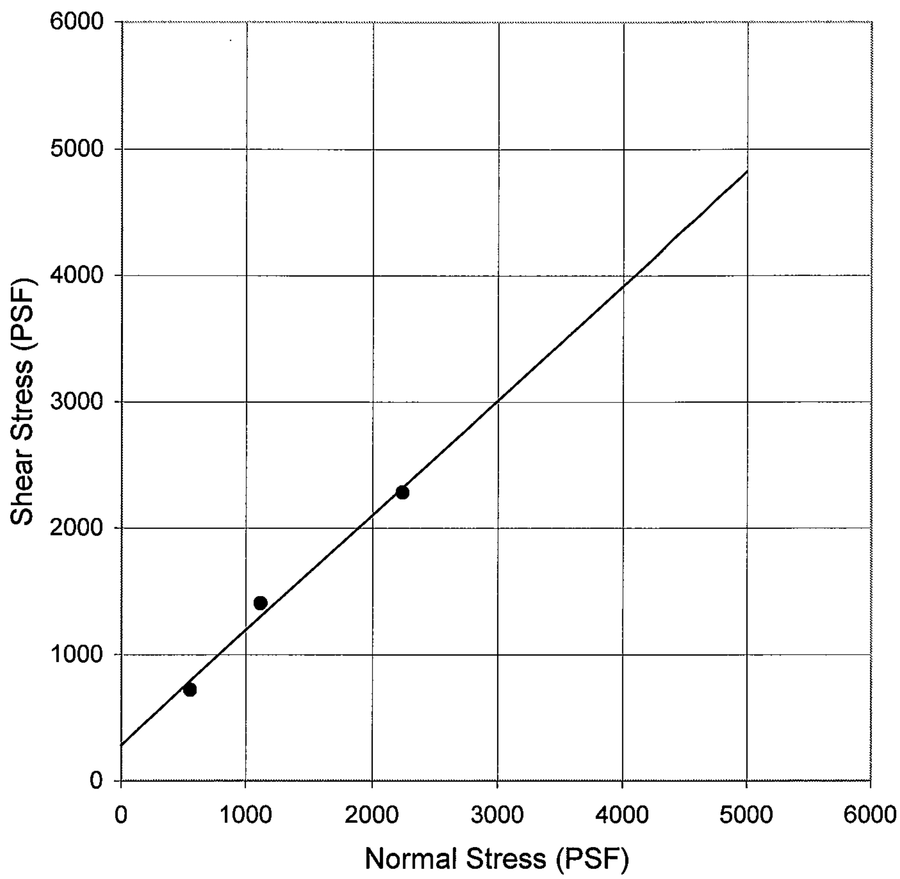
Friction Angle (ϕ): 35.2 DEG (Peak Strength)

Remark: 10/17/05

W.O. 04-3925	**Farrington Highway, Makaha Bridge No. 3 and No. 3A Replacement**
Hirata & Associates, Inc.	**DIRECT SHEAR TEST**

Plate B3.1

Direct Shear Test Results

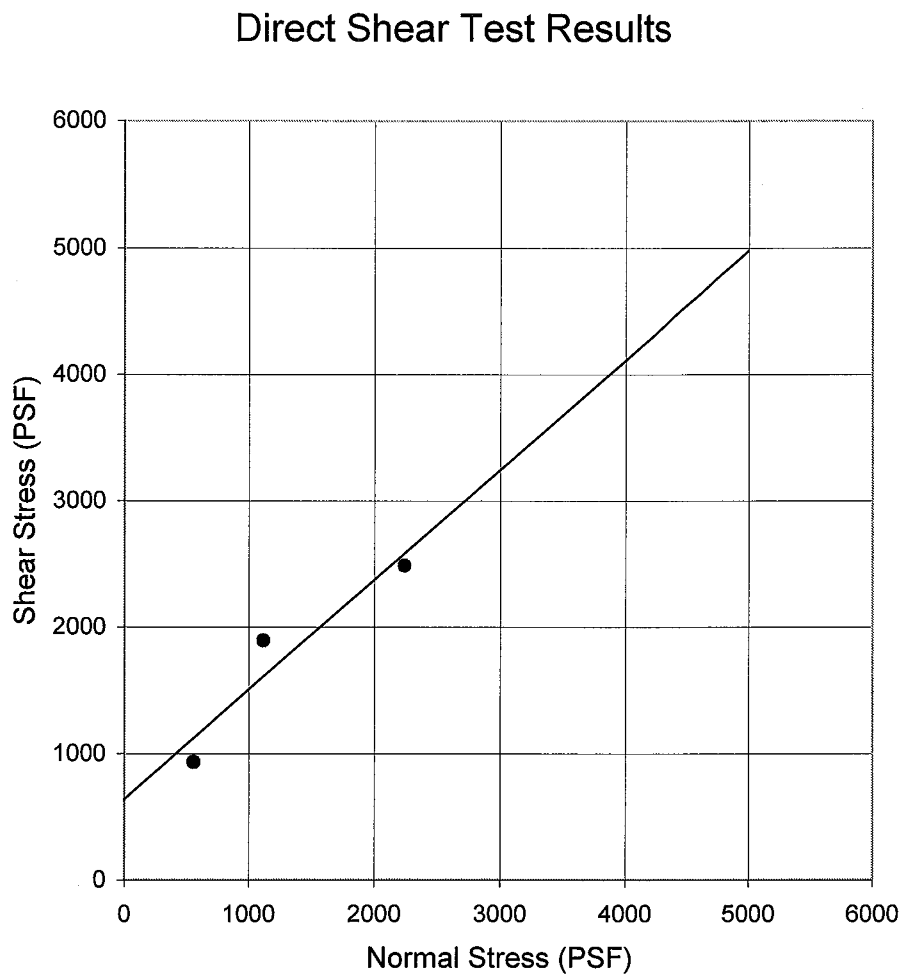


<u>Sample Description</u>

Boring No.: B1	Depth (ft): 49		
Soil Description:	Tan silty coralline gravel		
Strength Intercept (C):		282.2 PSF	(Peak Strength)
Friction Angle (ϕ):		42.2 DEG	(Peak Strength)

Remark: 10/17/05

W.O. 04-3925	**Farrington Highway, Makaha Bridge No. 3 and No. 3A Replacement**
Hirata & Associates, Inc.	**DIRECT SHEAR TEST** Plate B3.2



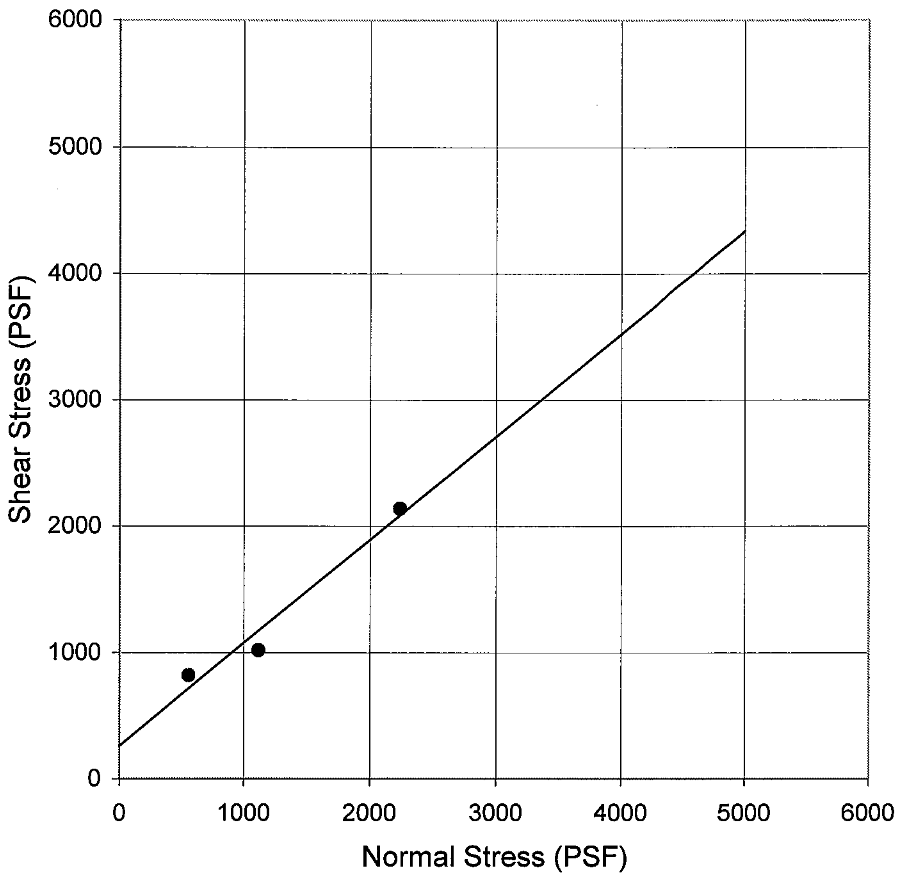
Sample Description

Boring No.: B2 Depth (ft): 28
Soil Description: Tan silty coralline gravel
Strength Intercept (C): 636.3 PSF (Peak Strength)
Friction Angle (ϕ): 41.0 DEG (Peak Strength)

Remark: 8/23/04

W.O. 04-3925	**Farrington Highway, Makaha Bridge No. 3 and No. 3A Replacement**
Hirata & Associates, Inc.	**DIRECT SHEAR TEST**

Direct Shear Test Results



Sample Description

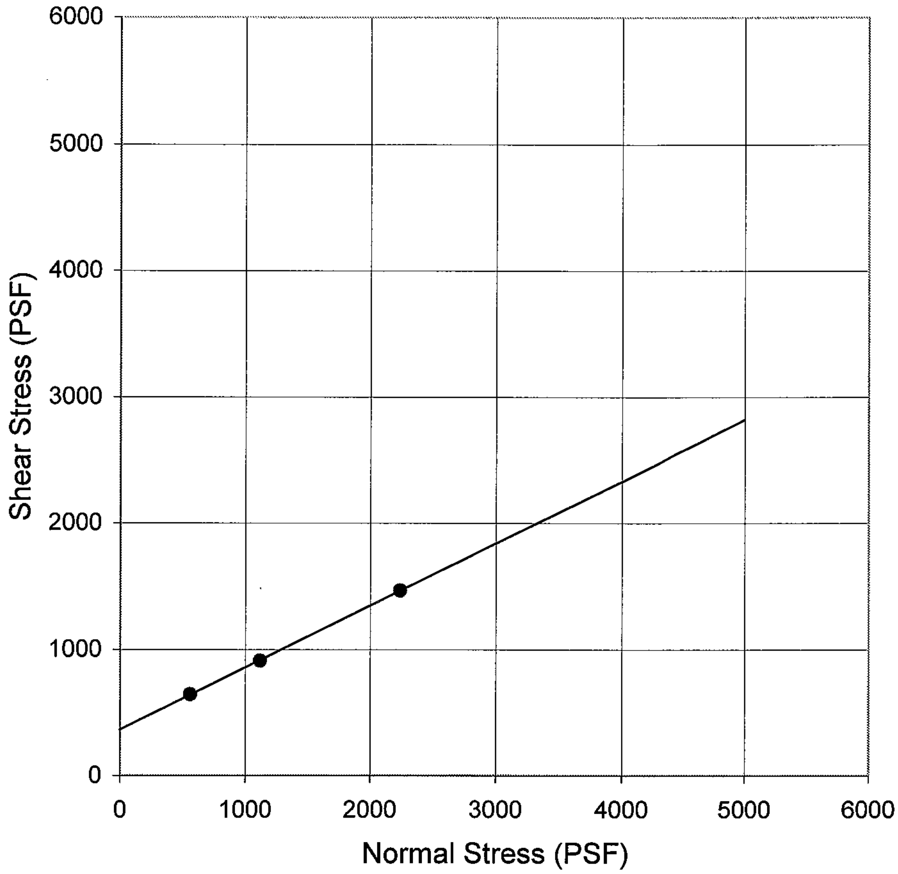
Boring No.: B2	Depth (ft): 83		
Soil Description:	Tan silty coralline gravel		
Strength Intercept (C):	259.4 PSF		(Peak Strength)
Friction Angle (ϕ):	39.2 DEG		(Peak Strength)

Remark: 8/23/04

W.O. 04-3925	**Farrington Highway, Makaha Bridge No. 3 and No. 3A Replacement**
Hirata & Associates, Inc.	**DIRECT SHEAR TEST**

Plate B3.4

Direct Shear Test Results



Sample Description

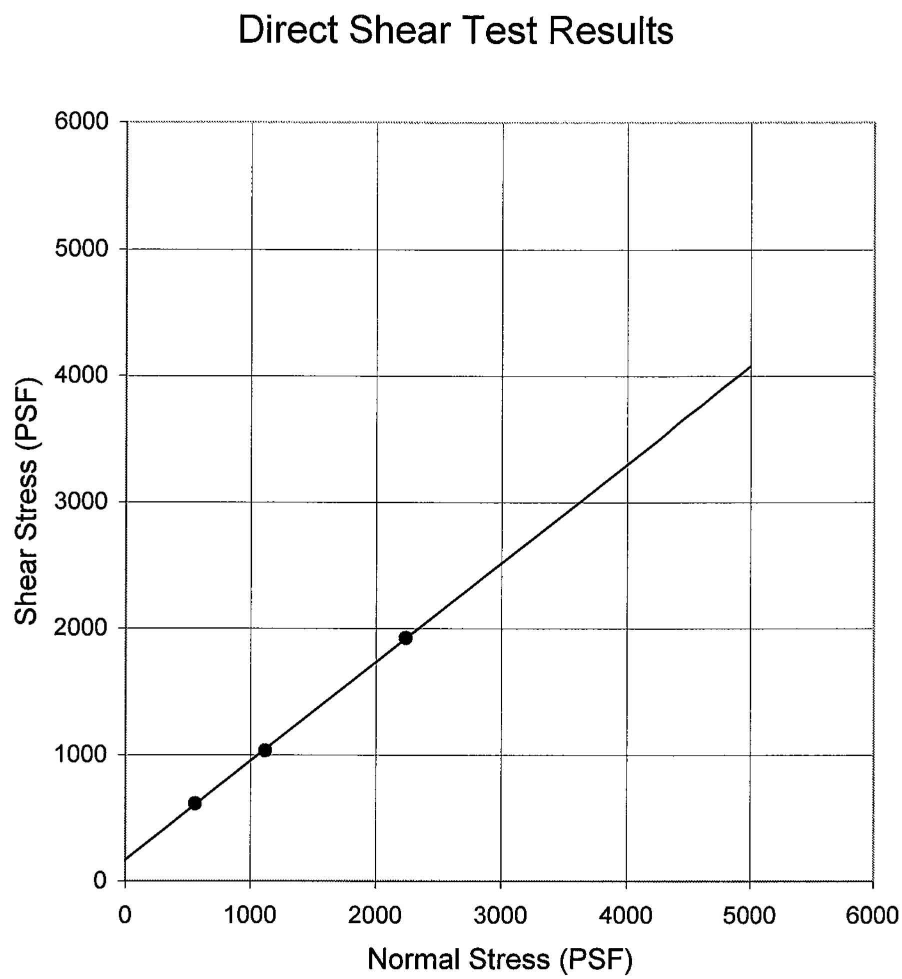
Boring No.: B4 Depth (ft): 3

Soil Description: Brown clayey silt

Strength Intercept (C):	366.0 PSF	(Peak Strength)
Friction Angle (ϕ):	26.1 DEG	(Peak Strength)

Remark: 9/22/05

W.O. 04-3925	**Farrington Highway, Makaha Bridge No. 3 and No. 3A Replacement**
Hirata & Associates, Inc.	**DIRECT SHEAR TEST**



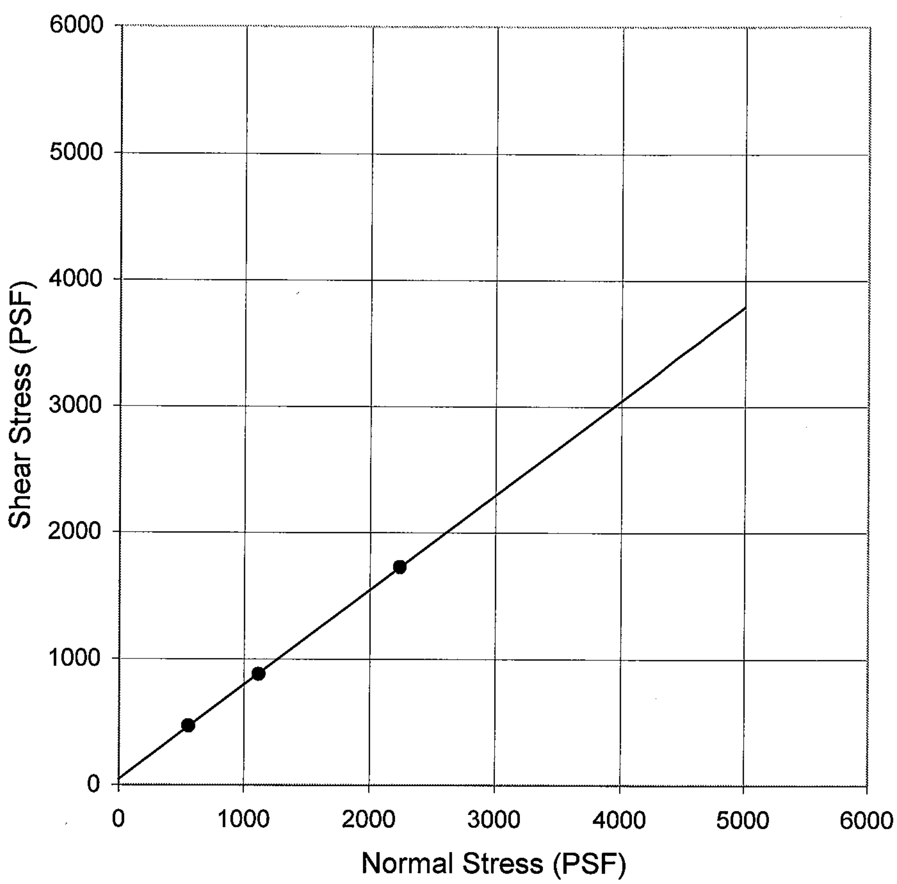
Sample Description

Boring No.: B4 Depth (ft): 69
Soil Description: Brown clayey sand
Strength Intercept (C): 168.0 PSF (Peak Strength)
Friction Angle (ϕ): 38.0 DEG (Peak Strength)

Remark: 9/23/05

W.O. 04-3925	**Farrington Highway, Makaha Bridge No. 3 and No. 3A Replacement**
Hirata & Associates, Inc.	**DIRECT SHEAR TEST**

Direct Shear Test Results



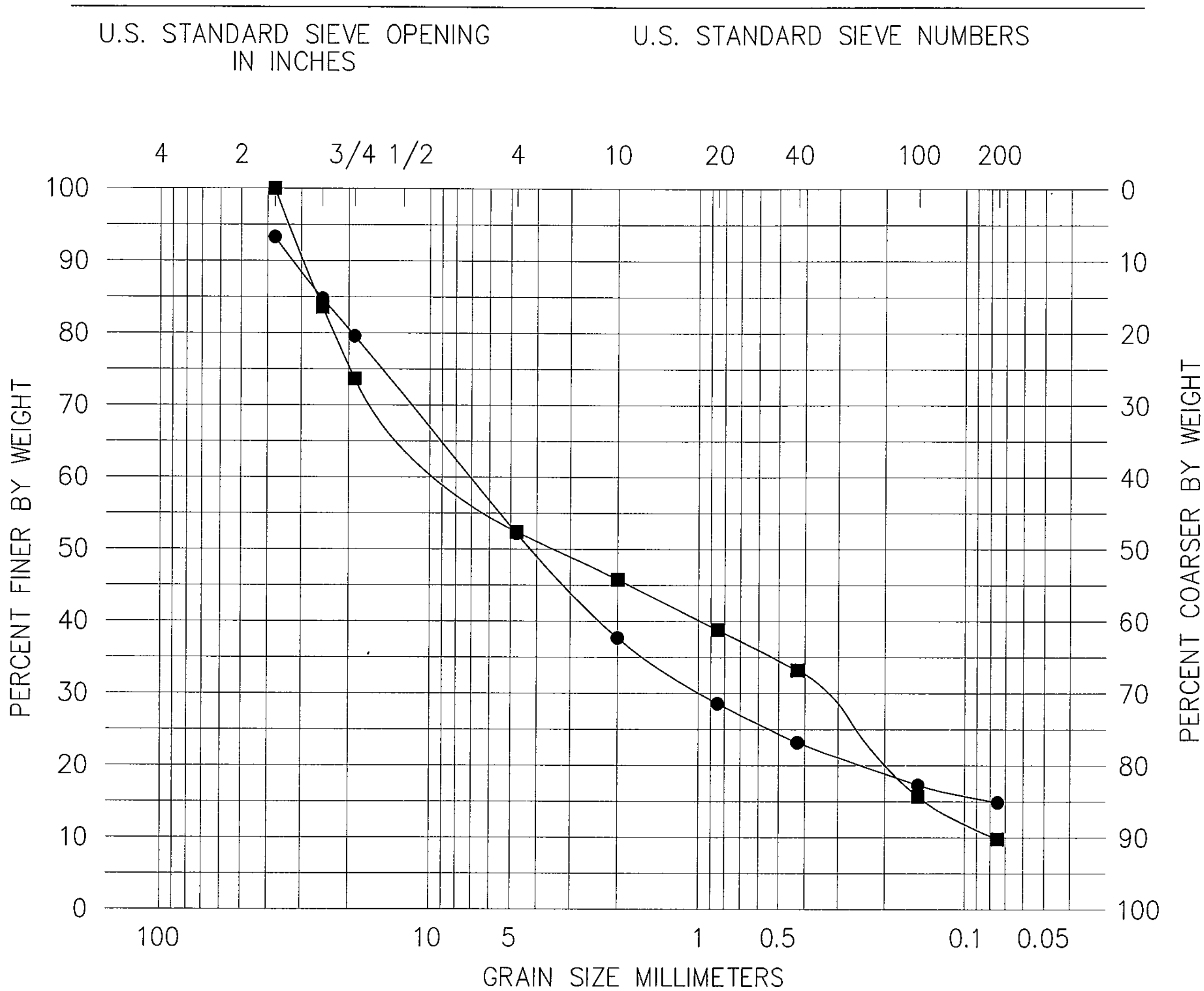
Sample Description

Boring No.: B5 Depth (ft): 34
Soil Description: Mottled gray sand
Strength Intercept (C): 46.2 PSF (Peak Strength)
Friction Angle (ϕ): 36.8 DEG (Peak Strength)

Remark: 9/23/05

W.O. 04-3925	**Farrington Highway, Makaha Bridge No. 3 and No. 3A Replacement**
Hirata & Associates, Inc.	**DIRECT SHEAR TEST**

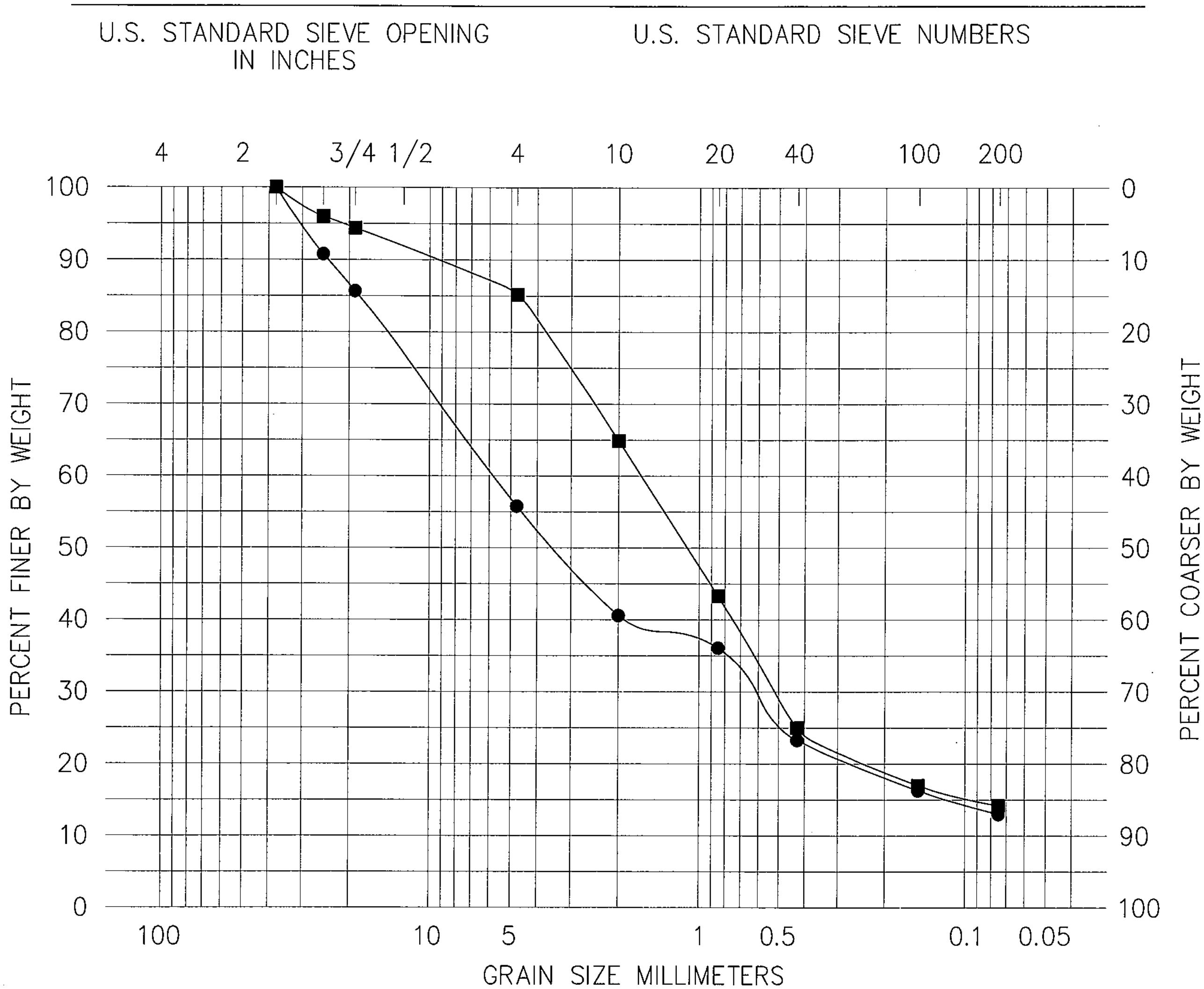
Plate B3.7



COBBLES	GRAVEL		SAND			SILT or CLAY
	Coarse	Fine	Coarse	Medium	Fine	

● Sample #1	Location:	Boring B1 at 54 feet
	Description:	Tan silty coralline gravel with sand
■ Sample #2	Location:	Boring B1 at 74 feet
	Description:	Brownish tan coralline gravel with silt and sand

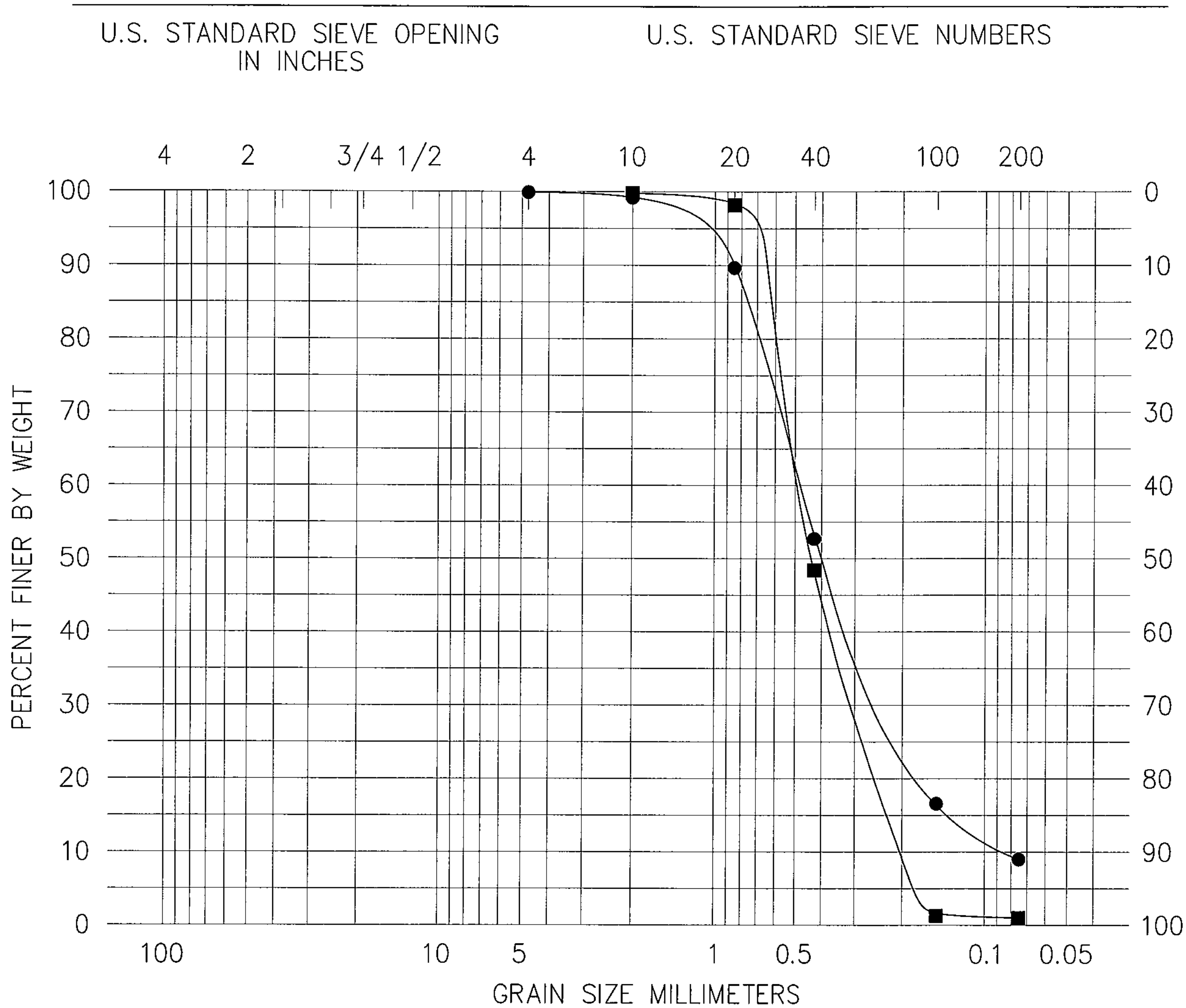
W.O. 04-3925	Farrington Highway, Makaha Bridge No. 3 and No. 3A Replacement
Hirata & Associates, Inc.	GRADATION CURVES Plate B4.1



COBBLES	GRAVEL		SAND			SILT or CLAY
	Coarse	Fine	Coarse	Medium	Fine	

● Sample #1	Location:	Boring B2 at 58 feet
	Description:	Tan silty coralline gravel with sand
■ Sample #2	Location:	Boring B4 at 44 feet
	Description:	Mottled tan and gray silty sand

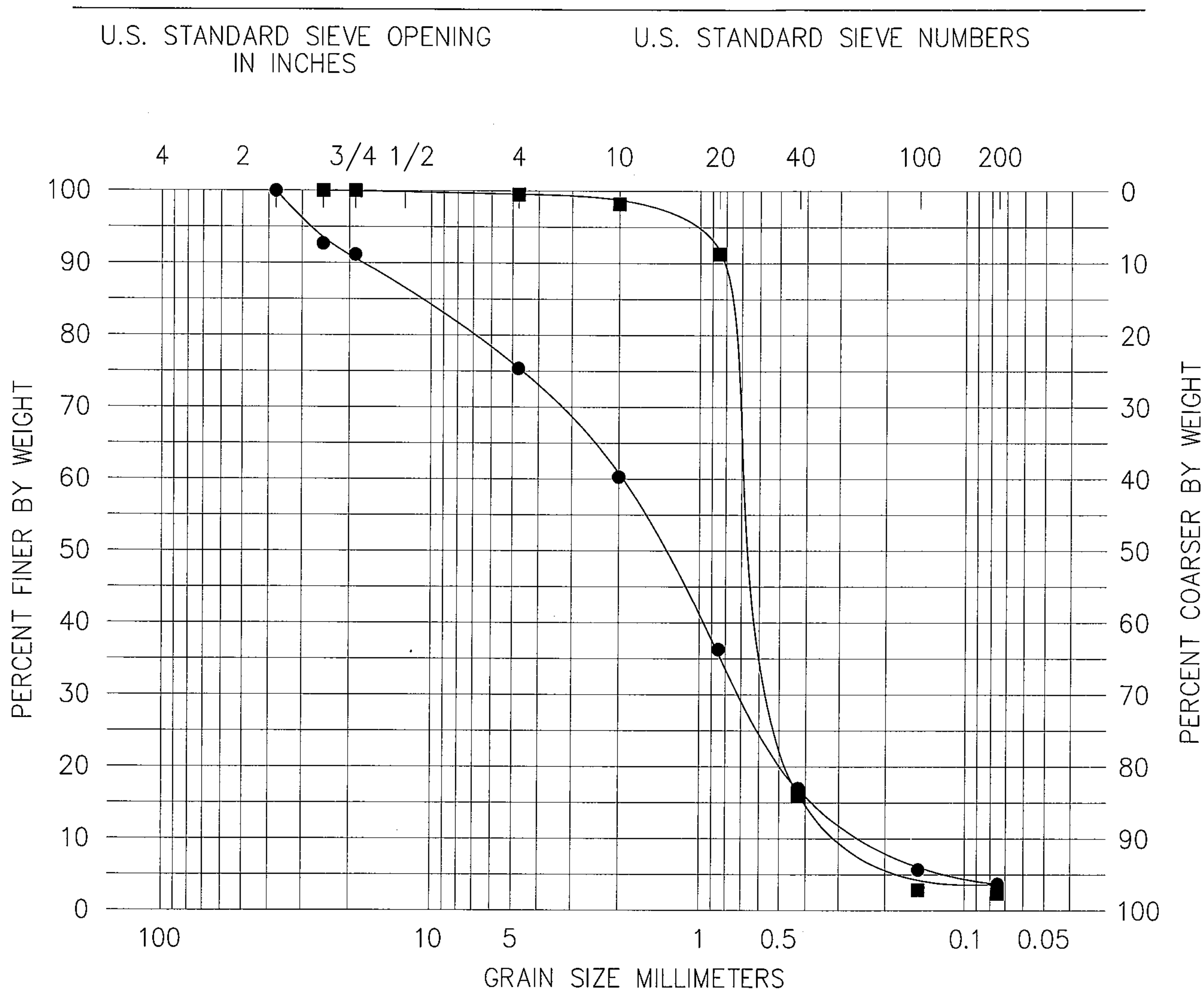
W.O. 04-3925	Farrington Highway, Makaha Bridge No. 3 and No. 3A Replacement
Hirata & Associates, Inc.	GRADATION CURVES Plate B4.2



COBBLES	GRAVEL		SAND			SILT or CLAY
	Coarse	Fine	Coarse	Medium	Fine	

● Sample #1	Location:	Boring B5 at 29 feet
	Description:	Mottled gray sand with silt
■ Sample #2	Location:	Boring B7 at 3 feet
	Description:	Mottled brown sand

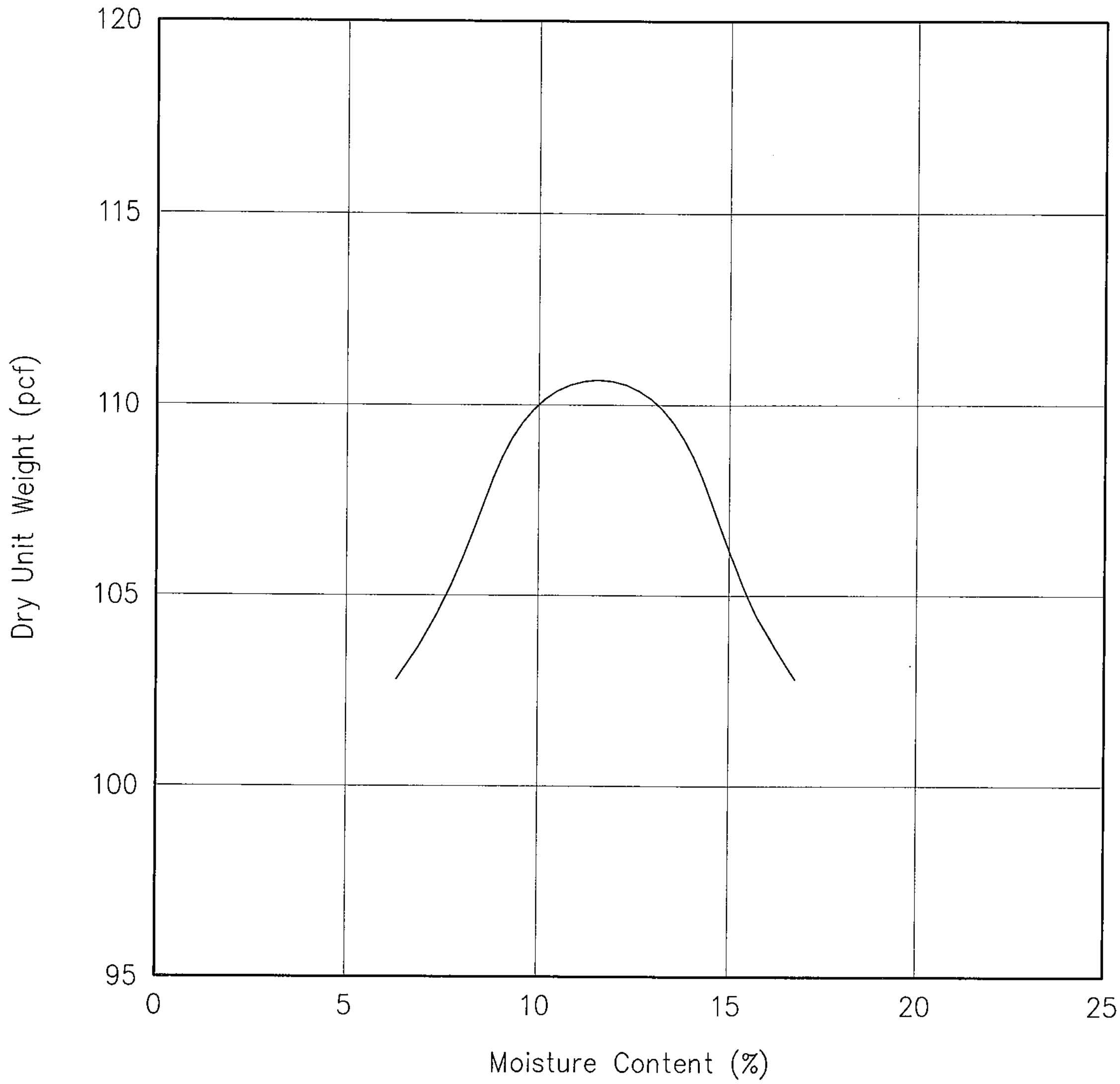
W.O. 04-3925	Farrington Highway, Makaha Bridge No. 3 and No. 3A Replacement
Hirata & Associates, Inc.	GRADATION CURVES Plate B4.3



COBBLES	GRAVEL		SAND			SILT or CLAY
	Coarse	Fine	Coarse	Medium	Fine	

● Sample #1	Location:	Bridge No. 3 at stream invert
	Description:	Brown silty sand with gravel
■ Sample #2	Location:	Bridge No. 3A at stream invert
	Description:	Mottled gray and tan sand

W.O. 04-3925	Farrington Highway, Makaha Bridge No. 3 and No. 3A Replacement
Hirata & Associates, Inc.	GRADATION CURVES Plate B4.4



Soil Data

Location: Boring B9 near surface

Description: Mottled tan silty sand

Test Results

Maximum Dry Density: 111 pcf

Optimum Moisture Content: 11.5%

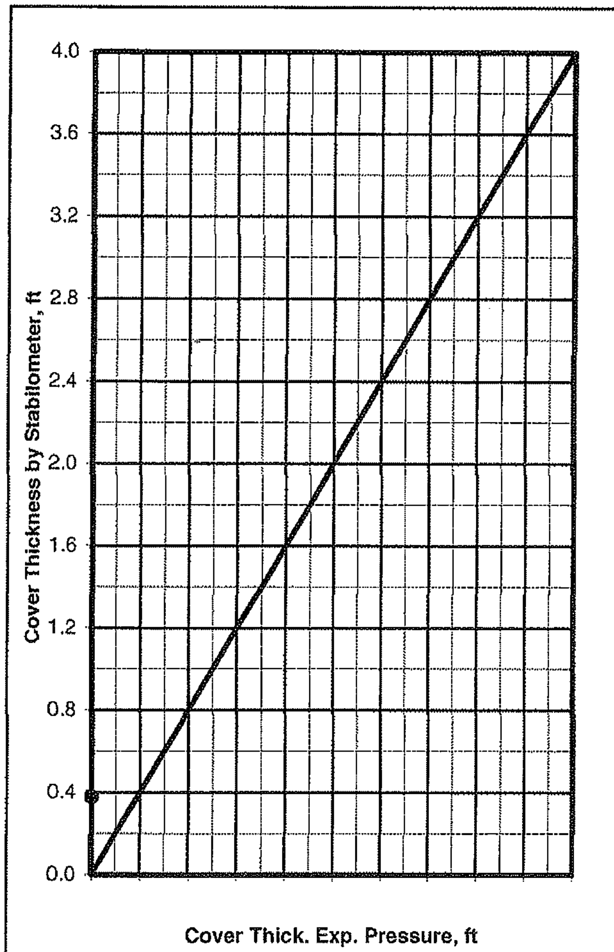
W.O. 04-3925	Farrington Highway, Makaha Bridge No. 3 and No. 3A Replacement
Hirata & Associates, Inc.	MODIFIED PROCTOR CURVE Plate B5.1

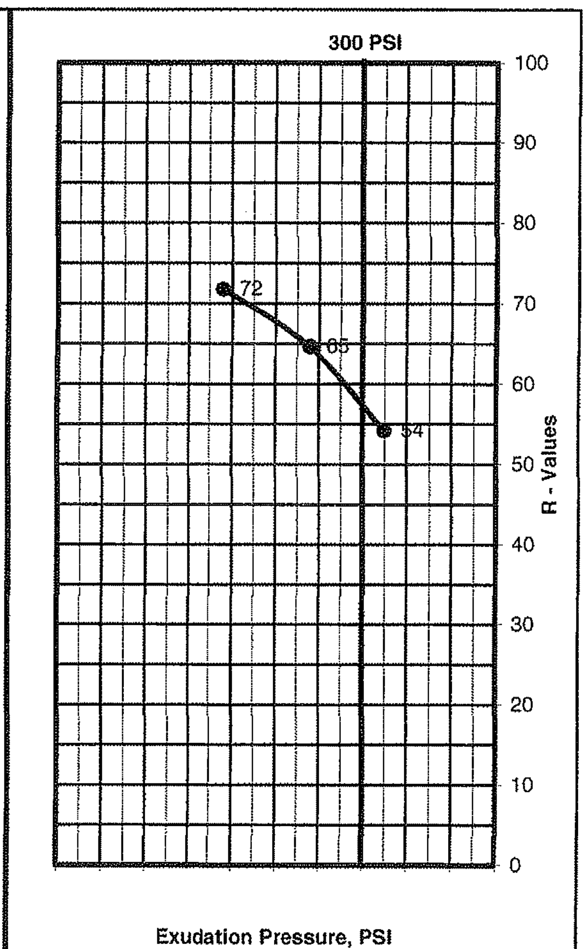
R - VALUE TEST
ASTM D - 2844 / CAL 301

Project Number	:	1204013
Project Name	:	E K Hirata & Associates
Date Tested	:	7/11/05
Date Sampled	:	8/16/05
Sample Location/Curve Number	:	Farrington Highway/ R-value # 1/ B-6
Work Order Number	:	04-3925
Soil Classification	:	SM w/ gravel

TEST	A	B	C
Percent Moisture @ Compaction, %	13.0	14.5	12.4
Dry Density, lbm/cu.ft.	123.2	121.6	123.1
Exudation Pressure, psi	420	250	620
Expansion Pressure, (Dial Reading)	0	0	0
Expansion Pressure, psf	0	0	0
Resistance Value R	65	54	72

R Value at 300 PSI Exudation Pressure	57
R Value by Expansion Pressure (TI =): 5	**Expansion Pressure nil**





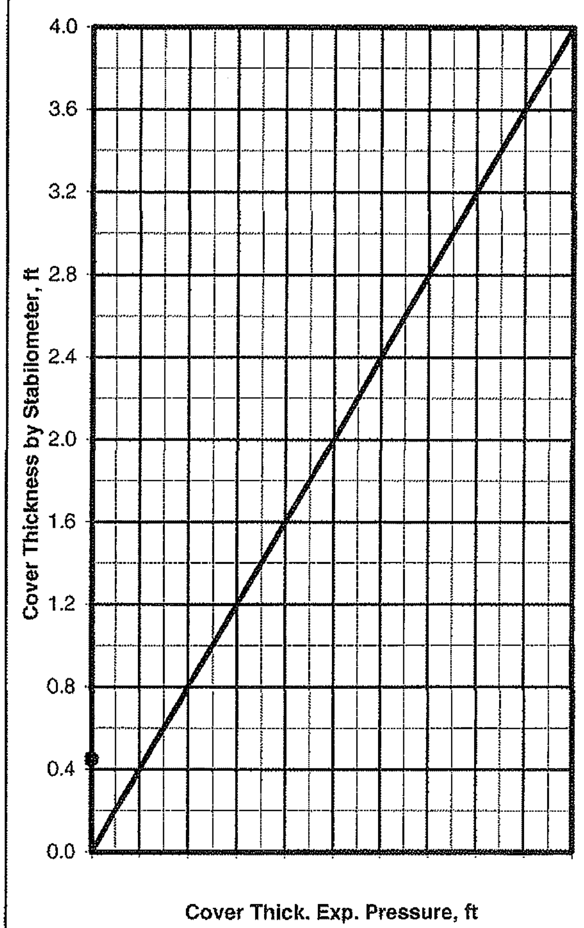
Krazan Testing Laboratory

R - VALUE TEST
ASTM D - 2844 / CAL 301

Project Number	:	1204013
Project Name	:	E K Hirata & Associates
Date Tested	:	7/11/05
Date Sampled	:	8/16/05
Sample Location/Curve Number	:	Farrington Highway/ R-value #2/ B-9
Work Order Number	:	04-3925
Soil Classification	:	SM w/ gravel

TEST	A	B	C
Percent Moisture @ Compaction, %	13.2	12.6	14.3
Dry Density, lbm/cu.ft.	110.2	110.9	108.6
Exudation Pressure, psi	345	620	190
Expansion Pressure, (Dial Reading)	0	0	0
Expansion Pressure, psf	0	0	0
Resistance Value R	58	64	51

R Value at 300 PSI Exudation Pressure	**56**
R Value by Expansion Pressure (TI =): 5	**Expansion Pressure nil**



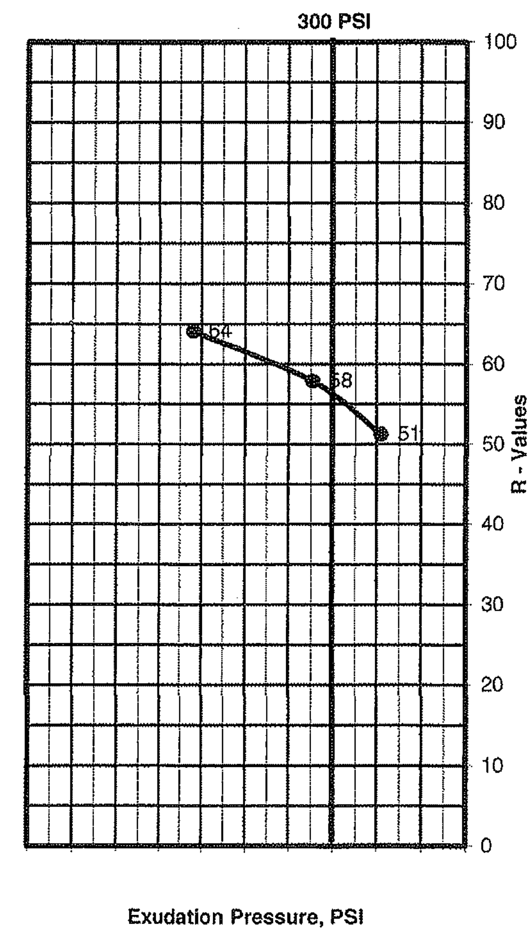


Plate B6.2

Krazan Testing Laboratory

APPENDIX C

LATERAL PILE ANALYSES

LATERAL LOAD ANALYSES

Bridge 3 Abutment 1 (Southern Abutment), Strength Limit State

Case 1, Axial = 1,709 kips, Shear = 424 kips, Moment = 2,291 kip-ft	
Computed Deflection	2.2 inches
Rotation at Top	0.0163 rad
Maximum Bending Moment	732.5 kip-ft
Location of Max. Moment	11 ft
Maximum Shear	60.6 kips
Case 2, Axial = 1,619 kips, Shear = 441 kips, Moment = 2,239 kip-ft	
Computed Deflection	2.28 inches
Rotation at Top	0.0166 rad
Maximum Bending Moment	748.3 kip-ft
Location of Max. Moment	11 ft
Maximum Shear	63 kips

LATERAL LOAD ANALYSES

Bridge 3 Abutment 2 (Northern Abutment), Strength Limit State

Case 1, Axial = 1,709 kips, Shear = 424 kips, Moment = 2,291 kip-ft			
	Upper Bound	Adopted Soil Profile	Lower Bound
Computed Deflection	0.7 inch	1.45 inches	2.37 inches
Rotation at Top	0.00828 rad	0.0126 rad	0.0169 rad
Maximum Bending Moment	558.3 kip-ft	656.7 kip-ft	730 kip-ft
Location of Max. Moment	7 ft	9 ft	11 ft
Maximum Shear	60.6 kips	60.6 kips	60.6 kips
Case 2, Axial = 1,619 kips, Shear = 441 kips, Moment = 2239 kip-ft			
	Upper Bound	Adopted Soil Profile	Lower Bound
Computed Deflection	0.72 inch	1.49 inches	2.45 inches
Rotation at Top	0.0084 rad	0.0129 rad	0.0173 rad
Maximum Bending Moment	564.2 kip-ft	666.7 kip-ft	746.7 kip-ft
Location of Max. Moment	7 ft	9 ft	11 ft
Maximum Shear	63 kips	63 kips	63 kips

LATERAL LOAD ANALYSES

Bridge 3A Abutments, Strength Limit State, No Scour

Case 1, Axial = 2,253 kips, Shear = 488 kips, Moment = 2,793 kip-ft	
Computed Deflection	1.5 inches
Rotation at Top	0.0136 rad
Maximum Bending Moment	704.2 kip-ft
Location of Max. Moment	8 ft
Maximum Shear	69.7 kips
Case 2, Axial = 2,131 kips, Shear = 516 kips, Moment = 2,683 kip-ft	
Computed Deflection	1.55 inches
Rotation at Top	0.0138 rad
Maximum Bending Moment	715 kip-ft
Location of Max. Moment	8 ft
Maximum Shear	73.7 kips