FOUNDATION INVESTIGATION UMAUMA STREAM BRIDGE REHABILITATION ROUTE 19, M.P. 16.02 NORTH HILO, HAWAII

for

NAGAMINE OKAWA ENGINEERS, INC.

HIRATA & ASSOCIATES, INC.
W.O. 10-4890
April 28, 2011

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April 28, 2011
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Mr. Norman Nagamine
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Dear Mr. Nagamine:

Our report, "Foundation Investigation, Rehabilitation of Umauma Stream Bridge, Route 19, M.P. 16.02, North Hilo, Hawaii" dated April 28, 2011, our Work Order 10-4890 is enclosed. This investigation was conducted in general conformance with the scope of work presented in our proposal dated May 14, 2008.

Our borings drilled behind the existing abutments encountered fill consisting of mottled brown clayey silt with sand and gravel below the existing pavement section. The clayey silt was in a medium stiff condition, and extended to depths of about 27 feet on the Hilo side of the bridge and to about 12 feet on the Honoka'a side. Portions of the clayey silt fill also appear to be mixed with volcanic ash. Underlying the fill was brown to mottled brown completely weathered rock in a medium stiff/medium dense to dense condition. Hard basalt was encountered at depths of about 36 and 47 feet, extending down to the maximum depths drilled.

Borings drilled near the piers encountered basalt at depths ranging from ground surface at Pier 1, to about 13 feet and 11 feet at Piers 2 and 3, respectively. The basalt was hard, fractured, and moderate to slightly weathered with occasional highly weathered seams. Overlying the basalt was brown to mottled brown clayey silt derived from volcanic ash.

Spread footing foundations are recommended for support of the new Piers 1 and 2. Due to the location of Pier 3, micropiles are recommended for support of the new pier. 5-foot diameters drilled piers behind the abutments are recommended to provide increased lateral support for the abutments.

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.

Paul S. Morimoto

Paul S. Morimoto President

PSM:CCT

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FOUNDATION INVESTIGATION
UMAUMA STREAM BRIDGE REHABILITATION
ROUTE 19, M.P. 16.02
NORTH HILO, HAWAII

INTRODUCTION

This report presents the results of our foundation investigation performed for the proposed rehabilitation of Umauma Stream Bridge in North Hilo, Hawaii. Our services for this study included the following:

- A visual reconnaissance of the site 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 5 exploratory test borings to depths ranging from about 48 to 76.5 feet. A description of our field investigation is summarized on Plates A1.1 and A1.2. The approximate exploratory test boring locations are shown on the enclosed Boring Location Plans, Plates A2.2 and A2.3, and the soils encountered in the borings are described on the Boring Logs, Plates A4.1 through A4.14.

- Laboratory testing of selected soil samples. Testing procedures are presented in the Description of Laboratory Testing, Plates B1.1 through B1.3. Test results are presented in the Description of Laboratory Testing, on the Boring Logs, Consolidation Test reports (Plates B2.1 through B2.3), Direct Shear Test reports (Plates B3.1 through B3.6), Modified Proctor Test reports (Plates B4.1 through B4.3), California Bearing Ratio Test reports (Plates B5.1 and B5.2), Sieve Analysis Test report (Plate B6.1), R-value Test reports (Figures B7.1 and B7.2), and Rock Core Unconfined Compression Test report (Plate B8.1).

- Engineering analyses of the field and laboratory data.

- Preparation of this report presenting geotechnical recommendations for the design of new foundations, including seismic considerations, resistance to lateral pressures, and site grading.

PROJECT CONSIDERATIONS

The existing Umauma Stream Bridge was initially built in 1911 and subsequently widened on both the upstream and downstream sides in 1949. The bridge is presently approximately 280 feet long and 35 feet wide. The structure is supported by 2 concrete abutments and 3 steel towers. The maximum span length is about 66 feet.

The steel towers are deteriorating, and the proposed rehabilitation concept consists of designing new concrete piers to structurally replace the existing towers. The new piers will be constructed within the towers and the existing steel structures will remain.

The rehabilitation will also include widening the bridge to allow for 12-foot lanes and 8-foot shoulders. Grading for the project will consist primarily of excavations necessary for construction of the new foundations.

Based on the bottom of footing elevations of the existing bridge foundations and boring logs on the 1949 as-built plans, the footings are expected to be founded on decomposed rock, soft and hard rock, except for Abutment No. 1 which might be founded on a layer of fill underlain by decomposed rock at shallow depths.

SITE CONDITIONS

Umauma Stream Bridge is located along Hawaii Belt Road (Route 19), between its intersection with Kauniho and Leopolino Roads in North Hilo. The bridge is approximately 280 feet in length, with Umauma Stream flowing about 115 feet below the bridge deck. The sides of the gully are steep, generally sloping at gradients of about 5/8H:1V, with some areas as steep as near vertical located at the bottom of the slope. Most of the slope areas are covered by a moderate growth of vegetation. The upper section of the slope faces generally expose weathered rock in areas that are bare, while steeper areas in the lower sections expose slight to moderately weathered basalt. Rock outcrops, along with numerous boulders are visible at the bottom of the gully, adjacent to the stream.

Existing cut slopes along the highway behind Abutment No. 2 generally stand at gradients on the order of 1/2H:1V or steeper and expose completely to highly weathered rock at the slope face.

SOIL CONDITIONS

Borings B1 and B2 drilled behind the existing abutments encountered fill consisting of mottled brown clayey silt with sand and gravel below the existing pavement section. The clayey silt was in a medium stiff condition and extended to depths of about 27 feet on the Hilo side of the bridge and to about 12 feet on the Honoka'a side.

Portions of the clayey silt fill also appear to be mixed with volcanic ash. Volcanic ash is generally characterized as having low dry density, high insitu moisture contents, and poor workability.

Underlying the fill was brown to mottled brown completely weathered rock. Completely weathered rock is defined as rock which has decomposed to soil, but with its fabric and structure preserved. The weathered rock encountered in the borings were in a medium stiff or medium dense to dense condition.

Basalt was encountered at depths of about 36 and 47 feet, extending down to the maximum depths drilled. The basalt was hard, fractured, and moderate to slightly weathered with occasional highly to completely weathered seams.

Borings B3 through B5, drilled near the piers, encountered basalt at depths ranging from ground surface at boring B3 (Pier 1), to depths of about 13 feet at boring B4 (Pier 2), and about 11 feet at boring B5 (Pier 3). The basalt was hard, fractured, and moderate to slightly weathered with occasional highly weathered seams and clinker down to the maximum depths drilled. Overlying the basalt was brown to mottled brown clayey silt derived from volcanic ash. The soil was in a medium stiff condition and mixed with sand and gravel.

Boring B3 drilled adjacent to the stream encountered groundwater at a depth of 29 feet. Neither groundwater nor seepage water was encountered in the remainder of the borings.

CONCLUSIONS AND RECOMMENDATIONS

Based on our test borings, and the existing topography, spread footings are recommended for support of new foundations at Piers 1 and 2. Since, Pier 3 is situated on at small flat area on a steep slope, micropile foundations are recommended for support of the new concrete pier.

Although cavities were not encountered in our test borings, we recommend, as a precautionary measure, that a probing and grouting program be implemented prior to construction of the foundations at Piers 1 and 2. All footing excavations should be probed to depths at least twice the footing width or to a minimum depth of 10 feet, measured from the bottom of footing elevation. All probe holes should be filled with sand-cement grout.

Underpinning and/or shoring of existing foundations may be required for construction of new foundations. Shoring of cuts extending into existing slopes may also be required for construction of the new foundations at Piers 1 and 2, and the pile cap at Pier 3.

Abutments

Foundations - We understand that existing abutment foundations will be re-used for the widened bridge. The existing abutment footings vary from about 10 to 14 feet in width. Abutment No. 1 is expected to be founded on a thin layer of fill underlain by completely weathered rock/clayey silt at shallow depths, and Abutment No. 2 is expected to be founded on completely weathered rock. The existing footings may be evaluated using bearing values of 6,000 and 13,000 pounds per square foot for strength limit states and extreme event limit states, respectively. A bearing value of 4,000 pounds per square foot may be assumed for service limit states.

We believe that settlement of existing abutment foundations due to loading from the existing bridge deck is complete. Additional settlement due to the added weight of the widened bridge deck is expected to be about 1 inch or less. Much of the settlement is expected to occur during construction, upon initial application of loads.

Lateral Design - Resistance to lateral loading may be provided by friction acting at the base of abutment foundations and by passive earth pressure acting on the buried portions of foundations.

Coefficients of friction of 0.45 and 0.53 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 may be computed as an equivalent fluid having a density of 220 and 440 pounds per cubic foot for strength limit state and extreme event limit state, respectively. The recommended passive earth pressure values are for level ground fronting the foundation. The passive earth pressure should be reduced or disregarded where the ground fronting the foundations slopes downward. Unless covered by pavement or concrete slabs, the upper 12 inches of soil should not be considered in computing lateral resistance.

For active earth pressure considerations, equivalent fluid pressures of 40 and 55 pounds per cubic foot per foot of depth may be used for freestanding level backfill and restrained level backfill conditions, respectively.

For dynamic lateral earth pressure considerations, a dynamic lateral force of $22H^2$ pounds per lineal foot of wall length may be used for level backfill conditions where walls are free to move laterally up to 1 to 2 inches or rotate in the event of an earthquake. The dynamic lateral force may be assumed to act through the mid-height of the wall.

Abutment Stiffness - An abutment backfill stiffness of 4 kips per square foot per inch of deflection may be assumed for resistance to lateral loads in the longitudinal direction during a seismic event. Maximum lateral resistance of the abutment backfill should be limited to 5 kips per square foot.

Drilled Shafts - Drilled shafts may also be used to provide additional lateral resistance at the abutments. Recommendations are based on the use of 5-foot diameter drilled shafts. Based on preliminary design, a row of 4 drilled shafts will be constructed behind Abutment No. 1 and a row of 3 drilled shafts will be constructed behind Abutment No. 2. The drilled shafts at Abutment No. 1 will be spaced about 14 and 18.5 feet apart, and the drilled shafts at Abutment No. 2 will be spaced 12.5 feet apart.

Although the drilled shafts will be connected to the abutments, we understand that the intent of the drilled shafts is primarily to provide additional lateral support to the abutment in a seismic event.

Based on our test borings, hard basalt was encountered at depths of approximately 36 and 47 feet below road grade, and in order to avoid potential rigid body behavior of short shaft under lateral loads, we recommend that the drilled shafts be socketed a minimum 10 feet into hard basalt. The actual lengths of the drilled shafts will need to be determined during construction. For cost estimating purposes, drilled shaft lengths of about 40 and 50 feet may be assumed at Abutments Nos. 1 and 2, respectively.

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.

Lateral capacities of the drilled shaft in the direction pushed into the slope will be different from those pushed away from slope in the longitudinal direction. In addition, due to the close proximity of the drilled shafts to the abutment walls and footings, the passive wedge of the abutments and drilled shafts will overlap when pushed into the slope. As a result, for our analysis, soil resistance along the portion of drilled shaft above the existing abutment footings was reduced in computing the lateral resistance of the drilled shaft when pushed into the slope. However, lateral capacities of drilled shaft, ignoring the potential effects from the passive wedge of the abutment walls and footings are also provided for comparison.

For our analysis, an axial load of 75 kips was assumed. In addition, a concrete compressive strength of 5,000 psi and a cracked section equal to 50% of the gross uncracked section were used in the analysis.

Results of lateral load analyses for deflection of 0.5, 1, and 1.5 inches at the top of drilled shaft are presented on Plates C1.1 and C1.2.

Drilled Shaft Construction - Excavations for the drilled shafts can be expected to extend through surface soil, weathered rock, and hard rock. 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 hard basalt.

We do not expect that casing will be required for construction of the drilled shafts. However if the excavated walls of the drilled shafts are sloughing and subject to collapse, temporary, non-corrugated steel casing should be used. The use of permanent casing will not be allowed.

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. If water was allowed to accumulate at the bottom of the drilled shaft excavation, concrete placed below the water level should be tremied through a pipe discharging below the surface of fresh concrete

Load Testing - Since the drilled shafts will not need to support axial loads, static load testing of the drilled shafts will not be required.

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 60-inch diameter drilled shafts, we recommend a minimum of 5 equally spaced 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.

Piers 1 and 2

Foundations - Spread footings founded on hard basalt may be used to support the proposed concrete pier structures. Foundations may be designed for a bearing value of 13,000 pounds per square foot under strength limit state and 30,000 pounds per square foot under extreme event limit state. A bearing value of 10,000 pounds per square foot may be used to evaluate the design of the foundations at service limit state.

Footings should be embedded a minimum 12 inches into the stratum of hard basalt. The bottom of footing excavations should be thoroughly cleaned of loose material prior to placement of reinforcing steel and concrete. Less hard, completely weathered material exposed at the bottom of footing excavations should be removed down to hard rock and replaced with concrete. Footings located on, or near the top of slopes, should be embedded such that a minimum horizontal distance of 5 feet is maintained between the bottom edge of footing and slope face.

Settlement of footings founded directly on hard basalt is expected to be negligible.

Lateral Design - 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 0.6 and 0.7 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 hard basalt may be computed as an equivalent fluid having a density of 400 and 800 pounds per cubic foot for strength limit state and extreme event limit state, respectively. Unless covered by pavement or concrete slabs, the upper 12 inches of rock should not be considered in computing lateral resistance.

The recommended coefficients of friction and passive pressures assumed that the footing is poured neat against the hard basalt.

Probing and Grouting - Although not encountered in our test borings, cavities or voids can be expected in the underlying basalt strata. As precautionary measure, we therefore recommend that a probing and grouting program be implemented prior to construction of the foundations.

All footing excavations should be probed with a drill or air track hammer. Probe holes should be drilled for every 100 square feet of foundation area. The holes should be a minimum 2 inches in diameter and extend to depths at least twice the footing width or a minimum 10 feet below the bottom of footings.

All probe holes should be filled with low strength sand-cement grout pumped under low to moderate pressure discharged through a grout pipe starting at the bottom of the probe hole. Placement of thin-wall plastic pipes in probe holes may be necessary to prevent holes from caving. Areas encountering large clinker pockets or voids that consume large quantities of grout may require additional probe holes. Voids encountered at the bottom of foundation excavations should be exposed and filled with lean concrete.

Pier 3

Foundations - Although hard basalt was encountered in our test boring at a depth of about 11 feet at the site of Pier 3, the use of a spread footing is not recommended since the pier is situated on a steep slope. As a result, micropiles embedded into hard basalt are recommended for support of the new pier.

In general, micropiles consist of small-diameter, drilled, and grouted in-place piles. The load bearing capacity of a micropile is provided structurally by the steel reinforcement, and geotechnically by the soil-grout bond zone. The steel reinforcement may consist of standard concrete reinforcing steel bars, continuous-threaded steel bars, continuous-threaded hollow-core steel bars, steel pile casing, or a combination of steel casings and reinforcing steel bars. Construction of micropile foundations generally consist of drilling a borehole, placing the reinforcement, and grouting the bore hole.

For this project, 7-inch diameter (outside diameter) micropiles with permanent steel casing and a reinforcing bar at the center are recommended. The micropiles should extend through the surface clayey silt and completely weathered rock, and be embedded into the underlying hard basalt layer.

The permanent steel casing should have a minimum thickness of 0.45 inch. The steel casing should extended from the top of pile to about 36 inches into the bearing layer or a minimum 10 feet, and uncased thereafter. The intent of the steel casing is to provide confinement to the cement grout and added flexural stiffness to the micropile where the bending moment and shear stresses are expected to be high. The micropiles will derive most of their load bearing capacity in friction from rock-grout bond in the uncased section extending into the hard basalt. 7-inch diameter micropiles with 15 feet of rock-grout bond length may be designed to support axial bearing loads of 150 kips and 220 kips for strength limit state and extreme event limit state, respectively. The micropiles may be also designed for an uplift load resistance of 75 kips and 150 kips for strength limit state and extreme event limit state, respectively.

The micropiles should be spaced a minimum of 30 inches on centers. As indicated earlier, the micropiles should extend a minimum 18 feet into the hard basalt (3 feet cased length plus 15 feet rock-grout bond length). The actual piles lengths can be expected to vary between pile locations, however, for preliminary cost estimating purposes, a pile length of 25 feet may be assumed.

Settlement of micropiles embedded into hard basalt is expected to be negligible.

Micropile Construction - Hard basalt with occasional highly weathered seams and clinkers are expected underlying site at shallow depths. The selected micropile system should be able to drill through the surface soil and the underlying hard basalt. The micropile installation should include drilling and casing the hole to the tip elevation, cleaning out all loose material in the drilled hole, installation of the reinforcing bar, grouting under pressure, and pull-out of the casing in the bottom 15 feet of the hole.

The reinforcing bar should be centered in the micropile drilled hole by centralizers and should extend through the cased section down to the bottom of the hole. The drilled hole and casing should be completely grouted using a tremie pipe. Each micropile should be constructed in one continuous pour.

Micropile Load Tests - Prior to construction of production micropiles, we recommend that static load tests be performed on sacrificial micropiles to confirm the load bearing capacity of the subsurface soils, as well as to verify the adequacy of the contractor's drilling, installation, and grouting operations. Based on the project requirements, we recommend one pre-production uplift and one pre-production compression load test be performed.

The pile load tests, which tests the micropile in compression and tension, should be conducted in general conformance to ASTM D1143 "Quick" test procedures, and the pile should be loaded to at least 100 percent of the design compression and uplift loads at extreme event limit state. The location of the load test pile can be determined after review of the micropile layout plan. In addition, at least 10 percent of the production micropiles should also be proof tested during construction.

Lateral Design - Resistance to lateral loading at Pier 3 may be provided by the lateral resistance of the micropiles. In addition to vertical micropiles, battered micropiles are recommended to provide increase lateral support. We understand that 1H:2V battered micropiles will be used to provide lateral support in the transverse direction. Results of lateral load analyses based on load combinations and pile group configuration provided by the project structural engineer are presented on Plates C2.1 through C2.8. The project structural engineer should verify the structural capacity of the micropile to support the induced shear, moment, and stresses.

We understand that lateral support of the Pier 3 foundation in the longitudinal direction will be provided by horizontal ground anchors in the away from slope direction and by passive earth pressure in the into slope direction. Passive earth pressure may be computed as an equivalent fluid having a density of 220 and 440 pounds per cubic foot for strength limit state and extreme event limit state, respectively. The backfill around the pile cap should be well compacted or the concrete of the pile cap should be poured neat against undisturbed on site materials.

Ground Anchors - As indicated above, horizontal ground anchors will be used to provide lateral support in the longitudinal, out of slope direction. Based on our test borings, we anticipate that ground anchors installed behind Pier 3 will encounter the surface soil, weathered rock, and hard, moderately weathered basalt. An average soil-grout bond strength of 1,500 pounds per square foot and a resistance factor of 0.7 may be assumed for design. We recommend that ground anchors be designed with a minimum unbonded length of 15 feet. The anchor bond length should also be a minimum 15 feet in length. A minimum anchor spacing of 5 feet on centers is recommended. Anchors should be designed at a minimum declination of 15 degrees from horizontal. All ground anchors should be proof tested during construction.

Seismic Design

Recommendations for Site Class classification and design response spectrum are presented on Plates D1.1 and D1.2.

Bridge Approach Slabs

Approach slabs behind the bridge abutments are recommended. 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).

Design Scour at Piers 1 and 2

Based on our laboratory test results, a D_{50} of 1 millimeter and a D_{90} of 38 millimeters may be assumed for the surface soil above the hard basalt at Piers 1 and 2. Based on our borings, the average Rock Quality Designation (RQD) of the basalt cores in the upper section of the basalt layer is greater than 50 percent and the unconfined compression strength of the rock core is generally greater than 5000 psi. Based on the 1991 memorandum for FHWA titled "Scourability of Rock Formation", it is our opinion that the hard basalt at Piers 1 and 2 has a low erodibility potential.

Reinforced Soil Slopes

Temporary cuts into the existing steep slopes will be required for construction of the pier foundations and the cuts will be backfilled after construction of the foundations. Due to the area constrains, fill slope gradients as steep as 1H:1V will be required in order for the fill slope transitioned into the existing steep slopes. Based on the grading plans, the fill slopes, constructed over the pier foundations, will generally be on the order of about 15 to 18 feet in height and about 50 to 80 lineal feet in width.

In order to improve the stability of the backfill slopes, we recommend that the fill slopes be reinforced with geogrids. In general, geogrid reinforced slopes consist of fill slope with layers of geogrids used to strengthen the fill soil. Recommended geogrids for the new fill slope will consist of primary reinforcement and intermediate geogrids. The primary reinforcement geogrids will be used to strengthen the new fill slope and should have a minimum allowable tensile strength of 1,000 pounds per foot, such as the Tensar's UX1000HS or equivalent. The geogrids, spaced about 3 feet in vertical spacing, should be a minimum 12 feet in length or extending to the back of the fill slope which ever is less.

Intermediate geogrid layers, consisting of geogrids such as the Tensar's biaxial BX1100 or equivalent, should be a minimum 4 feet in length and sandwiched between the primary reinforcement layers. The intent of the intermediate geogrid layers is to ensure stability at the slope face.

The geogrids should be handled with care and placed in accordance with the manufacturer's recommendations. To provide continuity in reinforcement, the geogrids should be connected or spliced following the manufacturer's guidelines. Tracked construction equipment should not be operated directly on the geogrids. In general, a minimum of 6 inches of fill over the geogrids is recommended prior to operating any construction equipment over the geogrids.

The reinforced fil should consist of imported granular structural fill material with angle of internal friction of at least 34 degrees.

Site Grading

Site Preparation - The project site should be cleared of all vegetation, large tree roots, and other deleterious material. Prior to placement of fill, the existing ground should first be scarified to a depth of six inches, moistened 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). Due to the relatively high in-situ moisture contents and the poor workability associated with volcanic ash, compaction of the clayey silt derived from volcanic ash to the conventional 90 percent compaction will be difficult. In lieu of this, we recommend a minimum compaction standard for the subgrade soil, equivalent to 100 percent of the wet density determined at the soil's in-situ moisture content in areas exposing the clayey silt/volcanic ash at subgrade level. Underlying soft or loose soils, indicated by pumping conditions, should be removed and replaced with either approved onsite material or imported granular structural fill.

Structural Excavation - Temporary cuts exposing the clayey silt and completely weathered rock should be stable at gradients of 1H:1V or flatter for temporary conditions. Cuts extending into the underlying hard basalt should be able to stand at a steeper slope gradient of about 1/4H :1V or flatter. However, the contractor should be responsible for conforming to OSHA safety standards for excavations.

The excavation adjacent to existing foundations should be adequately shored to reduce the potential for damage to the structures caused by earth movement toward the excavation or loss of support due to undermining.

Onsite Fill Material - Due to its relatively high in-situ moisture contents and poor workability, the onsite surface clayey silt/volcanic ash will not be acceptable for reuse in structural fills and backfills for structures. Reuse of the onsite clayey silt/volcanic ash should be limited to general fill areas. All rock fragments larger than 6 inches in maximum dimension should be removed prior to reuse of the material.

Imported Fill Material - Imported structural fill should be well-graded, non-expansive granular material. Specifications for imported granular 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).

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

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 and micropiles, including all drilling and concrete placement operations, as well as load testing, (2) observe probing and grouting operations in foundation areas, (3) observe footing excavations prior to placement of reinforcing steel and concrete, (4) observe structural fill and backfill fill placement and perform compaction testing, (5) review and/or perform laboratory testing on import borrow to determine its acceptability for use in compacted fills, and (6) 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 test borings were made, and may not represent conditions at other times and locations.

This report was prepared specifically for Nagamine Okawa Engineers, Inc. and their sub-consultants for design of the Rehabilitation of Umauma Stream Bridge in North Hilo, Hawaii. 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 test 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 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 under this agreement, either express or implied.

Respectfully submitted,

HIRATA & ASSOCIATES, INC.

Con C. Truong, P.E.

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, 2012

APPENDIX A

FIELD INVESTIGATION

DESCRIPTION OF FIELD INVESTIGATION

GENERAL

The site was explored between March 2, 2010 and April 7, 2010, by performing a visual site reconnaissance and drilling 5 exploratory test borings to depths ranging from about 48 to 76.5 feet with a CME 55 truck-mounted drill rig and portable drilling equipments..

During drilling operations, the soils were continuously logged by our field engineer 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; the Unified Soil Classification and Rock Weathering Classification Systems are shown on Plates A3.2 and A3.3, respectively. The soils encountered are logged on Plates A4.1 through A4.14.

Boring locations were located in the field by measuring/taping offsets from existing site features shown on the plans. The accuracy of the boring locations shown on Plates A2.2 and A2.3 are therefore approximate, in accordance with the field methods used. Ground surface elevations at boring locations were estimated using a topographic survey map prepared by ControlPoint Surveying, Inc.

SOIL SAMPLING

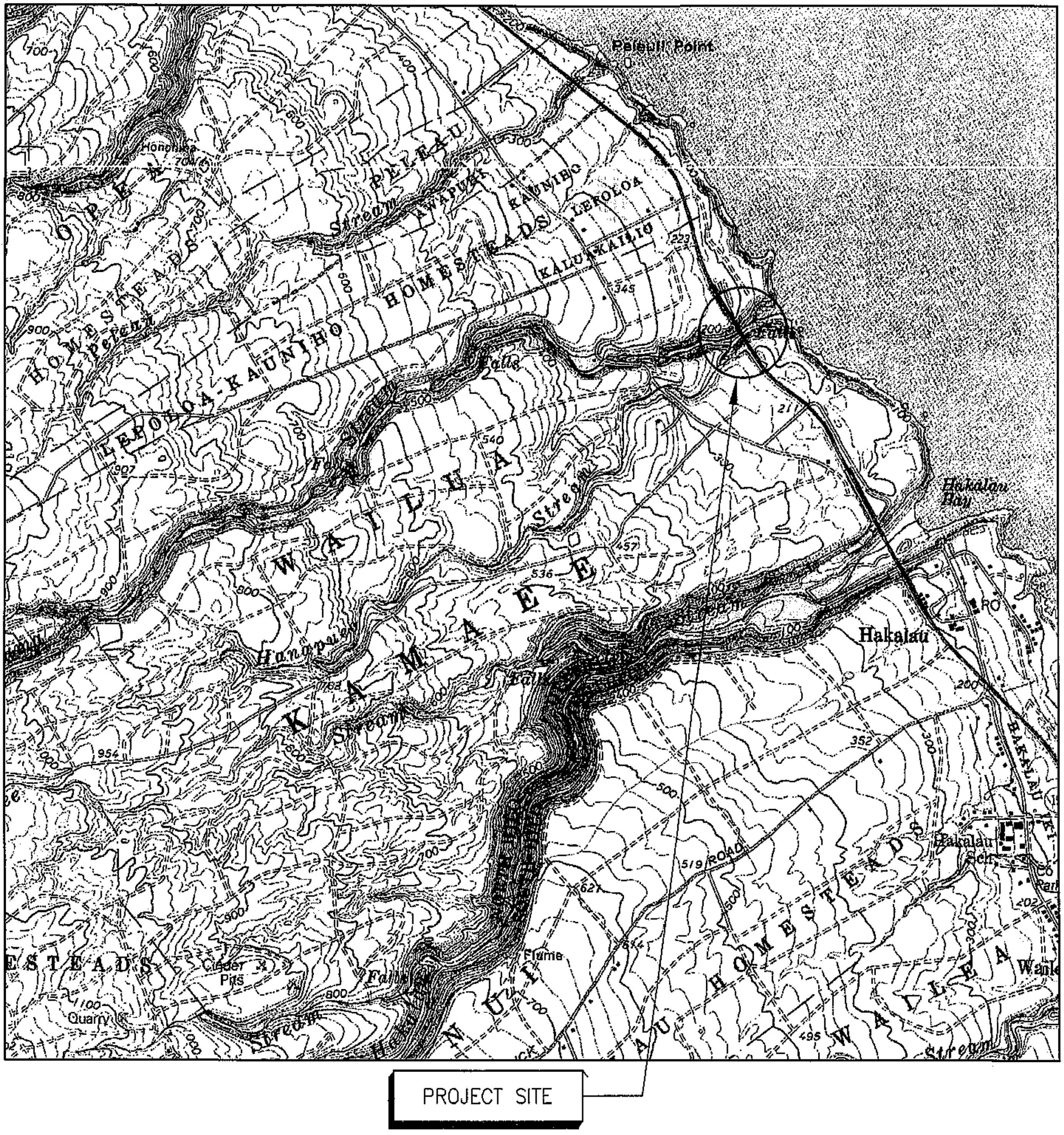
Representative soil samples and core samples of basalt and boulders were recovered from the borings for selected laboratory testing and analyses. Representative samples were recovered by driving a 3-inch O.D. split tube sampler a total of 18 inches with a 140-pound hammer dropped from a height of 30 inches. The number of blows required to drive the 3-inch O.D. split tube sampler the final 12 inches as well as blows counts from standard split spoon sampler 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. The rock quality designation (RQD) for each core run is also shown on the Boring Logs. This is a modified core recovery percentage which takes into account the number of fractures observed in the core samples. Only pieces of core 4 inches in length or longer, as measured along the centerline, were included in the determination of this modified core recovery percentage. Fractures caused by drilling or handling were ignored.

The following is a general correlation between RQD percentages and rock quality.

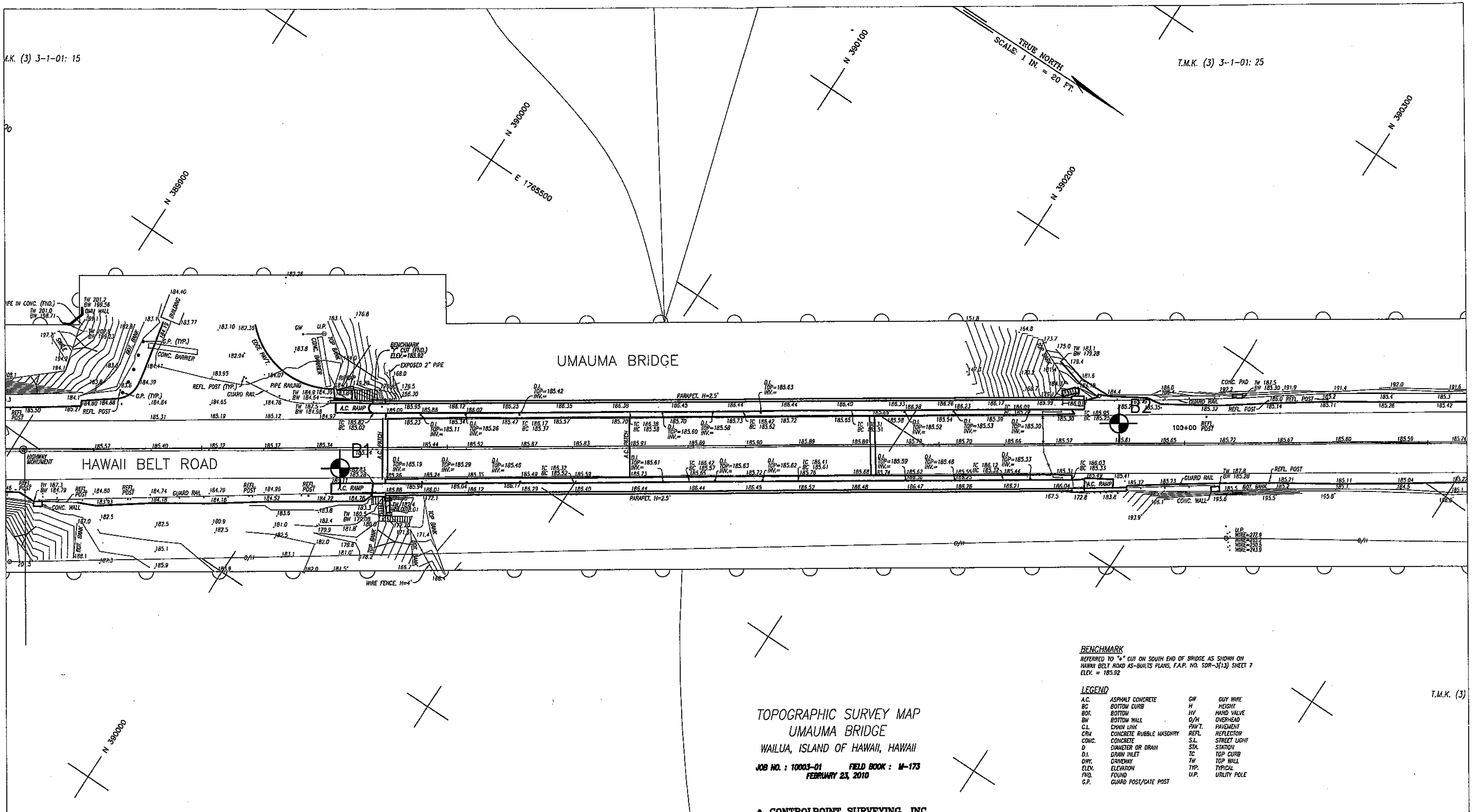
RQD (%)	Description of Rock Quality
0 - 25	Very Poor
25 - 50	Poor
50 - 75	Fair
75 - 90	Good
90 - 100	Excellent

Reference: Tunnel Engineering Handbook, Second Edition, edited by J.O. Bickel, T.R. Kuesel, and E.H. King, 1996.



Reference: Topographic quadrangle map prepared by the United States Department of the Interior Geologic Survey Papaaloa Quadrangle, Hawaii County, Hawaii. 1980.

W.O. 10-4890	Umauma Stream Bridge Rehabilitation, North Hilo
Hirata & Associates, Inc.	LOCATION MAP Plate A2.1



LEGEND:

Approximate location of borings

Reference: Topographic Survey Map prepared by ControlPoint Surveying, Inc.

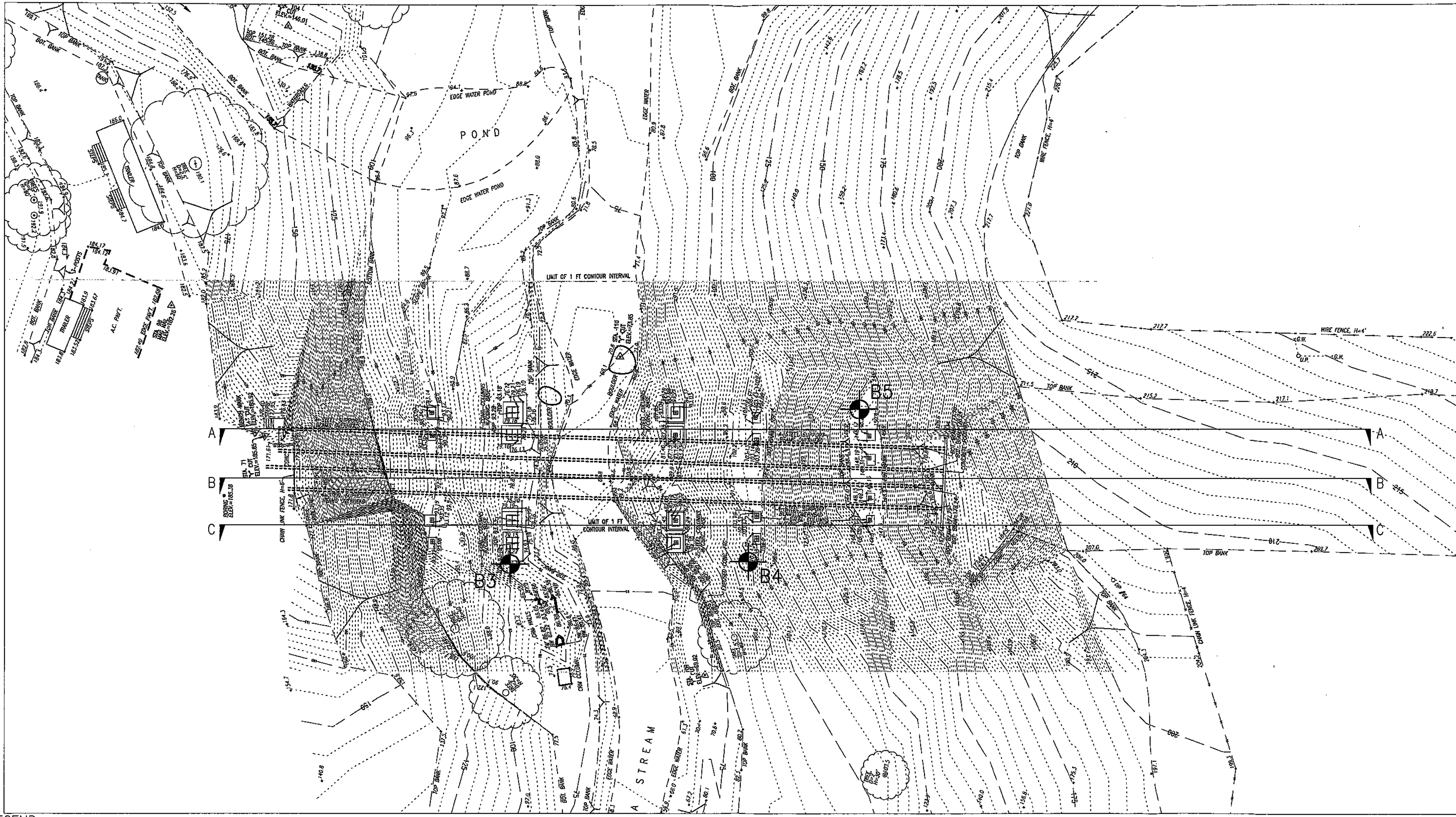
GRAPHIC SCALE:

0 20 40 80 FT.

1" = 40'

W.O. 10-4890	Umauma Stream Bridge Rehabilitation, North Hilo
Hirata & Associates, Inc.	BORING LOCATION PLAN

Plate A2.2



LEGEND:

Approximate location of borings

Reference: Topographic Survey Map prepared by ControlPoint Surveying, Inc.

GRAPHIC SCALE:

0 20 40 80 FT.

1" = 40'

W.O. 01-4890	Umauma Stream Bridge Rehabitation, North Hilo
Hirata & Associates, Inc.	BORING LOCATION PLAN

Plate A2.3

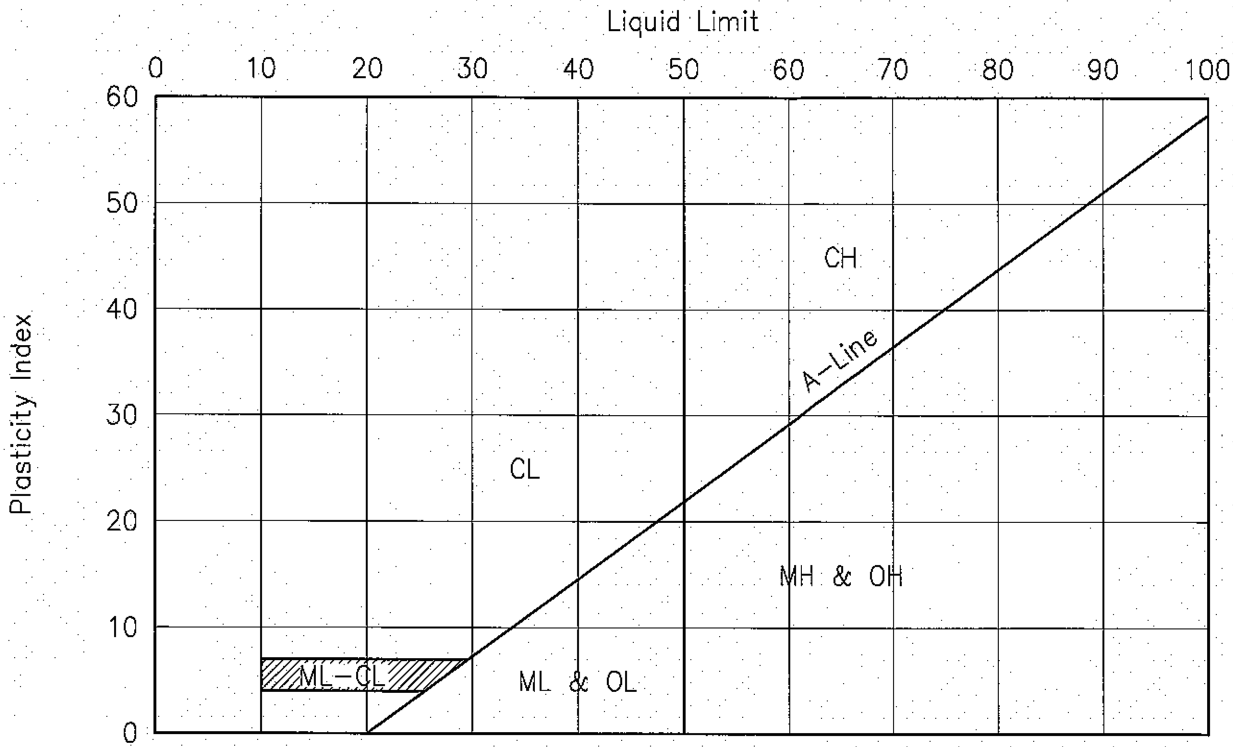
MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
COARSE GRAINED SOILS (More than 50% of the material is LARGER than No. 200 sieve size.)	GRAVELS (More than 50% of coarse fraction is LARGER than the No. 4 sieve size.)	CLEAN GRAVELS (Little or no fines.)	GW	Well graded gravels, gravel–sand mixtures, little 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 / PQ/ 4" Coring	Water Level

W.O. 10-4890	Umauma Stream Bridge Rehabilitation, North Hilo
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. 10-4890	Umauma Stream Bridge Rehabilitation, North Hilo
Hirata & Associates, Inc.	UNIFIED SOIL CLASSIFICATION SYSTEM Plate A3.2

Grade	Symbol	Description
Fresh	F	No visible signs of decomposition or discoloration. Rings under hammer impact.
Slightly Weathered	WS	Slight discoloration inwards from open fractures, otherwise similar to F.
Moderately Weathered	WM	Discoloration throughout. Weaker minerals such as feldspar decomposed. Strength somewhat less than fresh rock but cores cannot be broken by hand or scraped by knife. Texture preserved.
Highly Weathered	WH	Most minerals somewhat decomposed. Specimens can be broken by hand with effort or shaved with knife. Core stones present in rock mass. Texture becoming indistinct but fabric preserved.
Completely Weathered	WC	Minerals decomposed to soil but fabric and structure preserved (Saprolite). Specimens easily crumbled or penetrated.
Residual Soil	RS	Advanced state of decomposition resulting in plastic soils. Rock fabric and structure completely destroyed. Large volume change.

Reference: Soils Mechanics, NAVFAC DM-7.1, Department of the Navy, Naval Facilities Engineering Command, September, 1986.

W.O. 10-4890	Umauma Stream Bridge Rehabilitation, North Hilo
Hirata & Associates, Inc.	ROCK WEATHERING CLASSIFICATION SYSTEM Plate A3.3

HIRATA & ASSOCIATES, INC.

BORING LOG

W.O. 10-4890

BORING NO. B1 | DRIVING WT. 140 lb. | START DATE 3/2/10

SURFACE ELEV. 185±* | DROP 30 in. | END DATE 3/4/10

DEPTH	GRAPH	SAMPLE	BLOWS PER FOOT	DRY DENSITY (PCF)	MOIST. CONT. (%)	DESCRIPTION
0						Clayey SILT (MH) – Mottled brown, moist, medium stiff, with sand and gravel. (Fill) Covered by 8 inches of asphaltic concrete over 8 inches of base material.
		☐	11	76	34	
		☐	7	77	32	
5						
		☐	8	76	40	
10						
		☐	12	103	23	
15						
		☐	19	85	23	
20						
		☐	9	105	27	
25						
						Clayey SILT (MH) – Mottled brown, moist, medium stiff. (Completely Weathered Rock)
		☐	9	64	53	
30						

HIRATA & ASSOCIATES, INC.

BORING LOG

W.O. 10-4890

BORING NO. B1 (continued) DRIVING WT. 140 lb. START DATE 3/2/10
SURFACE ELEV. 185± DROP 30 in. END DATE 3/4/10

DEPTH	GRAPH	SAMPLE	BLOWS PER FOOT	DRY DENSITY (PCF)	MOIST. CONT. (%)	DESCRIPTION
30						
			14	62	59	
35						
						BASALT (WS) – Gray, dense to hard, fractured.
						Begin NX coring at 39 feet. 97% Recovery from 39 to 42 feet. RQD = 56%
40						
						80% Recovery from 42 to 47 feet. RQD = 48%
45						
						Highly weathered from 45.5 feet to 53 feet, dense to medium hard.
						25% Recovery from 47 to 52 feet. RQD = 0%
50						
			35/6" 50/2"			
						60% Recovery from 53.5 to 58.5 feet. RQD = 45%
55						
						Moderate to highly fractured from 57 feet.
						57% Recovery from 58.5 to 63.5 feet. RQD = 20%
60						

HIRATA & ASSOCIATES, INC.

BORING LOG

W.O. 10-4890

BORING NO.	B1 (continued)	DRIVING WT.	140 lb.	START DATE	3/2/10
SURFACE ELEV.	185±	DROP	30 in.	END DATE	3/4/10

DEPTH	GRAPH	SAMPLE	BLOWS PER FOOT	DRY DENSITY (PCF)	MOIST. CONT. (%)	DESCRIPTION
60						
						Highly fractured, with clinkers from 62 to 72 feet.
65						47% Recovery from 64.5 to 69.5 feet. RQD = 0%
70			35			
						70% Recovery from 71.5 to 76.5 feet. RQD = 28% moderately weathered, hard from 72 feet.
75						
						End boring at 76.5 feet.
						Neither groundwater nor seepage water encountered.
80						* Elevations based on topographic survey maps prepared by ControlPoint Surveying, Inc., dated February 23, 2010.
85						
90						

Plate A4.3

HIRATA & ASSOCIATES, INC.

BORING LOG

W.O. 10-4890

BORING NO. B2 | DRIVING WT. 140 lb. | START DATE 3/15/10

SURFACE ELEV. 185± | DROP 30 in. | END DATE 3/17/10

DEPTH	GRAPH	SAMPLE	BLOWS PER FOOT	DRY DENSITY (PCF)	MOIST. CONT. (%)	DESCRIPTION
0						Clayey SILT (MH) – Mottled brown, moist, stiff, with sand and gravel. (Fill) Covered by 7 inches of asphaltic concrete over 10 inches of base material.
		☐	42	96	30	
		☐	22	96	18	
5						
						Very moist at 6 feet.
		☐	17/6" 50/6"	84	37	
10						
						COMPLETELY WEATHERED ROCK – Mottled brown, moist, medium dense.
		☐	14	57	62	
15						
		☐	50/2"	Tip Recovery		Moderately weathered, dense to medium hard from 18 to 25 feet.
20						
		☐	32/6" 58/6"	105	16	
25						
		☐	17	76	46	
30						

Plate A4.4

HIRATA & ASSOCIATES, INC.

BORING LOG

W.O. 10-4890

BORING NO. B2 (continued) DRIVING WT. 140 lb. START DATE 3/15/10

SURFACE ELEV. 185± DROP 30 in. END DATE 3/17/10

DEPTH	GRAPH	SAMPLE	BLOWS PER FOOT	DRY DENSITY (PCF)	MOIST. CONT. (%)	DESCRIPTION
30						
		☐	25	74	33	
35						
		☐	22	58	82	
40						
		☐	50/3"	60	60	Dense to medium hard at 43 feet.
45						
						BASALT (WS) – Gray, hard, slightly weathered.
						Begin NX coring at 48 feet. 97% Recovery from 48 to 53 feet. RQD = 82%
50						
						60% Recovery from 53 to 58 feet. RQD = 40%
55						Clinker at 55 to 57 feet.
						95% Recovery from 58 to 63 feet. RQD = 72%
60						

Plate A4.5

HIRATA & ASSOCIATES, INC.

BORING LOG

W.O. 10-4890

BORING NO. B2 (continued) DRIVING WT. 140 lb. START DATE 3/15/10
SURFACE ELEV. 185± DROP 30 in. END DATE 3/17/10

DEPTH	GRAPH	SAMPLE	BLOWS PER FOOT	DRY DENSITY (PCF)	MOIST. CONT. (%)	DESCRIPTION
60						
						88% Recovery from 63 to 68 feet. RQD = 50%
65						
						100% Recovery from 68 to 70 feet. RQD = 88%
70						End boring at 70 feet.
						Neither groundwater nor seepage water encountered in the boring.
75						
80						
85						
90						

Plate A4.6

HIRATA & ASSOCIATES, INC.

BORING LOG

W.O. 10-4890

BORING NO. B3 | DRIVING WT. 140 lb. | START DATE 4/5/10
SURFACE ELEV. 76± | DROP 30 in. | END DATE 4/7/10

DEPTH	GRAPH	SAMPLE	BLOWS PER FOOT	DRY DENSITY (PCF)	MOIST. CONT. (%)	DESCRIPTION
0						BASALT (WS) – Gray, hard, slight to moderately fractured, slightly weathered. Begin NX coring from surface. 97% Percent recovery from 0 to 5 feet. RQD = 68%
5						100% Recovery from 5 to 10 feet. RQD = 72%
10						100% Recovery from 10 to 15 feet. RQD = 72% Moderate to highly fractured from 12 to 20 feet.
15						100% Recovery from 15 to 20 feet. RQD = 17%
20						100% Recovery from 20 to 25 feet. RQD = 97%
25						100% Recovery from 25 to 30 feet. RQD = 77% Brown, highly fractured, moderately weathered at 29 feet.
30						

Plate A4.7

HIRATA & ASSOCIATES, INC.

BORING LOG

W.O. 10-4890

BORING NO. B3 (continued) DRIVING WT. 140 lb. START DATE 4/5/10

SURFACE ELEV. 76± DROP 30 in. END DATE 4/7/10

DEPTH	GRAPH	SAMPLE	BLOWS PER FOOT	DRY DENSITY (PCF)	MOIST. CONT. (%)	DESCRIPTION
30						100% Recovery from 30 to 35 feet. RQD = 75%
35						100% Recovery from 35 to 40 feet. RQD = 82%
40						100% Recovery from 40 to 45 feet. RQD = 43% Moderately fractured, with clinkers from 41 to 50 feet.
45						100% Recovery from 45 to 50 feet. RQD = 42%
50						97% Recovery from 50 to 55 feet. RQD = 52% Reddish brown, highly weathered from 52 to 54 feet.
55						88% Recovery from 55 to 60 feet. RQD = 80%
60						

HIRATA & ASSOCIATES, INC.

BORING LOG

W.O. 10-4890

BORING NO. B3 (continued) DRIVING WT. 140 lb. START DATE 4/5/10
SURFACE ELEV. 76± DROP 30 in. END DATE 4/7/10

DEPTH	GRAPH	SAMPLE	BLOWS PER FOOT	DRY DENSITY (PCF)	MOIST. CONT. (%)	DESCRIPTION
60						100% Recovery from 60 to 65 feet. RQD = 32% Moderately fractured, with weathered seams from 62 to 64 feet.
65						97% Recovery from 65 to 70 feet. RQD = 72%
70						End boring at 70 feet. Groundwater encountered at 29 feet at 10:15 am on 4/8/10.
75						
80						
85						
90						

HIRATA & ASSOCIATES, INC.

BORING LOG

W.O. 10-4890

BORING NO. B4 | DRIVING WT. 140 lb. | START DATE 3/29/10

SURFACE ELEV. 100± | DROP 30 in. | END DATE 3/31/10

DEPTH	GRAPH	SAMPLE	BLOWS PER FOOT	DRY DENSITY (PCF)	MOIST. CONT. (%)	DESCRIPTION
0						Clayey SILT (MH) – Brown, moist, medium stiff, with gravel. (Volcanic Ash)
			10	53	47	
			10	66	41	
5						
			12	85	21	
10						
						Boulder at 11 feet.
						BASALT (WS) – Gray, hard, slightly weathered. Begin NX coring at 12.5 feet. 76% Recovery from 12.5 to 17.5 feet. RQD = 47% Moderately fractured from 12.5 to 17.5 feet.
15						
						93% Recovery from 17.5 to 22.5 feet. RQD = 52%
20						
						98% Recovery from 22.5 to 27.5 feet. RQD = 83%
25						
						100% Recovery from 27.5 to 32.5 feet. RQD = 95%
30						

HIRATA & ASSOCIATES, INC.

BORING LOG

W.O. 10-4890

BORING NO. B4 (continued) DRIVING WT. 140 lb. START DATE 3/29/10
SURFACE ELEV. 100± DROP 30 in. END DATE 3/31/10

DEPTH	GRAPH	SAMPLE	BLOWS PER FOOT	DRY DENSITY (PCF)	MOIST. CONT. (%)	DESCRIPTION
30						
						100% Recovery from 32.5 to 36.5 feet. RQD = 100%
35						
						100% Recovery from 37.5 to 42.5 feet. RQD = 100%
40						
						100% Recovery from 42.5 to 47.5 feet. RQD = 95%
45						
						100% Recovery from 47.5 to 52.5 feet. RQD = 95%
50						
						100% Recovery from 52.5 to 57.5 feet. RQD = 92%
55						
						Reddish brown, moderate to highly weathered from 56 to 63 feet. 100% Recovery from 57.5 to 62.5 feet. RQD = 28%
60						

Plate A4.11

HIRATA & ASSOCIATES, INC.

BORING LOG

W.O. 10-4890

BORING NO. B4 (continued)
DRIVING WT. 140 lb.
START DATE 4/5/10
SURFACE ELEV. 100±
DROP 30 in.
END DATE 4/7/10

DEPTH	GRAPH	SAMPLE	BLOWS PER FOOT	DRY DENSITY (PCF)	MOIST. CONT. (%)	DESCRIPTION
60						
						100% Recovery from 62.5 to 67.5 feet, RQD = 82%
65						
						End boring at 67.5 feet.
70						Neither groundwater nor seepage water encountered.
75						
80						
85						
90						

Plate A4.12

HIRATA & ASSOCIATES, INC.

BORING LOG

W.O. 10-4890

BORING NO. B5 | DRIVING WT. 140 lb. | START DATE 3/23/10
SURFACE ELEV. 147± | DROP 30 in. | END DATE 3/25/10

DEPTH	GRAPH	SAMPLE	BLOWS PER FOOT	DRY DENSITY (PCF)	MOIST. CONT. (%)	DESCRIPTION
0						Clayey SILT (MH) – Mottled brown, moist, medium stiff, with gravel. (Volcanic Ash)
			9	64	55	
5			5	53	72	
			17/6"	49	88	
10			35/6"			WEATHERED ROCK (WC) – Mottled brown, moist, medium dense to dense, completely weathered.
						BASALT (WS) – Gray, hard, slightly weathered. Highly to moderately weathered from 10.5 to 12 feet.
						Begin NX coring at 13 feet. 88% Recovery from 13 to 18 feet. RQD = 83%
15						
						100% Recovery from 18 to 23 feet. RQD = 33% Brown, highly weathered at 19 feet.
20						
						100% Recovery from 23 to 28 feet. RQD = 90%
25						
						92% Recovery from 28 to 33 feet. RQD = 47% Moderately fractured at 29 feet.
30						

Plate A4.13

HIRATA & ASSOCIATES, INC.

BORING LOG

W.O. 10-4890

BORING NO. B5 (continued) DRIVING WT. 140 lb. START DATE 3/23/10

SURFACE ELEV. 147± DROP 30 in. END DATE 3/25/10

DEPTH	GRAPH	SAMPLE	BLOWS PER FOOT	DRY DENSITY (PCF)	MOIST. CONT. (%)	DESCRIPTION
30						
33–38						98% Recovery from 33 to 38 feet. RQD = 75%
35						
38–43						100% Recovery from 38 to 43 feet. RQD = 37% Moderate to highly fractured, moderately weathered from 38 to 45 feet.
40						
43–48						100% Recovery from 43 to 48 feet. RQD = 78%
45						
48						End boring at 48 feet. Neither groundwater nor seepage water encountered.
50						
55						
60						

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. The final classifications are shown at the appropriate locations on the Boring Logs, Plates A4.1 through A4.14.

MOISTURE-DENSITY

Representative samples were tested for field 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 A4.14.

CONSOLIDATION

Selected representative samples were tested for their consolidation characteristics. Test samples were 2.42 inches in diameter and 1 inch high. Porous stones were placed in contact with the top and bottom of test samples 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 in 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 B3.6.

PROCTOR TESTS

Modified Proctor tests were performed in general accordance with ASTM D 1557 on bulk samples of near surface soils at selected boring locations. The test is used to determine the optimum moisture content at which the soil compacts to 100 percent density. Results are shown on Plates B4.1 through B4.3.

CALIFORNIA BEARING RATIO TESTS

CBR tests were performed on bulk samples of near surface soils. The tests were performed in general accordance with ASTM D 1883 but compacted to the soil's maximum wet density at its insitu moisture content. Results are shown on Plates B5.1 and B5.2.

SIEVE ANALYSIS

A sieve analysis test was performed on a representative soil sample in general accordance with ASTM D 422. Test results are presented on Plate B6.1.

R-VALUE TESTS

R-Value tests were performed on bulk samples of near surface soils. The tests were performed by Signet Testing Labs, Inc. in Hayward, California, in general accordance with ASTM D 2844. Test results are shown on Figures B7.1 and B7.2.

UNCONFINED COMPRESSION TESTS OF ROCK CORE

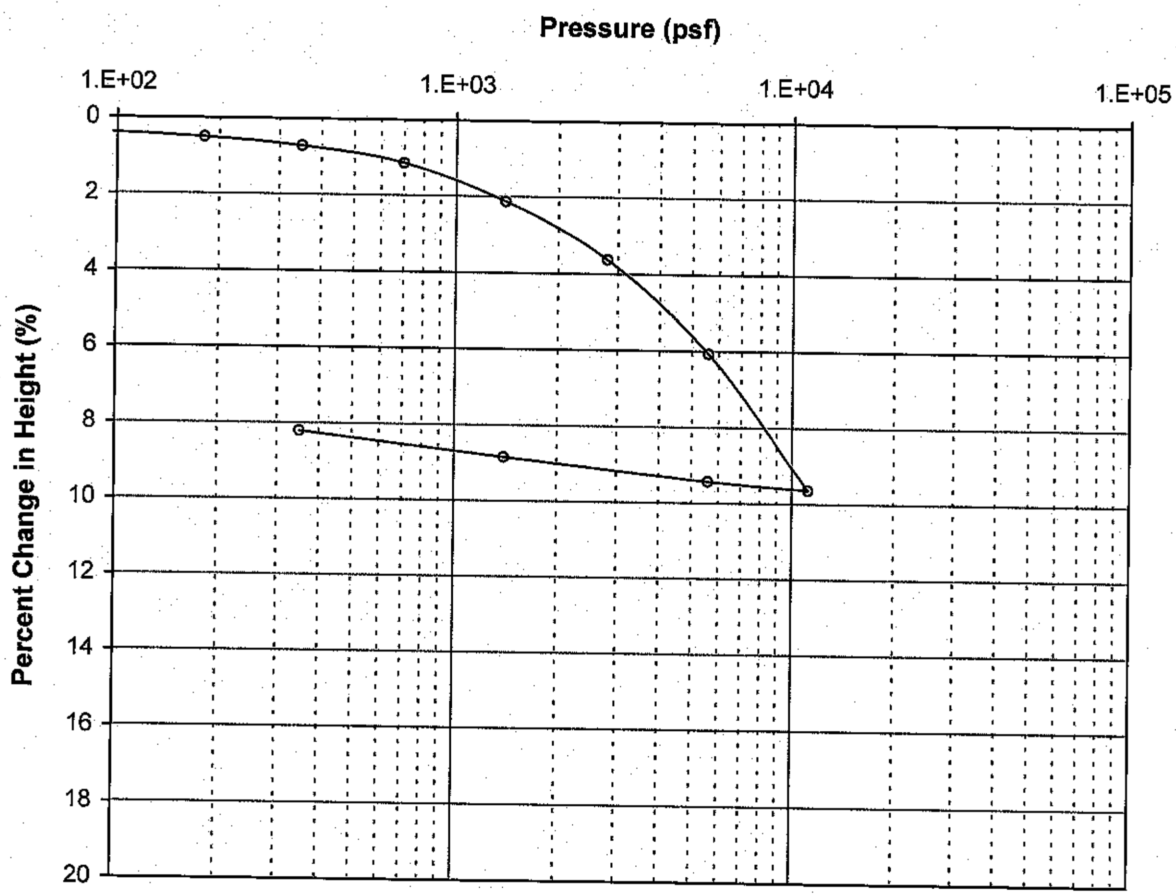
Unconfined compression tests were performed on selected basalt and boulder rock cores. The tests were performed by Construction Engineering Labs in Pearl City, Hawaii, in general accordance with ASTM D 2938. Test results are shown on Plate B8.1.

RESISTIVITY, pH, CHLORIDES, AND SULFATES TESTS

Four soil samples were tested for resistivity, pH, chlorides, and sulfates. The tests were performed by TestAmerica in Aiea, Hawaii. The following is a summary of the test results.

Sample	Resistivity (ohm-cm)	pH	Chlorides (ppm)	Sulfaces (ppm)
B2 @ 28'	11,800	7.25	14	16
B4 @ 4'	8,660	7.10	18	29
B4 @ 8'	9,280	7.32	11	11
B5 @ 4'	6,690	6.57	29	33

Consolidation Test Results



Sample Description

Boring No.: B1 Depth (ft): 28

Soil Description: Mottled brown clayey silt

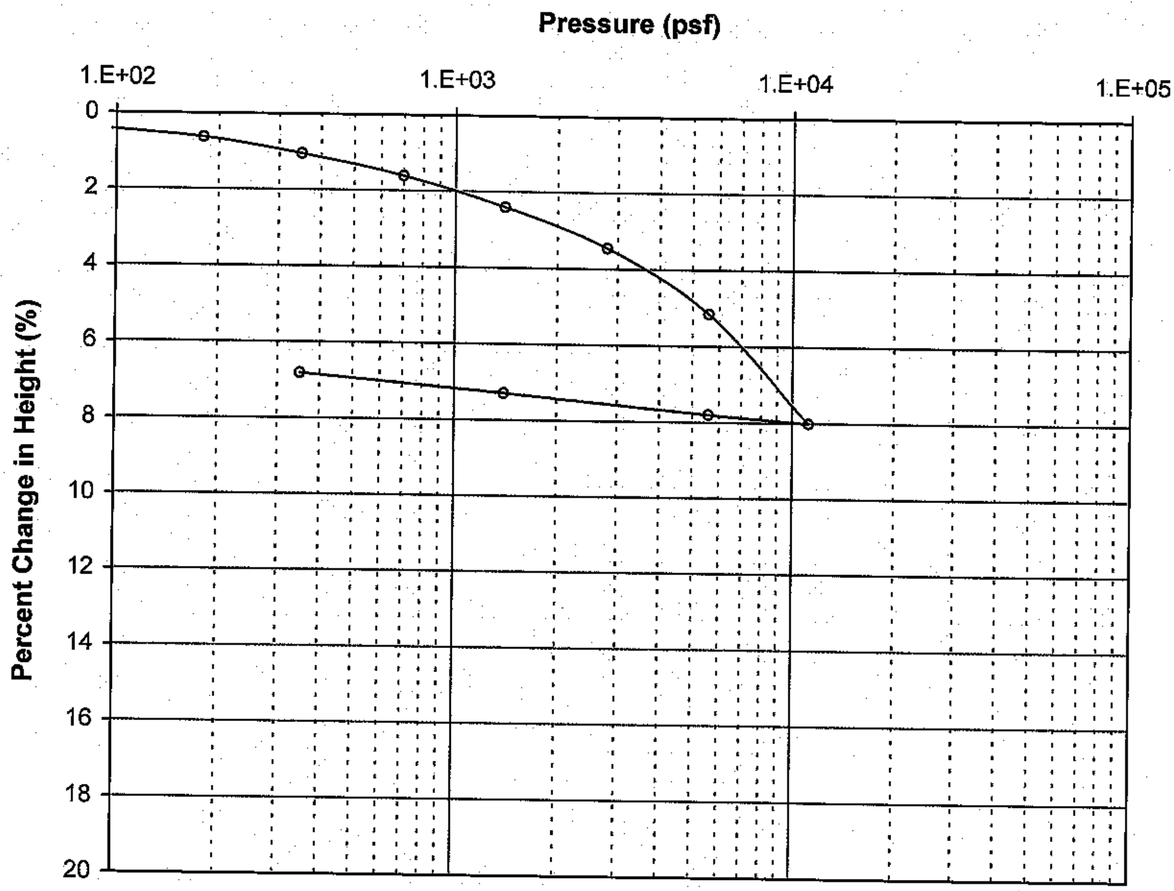
	Moisture Content (%)	Dry Density (pcf)
Initial	52.8	64.1
Final	48.7	69.8

Remark: 04/11/10

W.O. 10-4890	Umauma Stream Bridge Rehabilitation,North Hilo
Hirata & Associates, Inc.	**CONSOLIDATION TEST**

Plate B2.1

Consolidation Test Results



Sample Description

Boring No.: B2 Depth (ft): 13

Soil Description: Mottled brown completely weathered rock

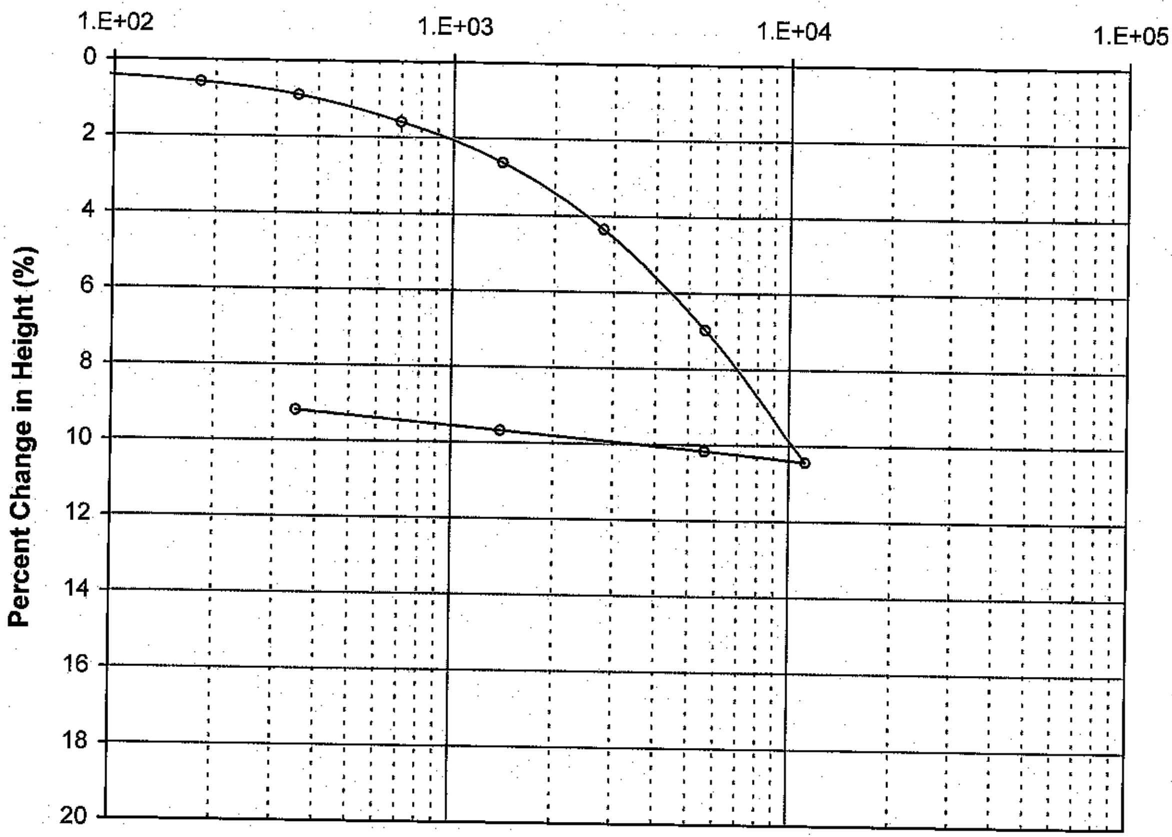
	Moisture Content (%)	Dry Density (pcf)
Initial	61.8	57.4
Final	54.0	61.6

Remark: 03/24/10

W.O. 10-4890	Umauma Stream Bridge Rehabilitation, North Hilo
Hirata & Associates, Inc.	CONSOLIDATION TEST

Plate B2.2

Consolidation Test Results



Sample Description

Boring No.: B4 Depth (ft): 4

Soil Description: Brown clayey silt

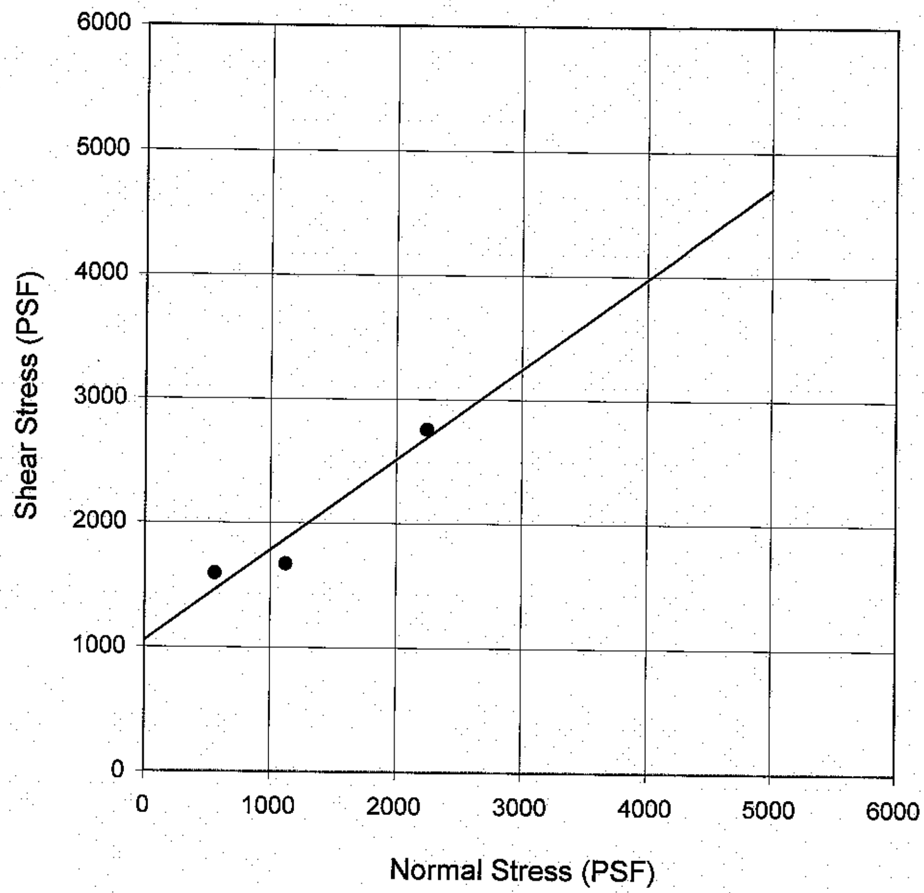
	Moisture Content (%)	Dry Density (pcf)
Initial	41.1	66.2
Final	35.6	72.9

Remark: 04/15/10

W.O. 10-4890	Umauma Stream Bridge Rahabilitation, North Hilo
Hirata & Associates, Inc.	CONSOLIDATION TEST

Plate B2.3

Direct Shear Test Results



Sample Description

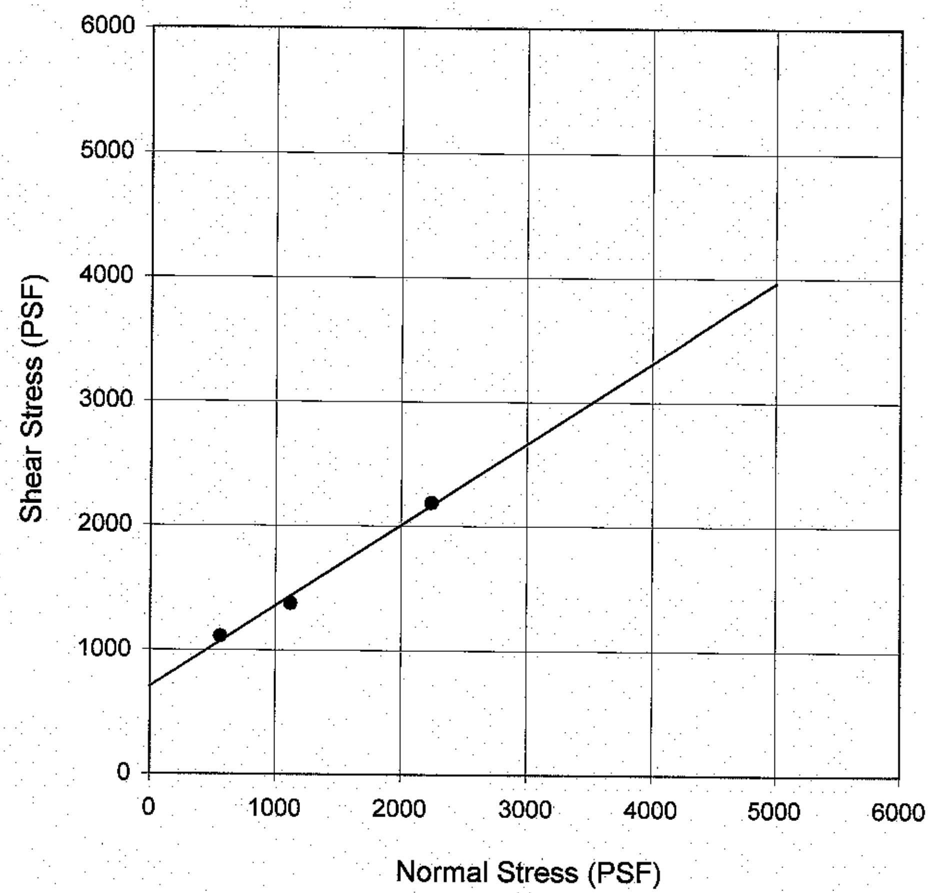
Boring No.: B1 Depth (ft): 13
Soil Description: Mottled brown clayey silt with sand and gravel
Strength Intercept (C): 1052.4 PSF
Friction Angle (ϕ): 36.2 DEG

Remark: 03/16/10

W.O. 10-4890	**Umauma Stream Bridge Rehabilitation, North Hilo**
Hirata & Associates, Inc.	**DIRECT SHEAR TEST**

Plate B3.1

Direct Shear Test Results



Sample Description

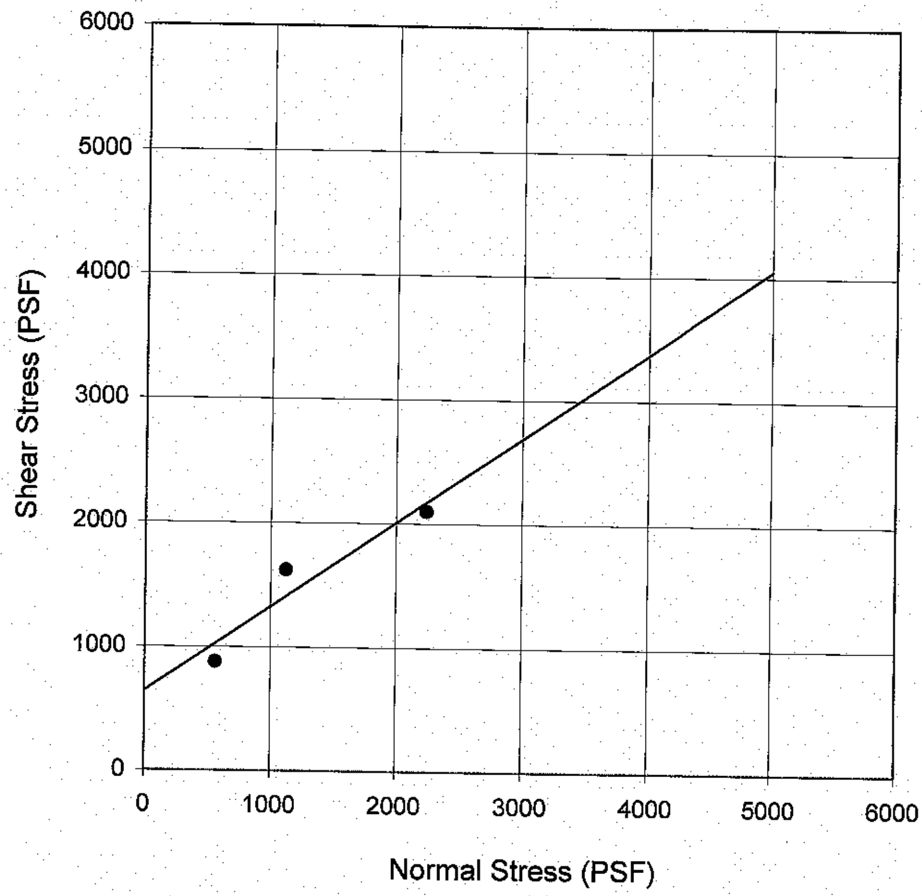
Boring No.: B1 Depth (ft): 33
Soil Description: Mottled brown clay silt
Strength Intercept (C): 705.0 PSF
Friction Angle (ϕ): 33.1 DEG

Remark: 03/16/10

W.O. 10-4890	Umauma Stream Bridge Rehabilitation, North Hilo
Hirata & Associates, Inc.	**DIRECT SHEAR TEST**

Plate B3.2

Direct Shear Test Results



Sample Description

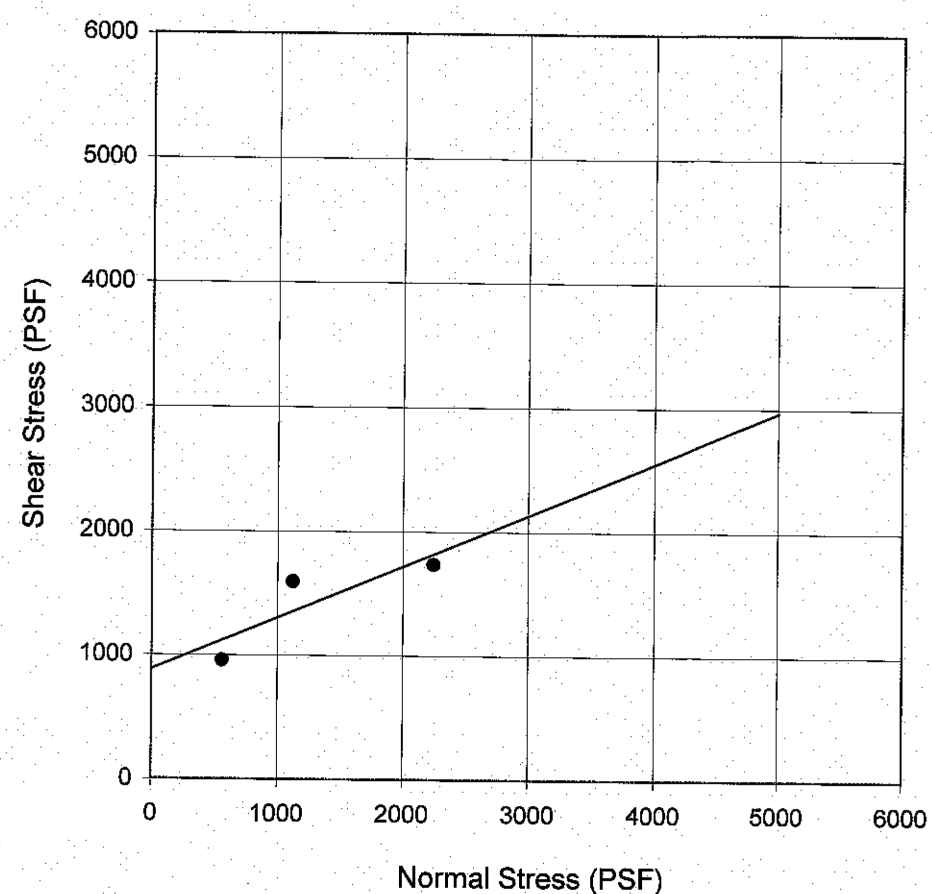
Boring No.: B2 Depth (ft): 4
Soil Description: Mottled brown clayey silt with gravel
Strength Intercept (C): 643.1 PSF
Friction Angle (ϕ): 34.3 DEG

Remark: 03/25/10

W.O. 10-4890	**Umauma Stream Bridge Rehabilitation, North Hilo**
Hirata & Associates, Inc.	**DIRECT SHEAR TEST**

Plate B3.3

Direct Shear Test Results



Sample Description

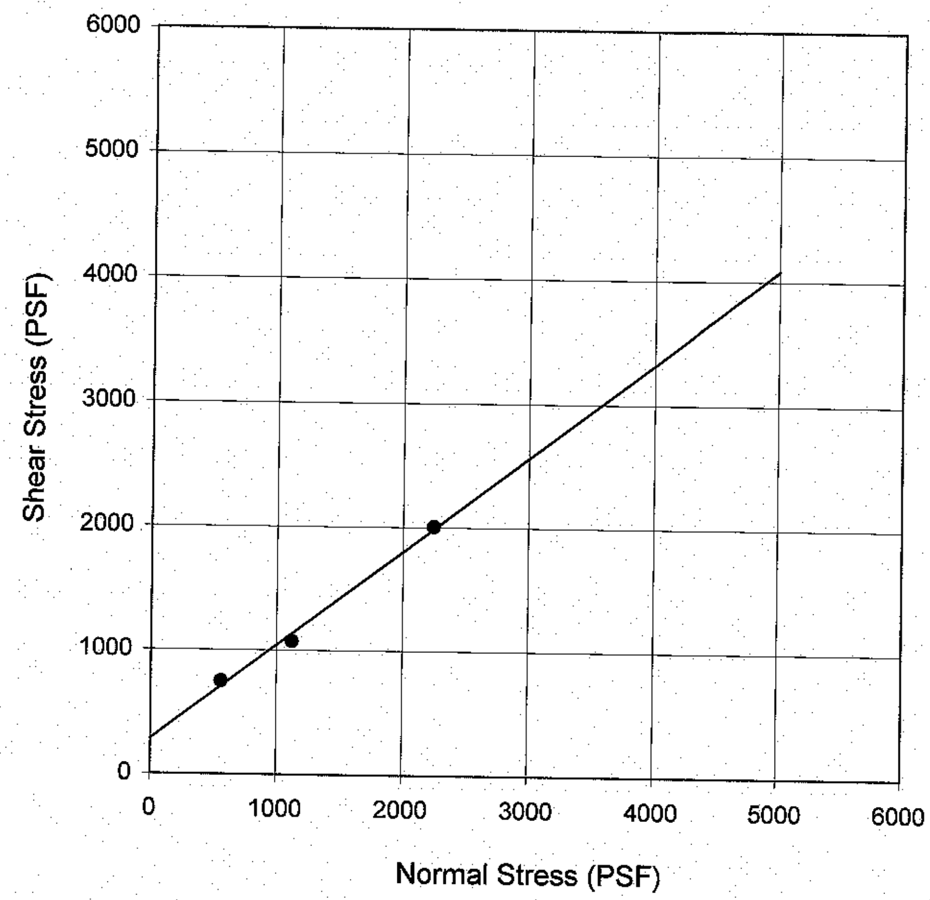
Boring No.: B2 Depth (ft): 28
Soil Description: Mottled brown completely weathered rock
Strength Intercept (C): 885.8 PSF
Friction Angle (ϕ): 22.6 DEG

Remark: 03/25/10

W.O. 10-4890	**Umauma Stream Bridge Rehabilitation, North Hilo**
Hirata & Associates, Inc.	**DIRECT SHEAR TEST**

Plate B3.4

Direct Shear Test Results



<u>Sample Description</u>

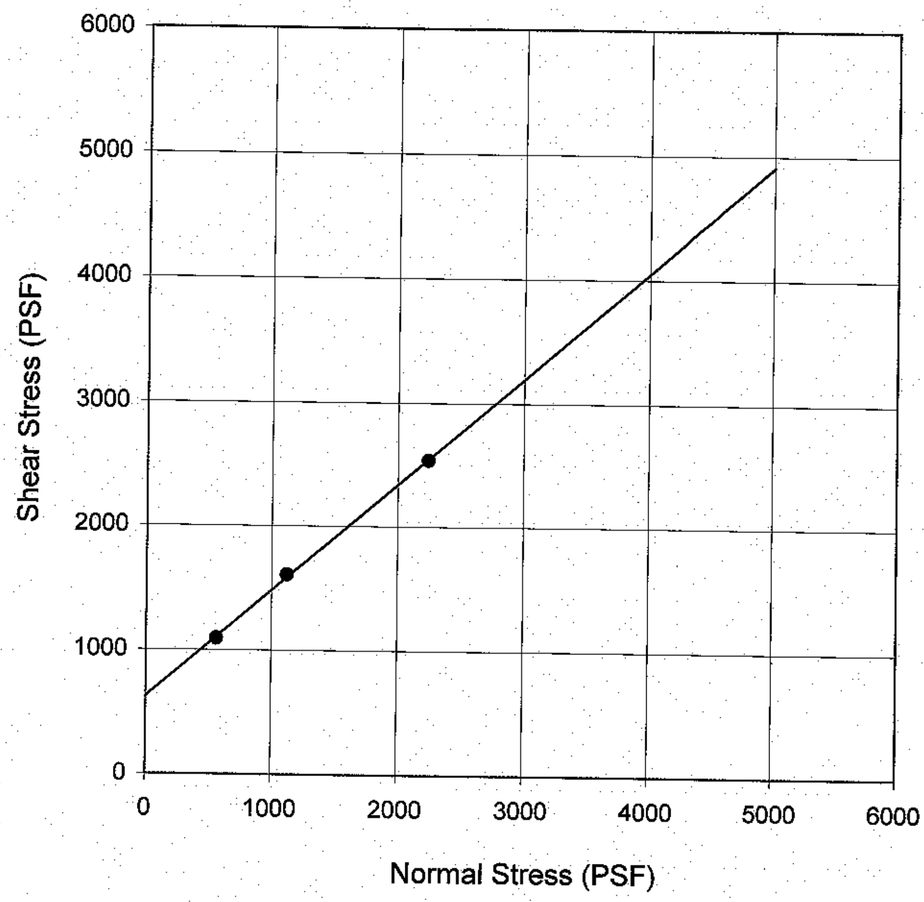
Boring No.: B4 Depth (ft): 2
Soil Description: Brown clayey silt with gravel
Strength Intercept (C): 281.4 PSF
Friction Angle (ϕ): 37.2 DEG

Remark: 04/14/10

W.O. 10-4890	Umauma Stream Bridge Rehabilitation, North Hilo
Hirata & Associates, Inc.	**DIRECT SHEAR TEST**

Plate B3.5

Direct Shear Test Results



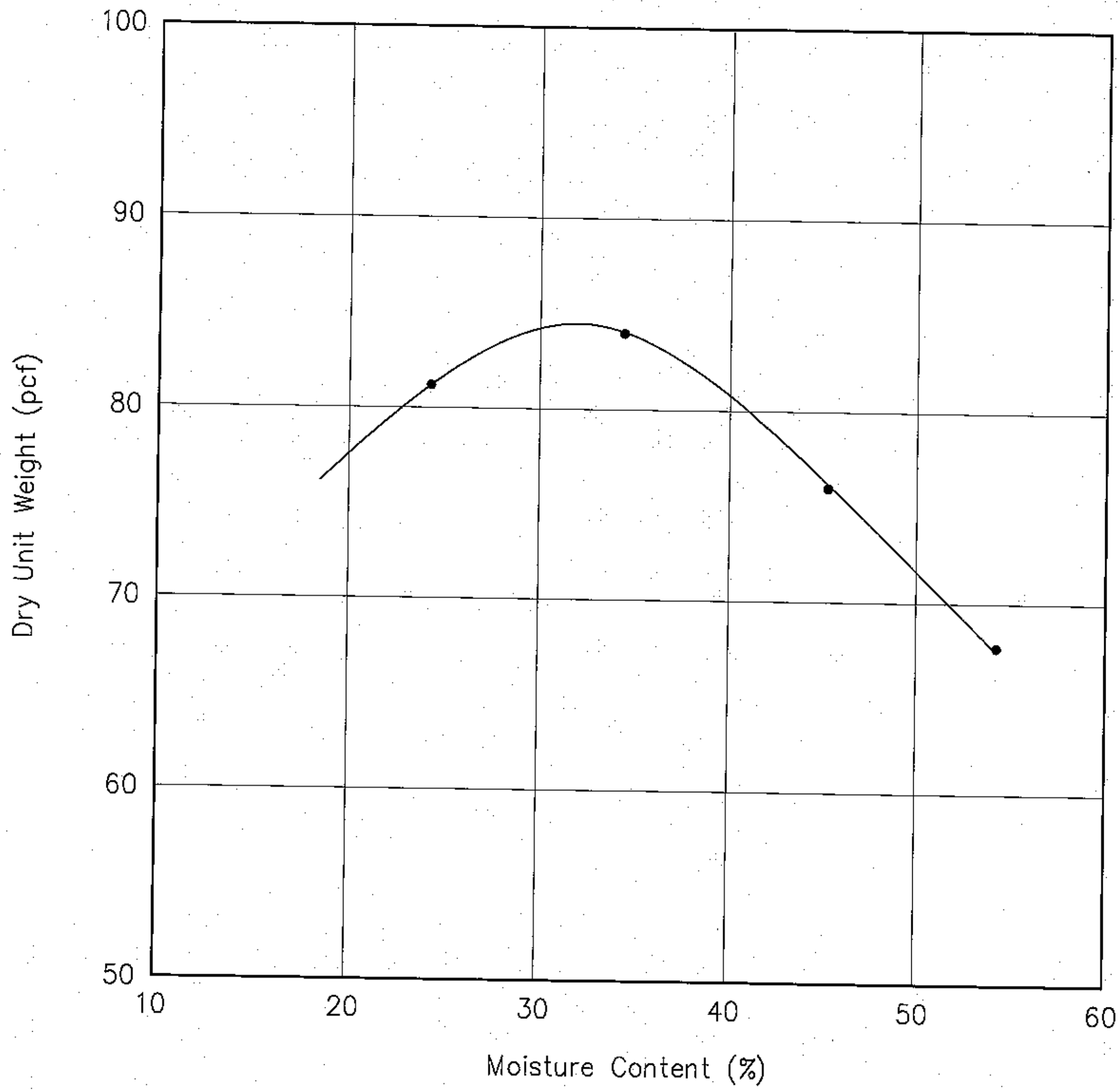
Sample Description

Boring No.: B5 Depth (ft): 2
Soil Description: Mottled brown clayey silt with gravel
Strength Intercept (C): 624.1 PSF
Friction Angle (ϕ): 40.6 DEG

Remark: 04/08/10

W.O. 10-4890	**Umauma Stream Bridge Rehabilitation, North Hilo**
Hirata & Associates, Inc.	**DIRECT SHEAR TEST**

Plate B3.6



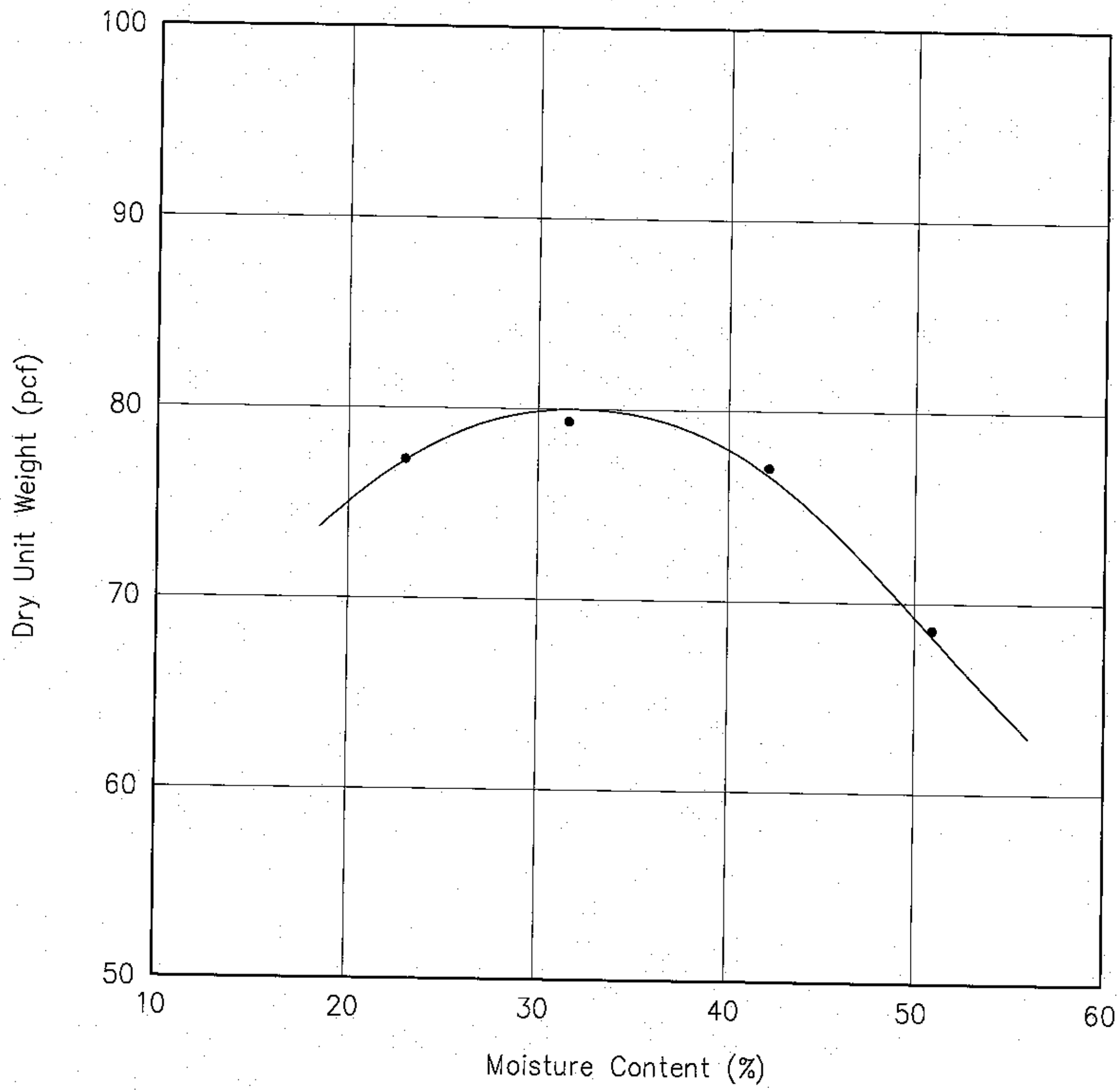
Soil Data

Location: Boring B1 at 2 to 4 ft
Description: Brown clayey silt with sand

Test Results

Maximum Dry Density: 84.5 pcf
Optimum Moisture Content: 32%

W.O. 10-4890	Umauma Stream Bridge Rehabilitation, North Hilo
Hirata & Associates, Inc.	MODIFIED PROCTOR CURVE Plate B4.1



Soil Data

Location: Boring B2 at 2 to 4 ft

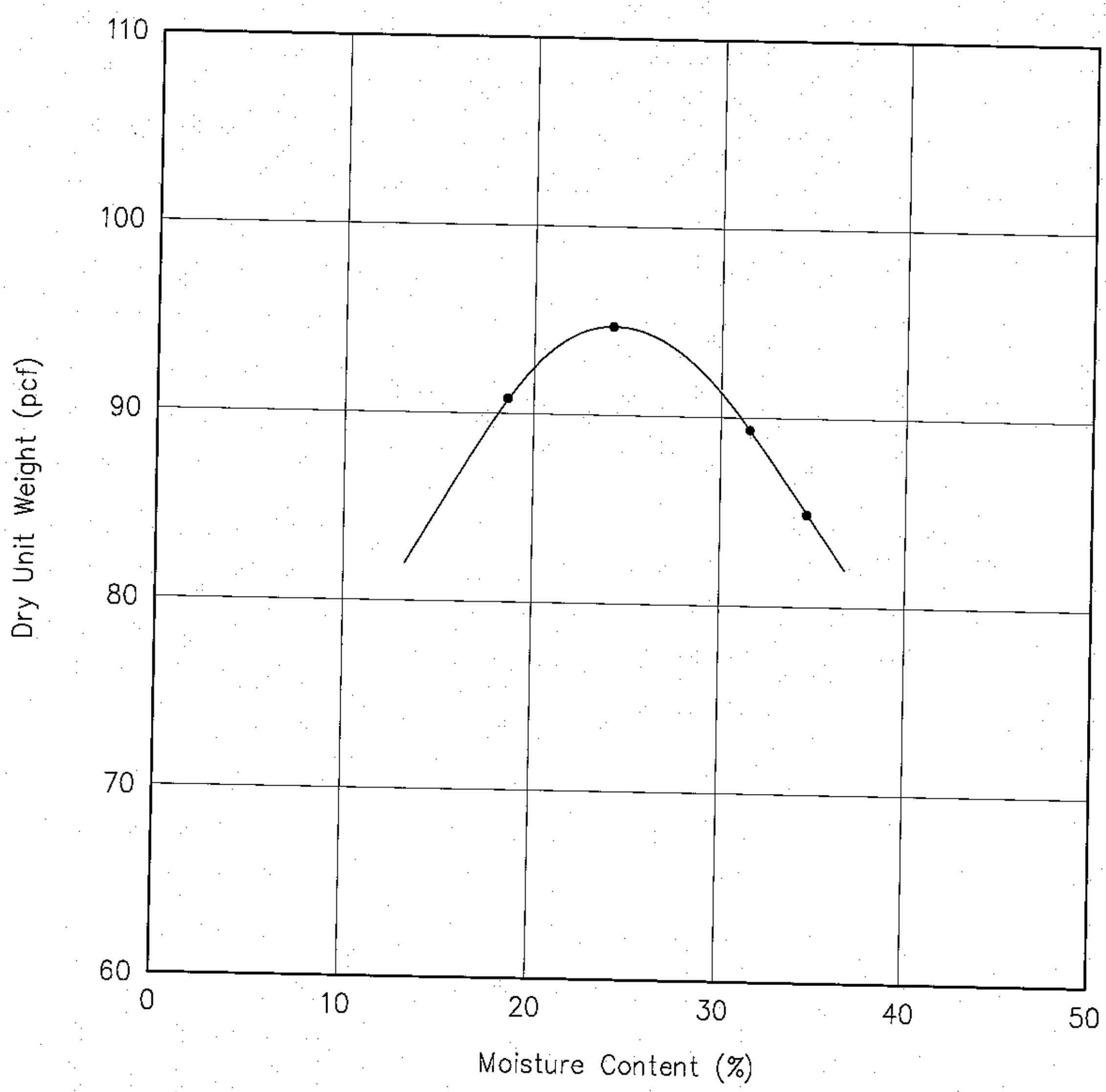
Description: Brown clayey silt with sand

Test Results

Maximum Dry Density: 80 pcf

Optimum Moisture Content: 32%

W.O. 10-4890	Umauma Stream Bridge Rehabilitation, North Hilo
Hirata & Associates, Inc.	MODIFIED PROCTOR CURVE Plate B4.2



Soil Data

Location: Boring B4 at near surface

Description: Brown clayey silt with weathered rock fragments

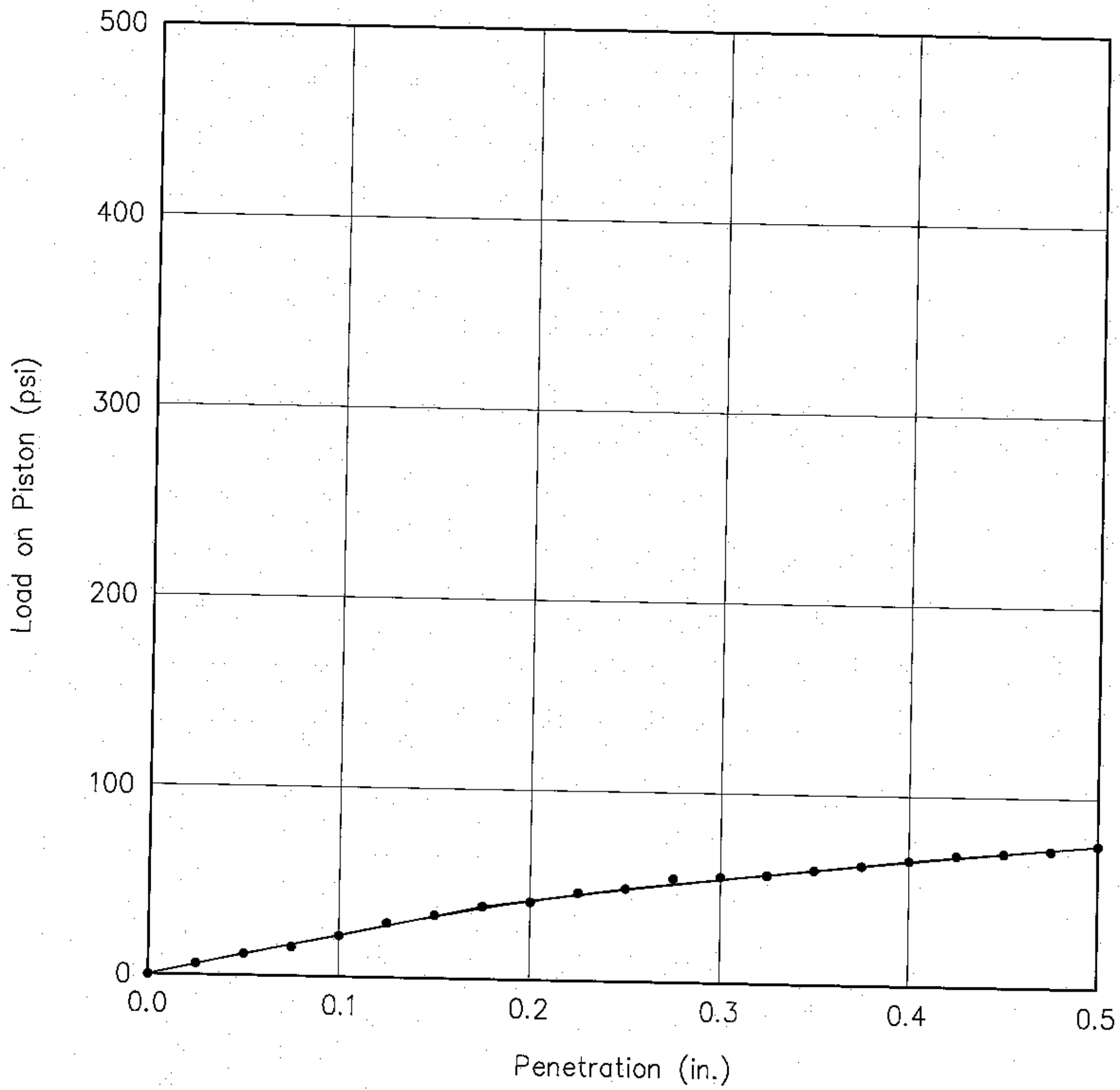
Test Results

Maximum Dry Density: 95 pcf

Optimum Moisture Content: 25%

W.O. 10-4890	Umauma Stream Bridge Rehabilitation, North Hilo
Hirata & Associates, Inc.	MODIFIED PROCTOR CURVE

Plate B4.3



Soil Data

Location:	Boring B1 at 2 to 4 ft
Description:	Brown clayey silt
Sample Dry Density:	74 pcf
Sample Moisture Content:	45%

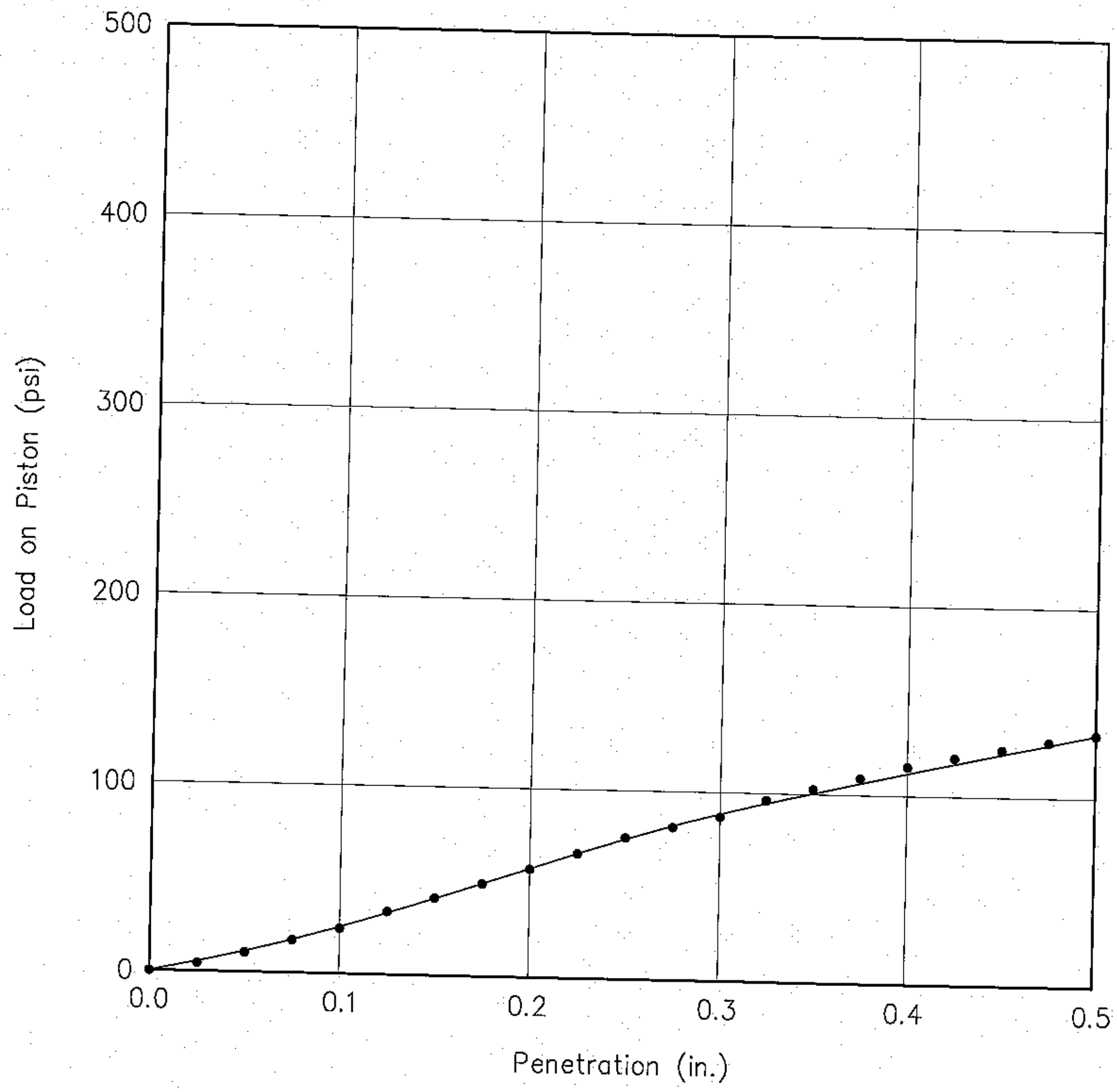
Test Results

CBR Value:	2.2%
Expansion:	0.4%

Note: Test performed at insitu moisture content of soil sample.

W.O. 10-4890	Umauma Stream Bridge Rehabilitation, North Hilo
Hirata & Associates, Inc.	CBR STRESS PENETRATION CURVE

Plate B5.1



Soil Data

Location:	Boring B2 at 2 to 4 ft
Description:	Brown clayey silt
Sample Dry Density:	74 pcf
Sample Moisture Content:	43%

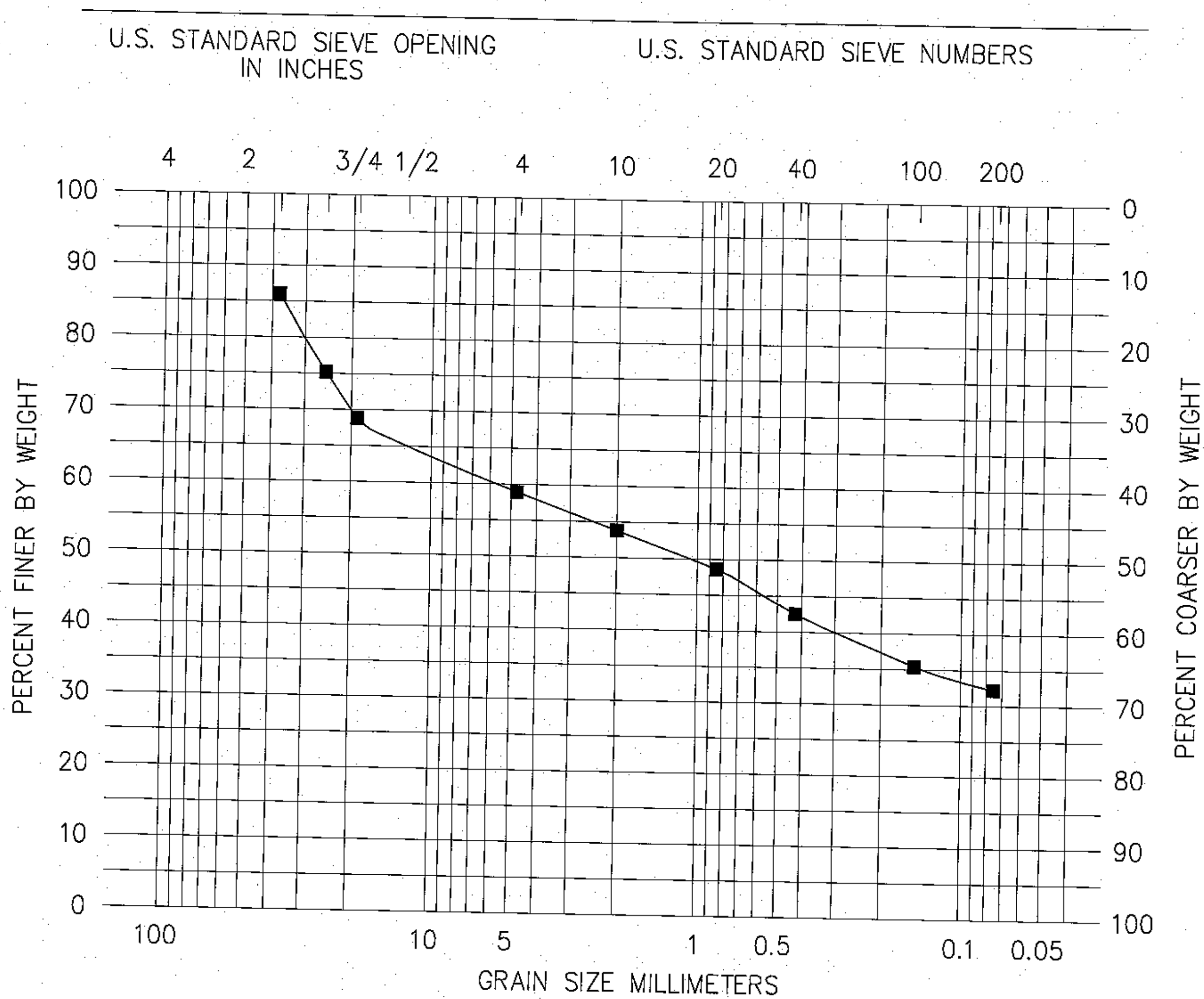
Test Results

CBR Value:	2.4%
Expansion:	1.4%

Note: Test performed at insitu moisture content of soil sample.

W.O. 10-4890	Umauma Stream Bridge Rehabilitation, North Hilo
Hirata & Associates, Inc.	CBR STRESS PENETRATION CURVE

Plate B5.2



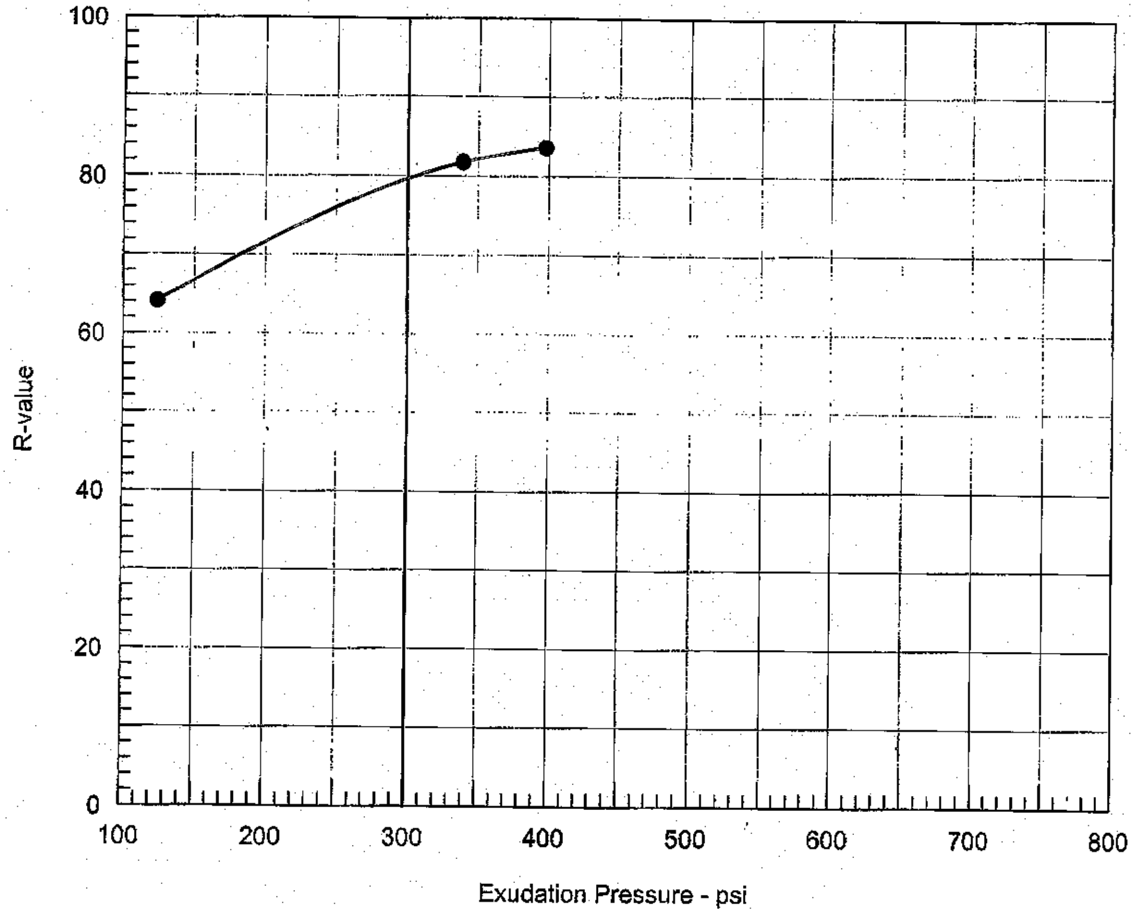
<table>
<tr><td rowspan="2">COBBLES</td><td colspan="2">GRAVEL</td><td colspan="3">SAND</td><td rowspan="2">SILT or CLAY</td></tr>
<tr><td>Coarse</td><td>Fine</td><td>Coarse</td><td>Medium</td><td>Fine</td></tr>
</table>

	Location	Description
■ Sample #2	Boring B4 at 8 ft	Brown clayey silt with sand and gravel

W.O. 10-4890	Umauma Stream Bridge Rehabilitation, North Hilo
Hirata & Associates, Inc.	GRADATION CURVES

Plate B6.1

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - ASTM D 2844

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psi	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	250	117.9	17.2	0.00	42	2.50	124	64	64
2	350	118.2	16.4	0.00	17	2.44	398	84	84
3	325	118.1	16.7	0.00	20	2.48	339	82	82

Test Results	Material Description
R-value at 300 psi exudation pressure = 80	Brown gravelly sandy silt, B1, sample received 4/22/2010

Project No.: 0020078

Project:

Location: Umauma Stream Bridge Rehab, WO #10-4890

Sample Number: 2110-1 (SL397) **Depth:** 2'-4'

Date: 4/28/2010

Tested by: DTN

Checked by: LKL

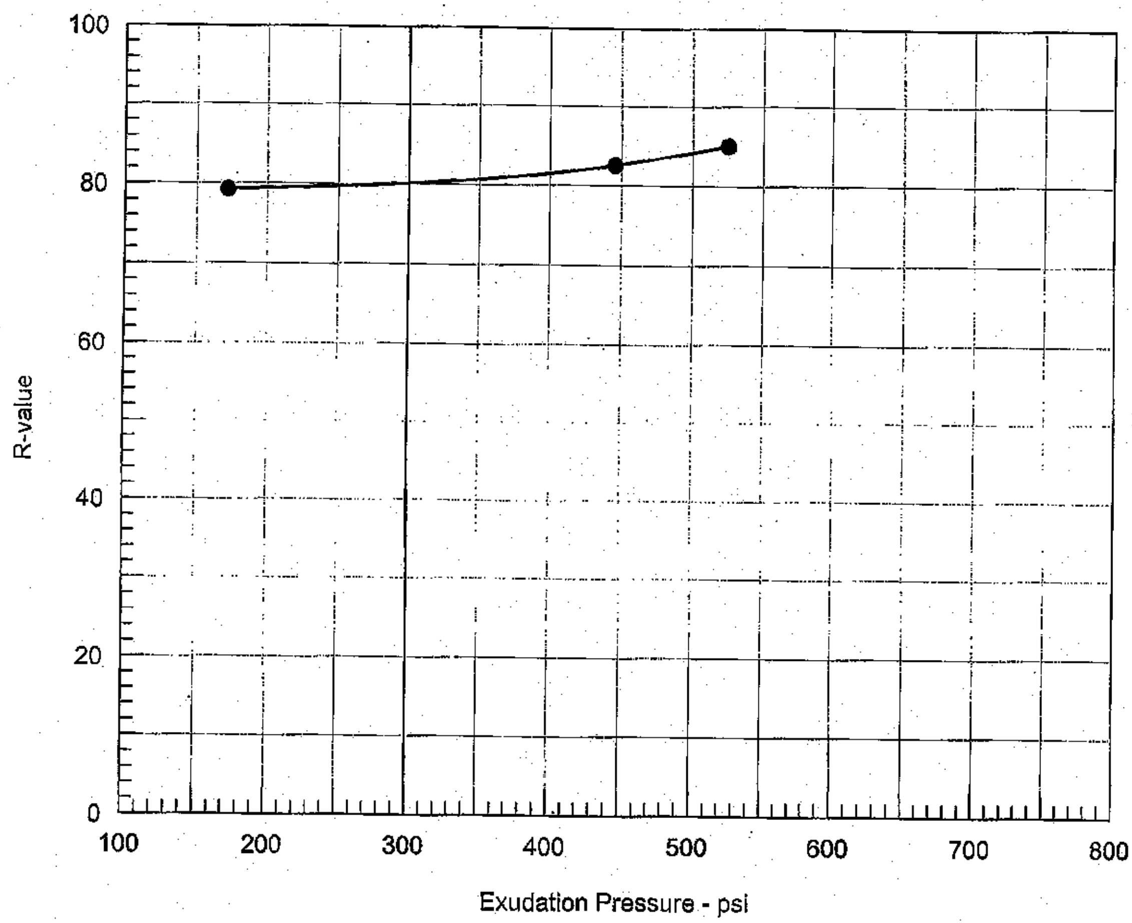
Remarks:
B1

R-VALUE TEST REPORT

SIGNET TESTING LABS, INC.

Figure B7.1

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - ASTM D 2844

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psi	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	350	113.0	18.0	0.21	17	2.44	525	86	85
2	350	113.5	18.9	0.00	24	2.46	172	79	79
3	350	113.9	18.4	0.00	19	2.42	446	83	83

Test Results	Material Description
R-value at 300 psi exudation pressure = 80	Reddish brown gravelly sandy silt, B2, sample received 4/22/2010

Project No.: 0020078

Project:

Location: Umauma Stream Bridge Rehab, WO #10-4890

Sample Number: 2110-2 (SL397) **Depth:** 2'-4'

Date: 4/28/2010

Tested by: DTN

Checked by: LKL

Remarks:
B2

R-VALUE TEST REPORT

SIGNET TESTING LABS, INC.

Figure B7.2

Construction Engineering Labs

Hirata & Associates, Inc.
99-1433 Koaha Pl.
Aiea, Hawaii 96701

Date: 11/24/10
Report: 23508

TEST REPORT

Project: Umauma Stream Bridge Rehab (Job #10-4890)	W.O. No. 23508
Client: Hirata & Associates	Received: 11/19/10
Description of material: Rock Cores	Tech: HL
Source: See Below	Sample #: 23508

Core Identification	Test Method	Compressive Strength (psi)
B1 at 39'-42'	ASTM D 2938	13024
B2 at 48'-50'	ASTM D 2938	11332
B2 at 50'-52'	ASTM D 2938	9832
B3 at 5'-10'	ASTM D 2938	5741
B3 at 10'-15'	ASTM D 2938	18625
B4 at 12'-17'	ASTM D 2938	10258
B5 at 13'-18'	ASTM D 2938	6940

Please contact our office if you have any questions or need more information.

Respectfully,
CONSTRUCTION ENGINEERING LABS, INC.

By: Ronald A. Pickering II
Its: President

APPENDIX C

LATERAL LOAD ANALYSIS

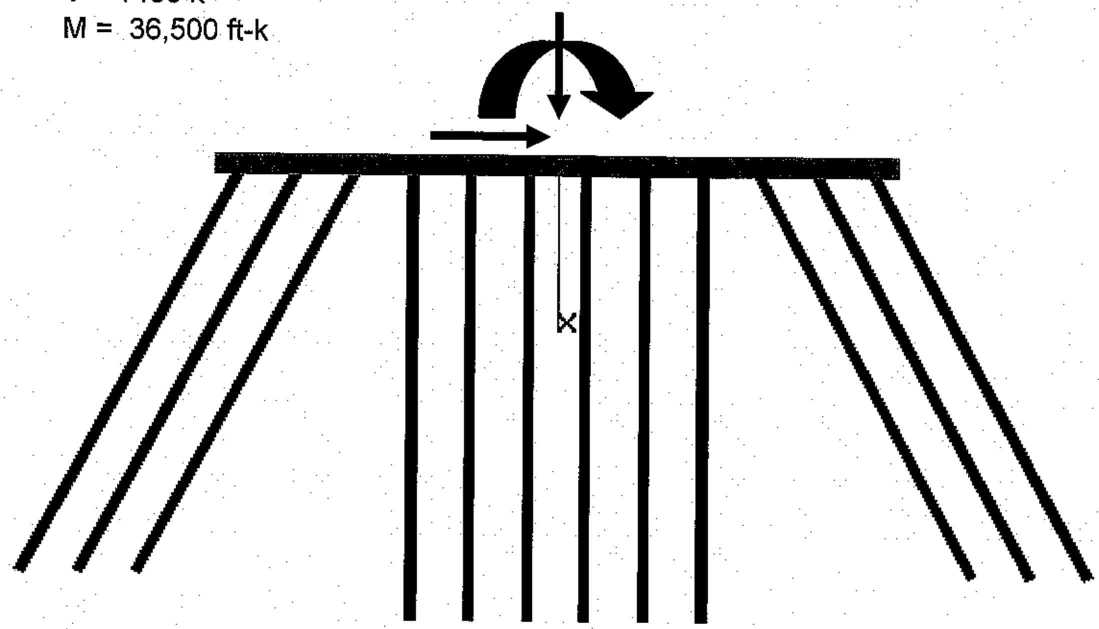
Lateral Resistance of 5-Ft Diameter Drilled Shafts At Abutment #1			
Deflection at top	0.5 in	1 in.	1.5 in
Longitudinal Direction - Free head condition (Into slope direction)	55 Kips	95 kips	135 kips
Longitudinal Direction - Free head condition (Into slope direction, ignore potential effects from adjacent abutment walls and footings)	95 kips	145 kips	190 kips
Longitudinal Direction - Free head condition (Away from slope direction)	40 kips	75 kips	115 kips
Transverse Direction - Fixed head condition	195 kips	345 kips	485 kips

Lateral Resistance of 5-Ft Diameter Drilled Shafts At Abutment #2			
Deflection at top	0.5 in	1 in.	1.5 in
Longitudinal Direction - Free head condition (Into slope direction)	70 kips	105 kips	135 kips
Longitudinal Direction - Free head condition (Into slope direction, ignoring potential effects from adjacent abutment walls and footings)	100 kips	145 kips	175 kips
Longitudinal Direction - Free head condition (Away from slope direction)	45 kips	65 kips	85 kips
Transverse Direction - Fixed head condition	145 kips	220 kips	295 kips

Umauma Stream Bridge, Pier 3 Micropile Group

W.O. 10-4890
4/27/2011

Applied Load at Pile cap in transverse direction
P = 2300 k
V = 1400 k
M = 36,500 ft-k



Note: each row has 4 (7-in diameter) micropiles

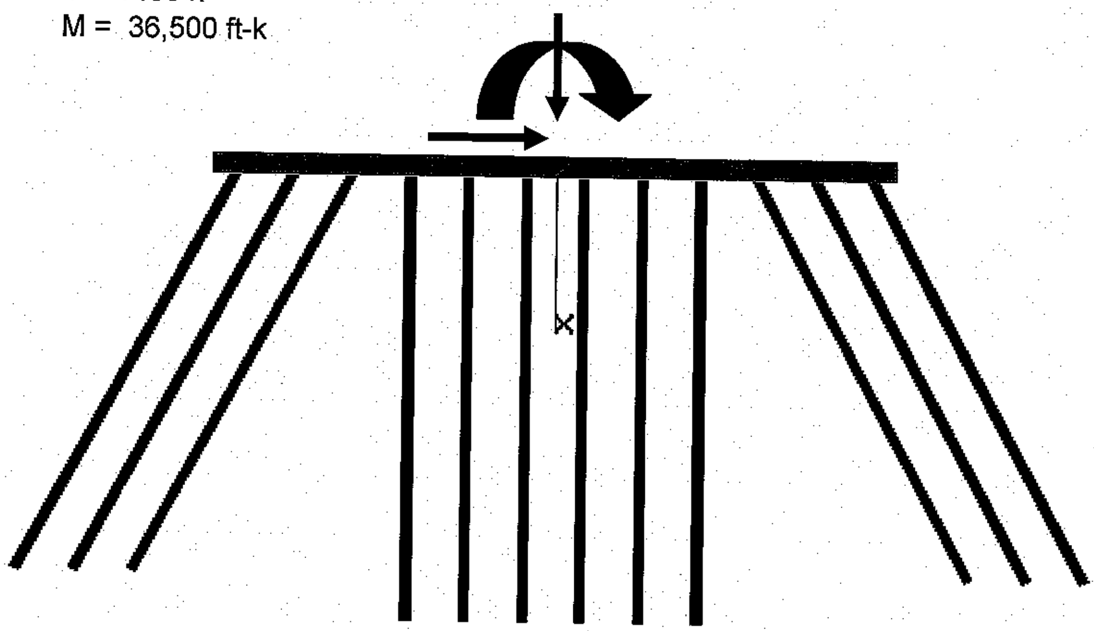
Row No.	Verical Load (kips)	Lateral Load (kips)	Axial Load (kips)	Shear (kips)	Bending Moment (ft-kips)
1	157	80.9	176.6	2.2	3.37
2	144.7	75	163	2.4	3.83
3	132.5	69.1	149.4	2.5	4.28
4	91.1	4	91.1	4	8.04
5	75.9	4	75.9	4	8.04
6	60.7	4	60.7	4	8.05
7	45.5	4	45.5	4	8.05
8	30.3	4	30.3	4	8.05
9	15.2	4	15.2	4	7.97
10	-47	27.8	-54.5	3.8	7.28
11	-59.3	33.7	-68.1	3.7	6.86
12	-71.5	39.7	-81.7	3.5	6.44

Pile Cap Deflection = 0.06 inch

Umauma Stream Bridge, Pier 3 Micropile Group

W.O. 10-4890
4/27/2011

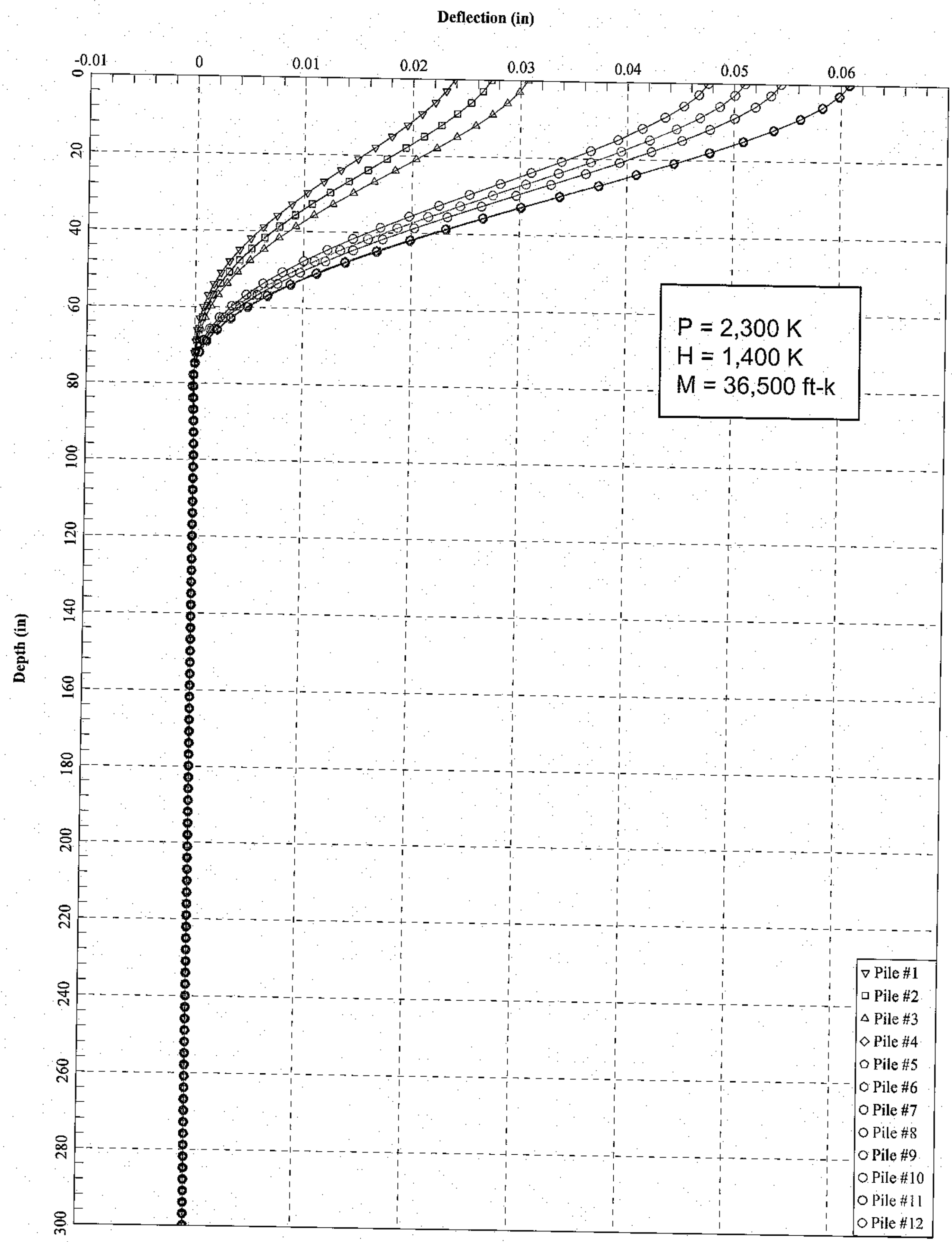
Applied Load at Pile cap in transverse direction
P = 1500 k
V = 1400 k
M = 36,500 ft-k



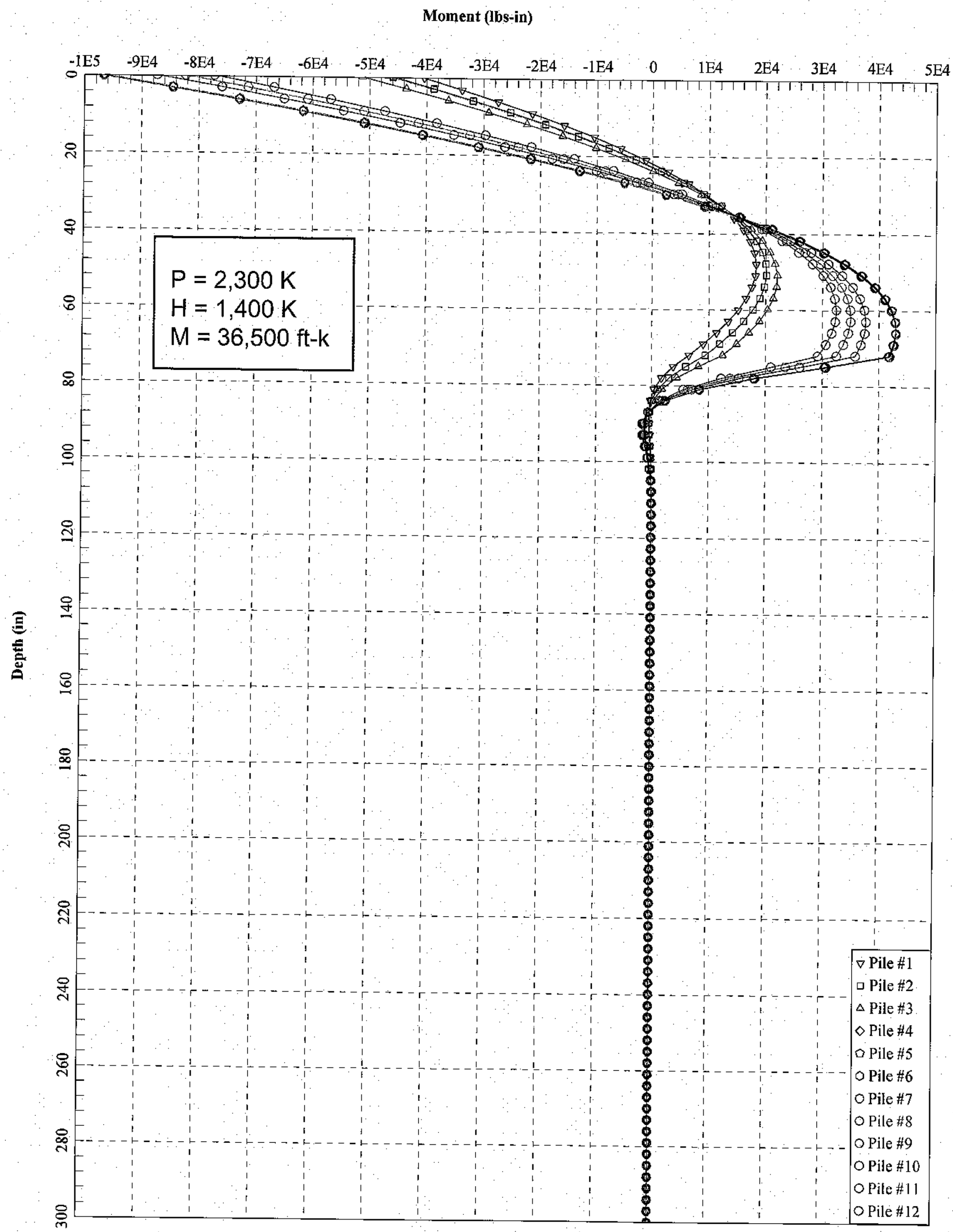
Note: each row has 4 (7-in diameter) micropiles

Row No.	Vertical Load (kips)	Lateral Load (kips)	Axial Load (kips)	Shear (kips)	Bending Moment (ft-kips)
1	142.1	73.7	160.0	2.4	3.90
2	129.8	67.8	146.4	2.6	4.35
3	117.6	61.8	132.8	2.7	4.79
4	72.7	4	72.7	4	8.02
5	57.5	4	57.5	4	8.02
6	42.3	4	42.3	4	8.02
7	27	4	27	4	8.02
8	11.8	4	11.8	4	8.02
9	-3.3	4	-3.3	4	8.03
10	-61.9	35	-71	3.6	6.74
11	-74.1	40.9	-84.6	3.5	6.32
12	-86.4	46.9	-98.2	3.3	5.89

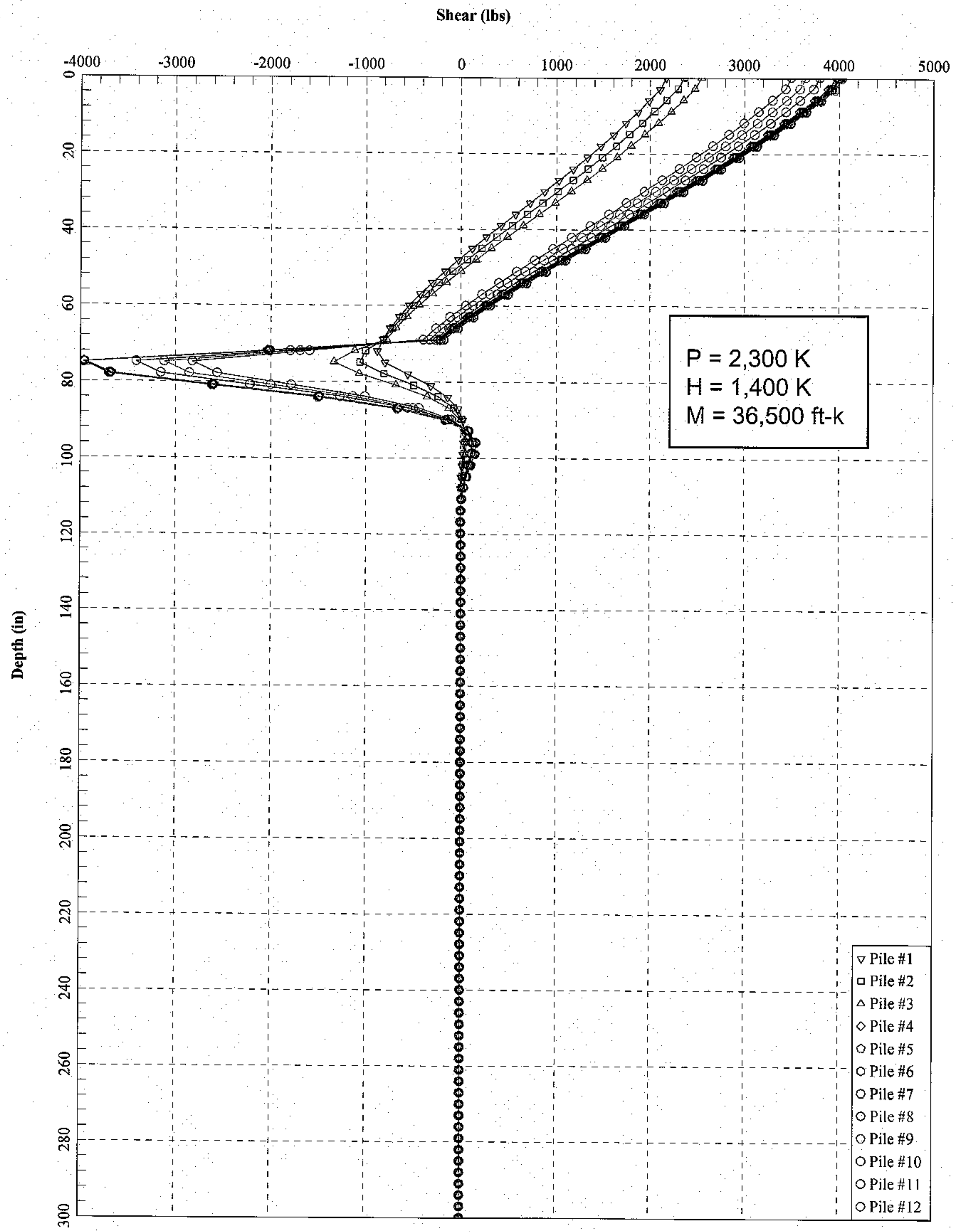
Pile Cap Deflection = 0.06 inch



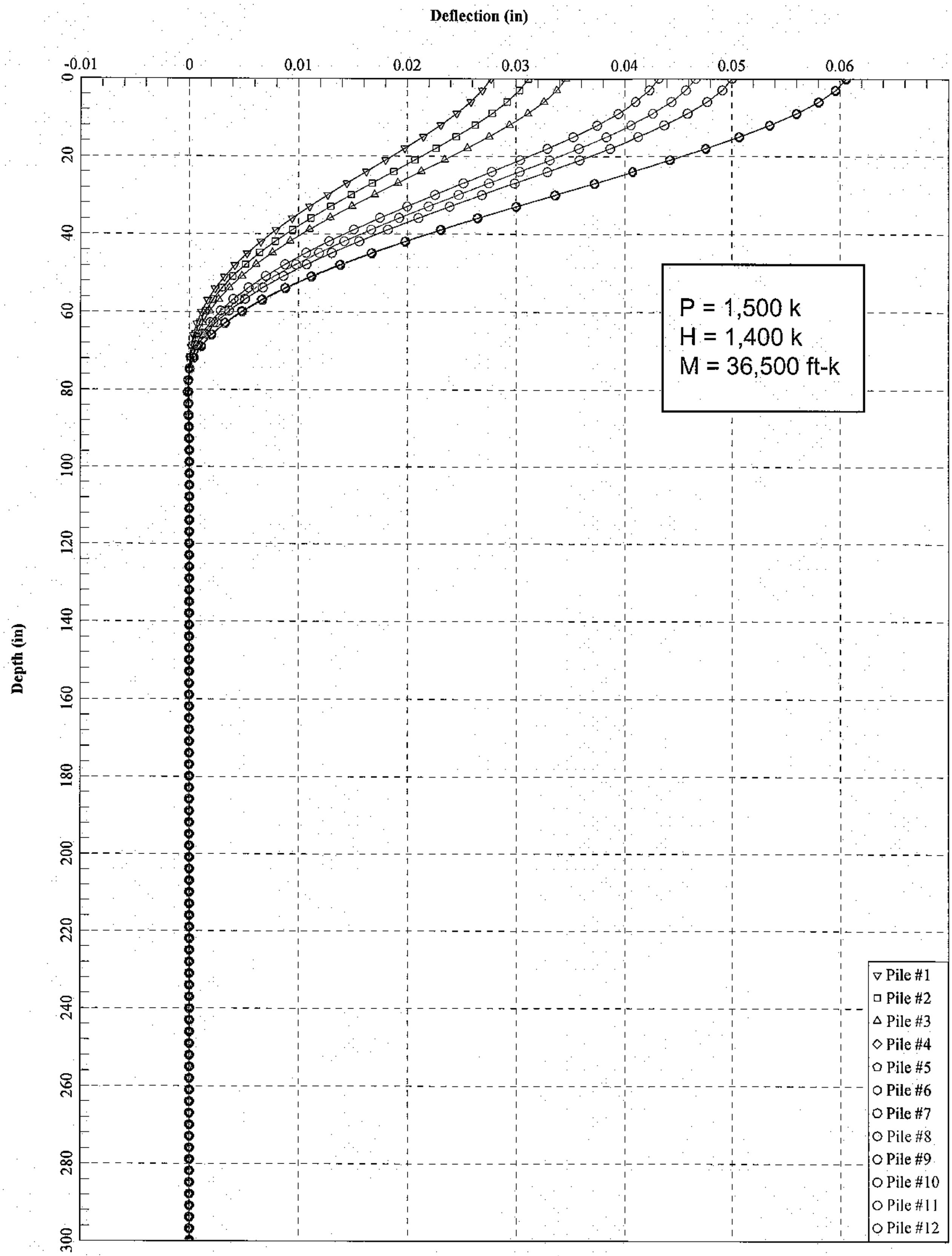
W.O. 4890 Umauma Stream Bridge Pier 3, 7-inch diameter micropiles transverse 4/20/11



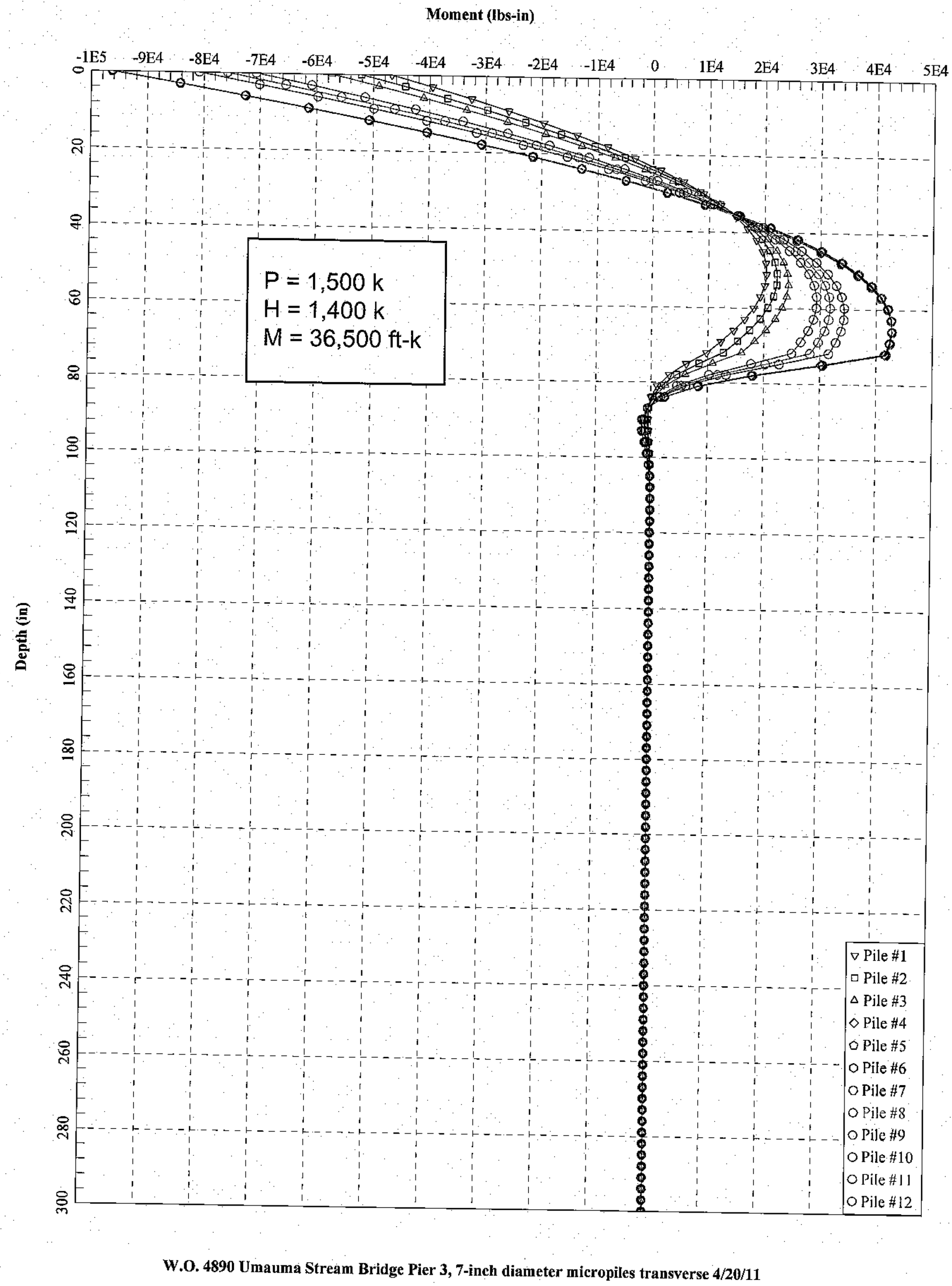
W.O. 4890 Umauma Stream Bridge Pier 3, 7-inch diameter micropiles transverse 4/20/11



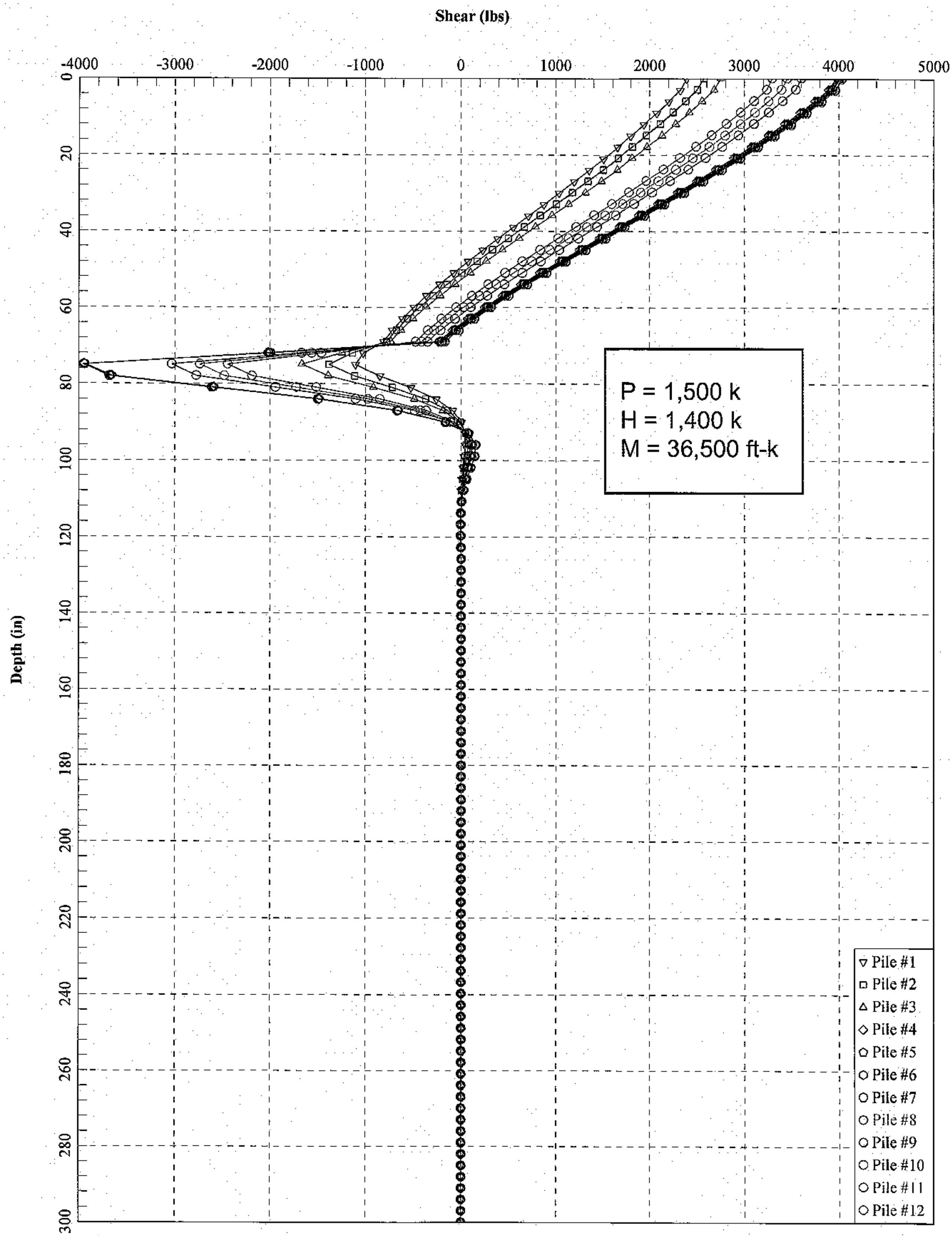
W.O. 4890 Umauma Stream Bridge Pier 3, 7-inch diameter micropiles transverse 4/20/11



W.O. 4890 Umauma Stream Bridge Pier 3, 7-inch diameter micropiles transverse 4/20/11



W.O. 4890 Umauma Stream Bridge Pier 3, 7-inch diameter micropiles transverse 4/20/11



W.O. 4890 Umauma Stream Bridge Pier 3, 7-inch diameter micropiles transverse 4/20/11

APPENDIX D

SITE CLASS CLASSIFICATION

AND

DESIGN RESPONSE SPECTRUM

Applied Geosciences, LLC

• 2922 Kahaloa Drive, Honolulu, HI 96822 • Phone: (808) 221-0104 • ags@pixi.com

February 8, 2011

Con Truong, P.E.
Ernest K. Hirata & Associates, Inc.
99-1433 Koaha Place
Aiea, HI 96701-3279

Project No. SRSS00210

Re: Design Response Spectrum, Umauma Stream Bridge

Dear Con:

Attached find the design response spectrum for the Umauma Stream Bridge Rehabilitation project.

Approach

The spectrum was developed in accordance with the *AASHTO LRFD Bridge Design Specifications, 2010, 5th Edition*. It represents the conditions to be expected at the location of the project with a 7% probability of excedance in 75 years (5% of critical damping). This represents a return period of approximately 1,000 years. A review of borings B1, B2 and related subsurface geophysical measurements taken nearby indicates interpreted average shear wave velocities in the upper 100 feet of about 1,000 ft/s for boring B1 and about 1,700 ft/s for boring B2. This suggests a site class D for boring B1 and site class C for boring B2. A uniform conservative site class C was assumed to develop the design spectrum. The computed spectral acceleration values are shown in tabular and graphical form in the Figure 1.

Discussion

The AASHTO code procedure takes the site-specific soil conditions into account in a simple manner, but it does so based on experience gained primarily in the continental U.S. It is not entirely clear how basaltic rock and weathered volcanic soils may affect ground motions. The calculated spectral values are therefore correspondingly conservative. On the other hand, the ASHTO method assumes a level ground surface and makes no allowance for topographic effects. Given the steep nature of the Umaumu gulch, this is potentially a significant factor. In general, amplification of motions occurs as a result of topographic highs (bridge abutments), whereas de-amplification occurs in concave shapes (gulch bottom). This is only a general rule of thumb and more elaborate numerical site response analyses would have to be conducted to evaluate surface ground motions along the entire alignment of the bridge.

If you have any questions, do not hesitate to contact me.

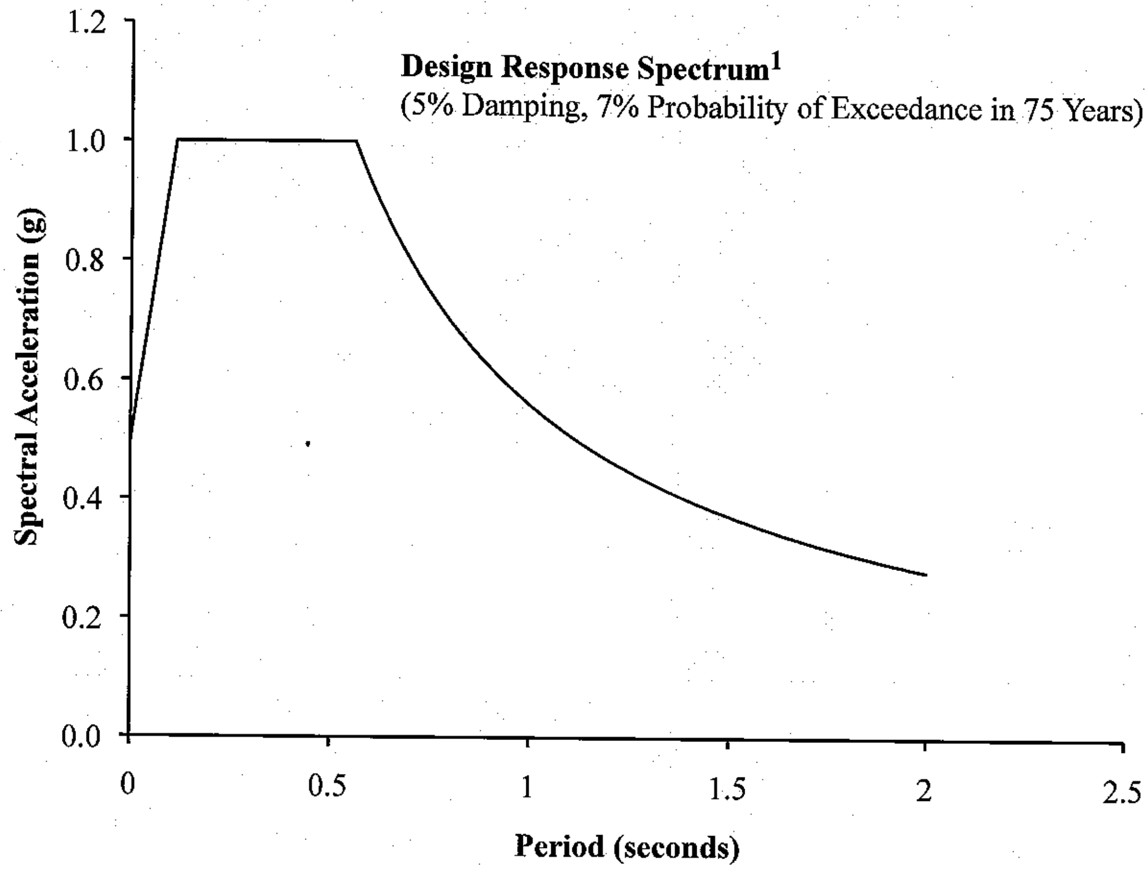
Sincerely,

Horst Brandes

Horst G. Brandes, Ph.D., P.E.
President

Att: Figure 1 (Design Response Spectrum)

HORST G. BRANDES
LICENSED PROFESSIONAL ENGINEER
No. 9336
HAWAII, U.S.A.



[1]AASHTO LRFD Bridge Design Specifications, 2010

Period (sec)	Spectral Acceleration (g)
0.00	0.50
0.01	0.54
0.02	0.59
0.03	0.63
0.04	0.68
0.05	0.72
0.06	0.77
0.07	0.81
0.08	0.86
0.09	0.90
0.10	0.95
0.11	0.99
0.11	1.00
0.12	1.00
0.13	1.00
0.14	1.00
0.15	1.00
0.16	1.00
0.17	1.00
0.18	1.00
0.19	1.00
0.20	1.00
0.25	1.00
0.30	1.00
0.35	1.00

Period (sec)	Spectral Acceleration (g)
0.40	1.00
0.45	1.00
0.50	1.00
0.56	1.00
0.58	0.97
0.60	0.93
0.62	0.90
0.64	0.88
0.66	0.85
0.68	0.82
0.70	0.80
0.72	0.78
0.74	0.76
0.76	0.74
0.78	0.72
0.80	0.70
0.82	0.68
0.84	0.67
0.86	0.65
0.88	0.64
0.90	0.62
0.92	0.61
0.94	0.60
0.96	0.58
0.98	0.57

Period (sec)	Spectral Acceleration (g)
1.00	0.56
1.02	0.55
1.04	0.54
1.06	0.53
1.08	0.52
1.10	0.51
1.12	0.50
1.14	0.49
1.16	0.48
1.18	0.47
1.20	0.47
1.22	0.46
1.24	0.45
1.26	0.44
1.28	0.44
1.30	0.43
1.32	0.42
1.34	0.42
1.36	0.41
1.38	0.41
1.40	0.40
1.42	0.39
1.44	0.39
1.46	0.38
1.48	0.38

Period (sec)	Spectral Acceleration (g)
1.50	0.37
1.52	0.37
1.54	0.36
1.56	0.36
1.58	0.35
1.60	0.35
1.62	0.35
1.64	0.34
1.66	0.34
1.68	0.33
1.70	0.33
1.72	0.33
1.74	0.32
1.76	0.32
1.78	0.31
1.80	0.31
1.82	0.31
1.84	0.30
1.86	0.30
1.88	0.30
1.90	0.29
1.92	0.29
1.94	0.29
1.96	0.29
1.98	0.28
2.00	0.28