Appendix IS-3

Geotechnical Report & Memorandum

GEODESIGN INC

Memorandum

To:	Yuri Moshinski	From:	Chris Zadoorian
Company:	Sodo Builders, LLC	Date:	June 13, 2018
Address:	270 South Hanford Street, Suite #100 Seattle, WA 98134		

cc:	Steven Rupert, GBD Architects (via email only) Brad Napientek and Ashley Rogers, Eyestone Environmental (via email only)

GDI Project:	SodoBuild-2-01
RE:	Responses to California Environmental Quality Act Items Proposed Mixed-Use Hotel Development 100 East Ocean Boulevard, Long Beach, California

INTRODUCTION

We performed a geotechnical investigation at the site of the proposed hotel development to be constructed at 100 East Ocean Boulevard in Long Beach, California and we summarized the results in a preliminary report dated November 9, 2018 and a draft report dated January 24, 2017.

We are currently working on a final design report that includes more detailed programming information including updated building height and column loading.

Brad Napientek of Eyestone Environmental reviewed our January 24, 2017 draft report requested clarification regarding (1) the potential for lateral spreading at the site, (2) the presence of expansive soils and (3) the current planned building height, 402 feet, which is approximately 100 feet taller than indicated in our draft report dated January 24, 2017.

Each item is addressed below.

<u>Item 1 – Potential for Lateral Spreading at the Site</u>

Lateral spreading is a secondary seismic hazard that may occur where the ground surface level is sloping and potentially liquefiable soils are present within the slope.

The ground surface level at the site does slope gently to the south with the project limits, and a grade change is present immediately north of the proposed building site, however, liquefiable soils are not present at the site and therefore, the potential for lateral spreading is not present at the site.

Item 2 – Expansive Soils

Fill soils ranging from approximately 4.0 to 14.5 feet in thickness were encountered in the borings. The fill soils generally consist of medium dense to very dense silty sand and hard sandy silt and contained various amounts of asphalt and brick fragments.

Native soil encountered beneath the fill generally consists of medium dense to very dense sand, sand with silt, and silty sand with intermittent layers of very stiff to hard silt and sandy silt approximately 2.5 to 8.5 feet in thickness.

None of the soils encountered exhibited the potential for expansion and therefore, expansive soils are not present at the site and as such mitigation measures are not required to address expansive soils.

Item 3 – Current Planned Building Height

The current planned building height is approximately 100 feet taller than the previously planned building height as described in our draft report dated January 24, 2017. The current planned development will result in an increased building weight when compared to the earlier concept and we estimated the increase to be on the order of 33 percent and based on the preliminary loading information provided by DCI, the project structural engineer, we estimate the current planned dead-plus-live foundation loading for the tower to be on the order of 8,500 psf.

While the proposed increased building height does not change the conclusions nor recommendations presented in our draft report, total settlement of the proposed tower will increase from a previously estimated 1½ inch or less to 2.0 inches or less. The increased settlement is tolerable for the planned foundation system (mat foundation) and remains compatible with the adjacent podium structure settlement of the proposed podium structure.

♦ ♦ ♦

We appreciate the opportunity to be of continued service to you. Please call if you have questions concerning this report or if we can provide additional services.

CJZ

Attachments

One copy submitted (via email only)

Document ID: SodoBuild-2-01-061318-geom-cjz

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GEODESIGN INC

REPORT OF GEOTECHNICAL ENGINEERING SERVICES

Proposed Hotel Development
100 East Ocean Boulevard
Long Beach, California

For
Sodo Builders, LLC
January 24, 2017

GeoDesign Project: SodoBuild-2-01

GEODESIGN INC

January 24, 2017

Sodo Builders, LLC
270 South Hanford Street, #100
Seattle, WA 98134

Attention: Edward Kirk and Yuri Moshinski

Report of Geotechnical Engineering Services
Proposed Hotel Development
100 East Ocean Boulevard
Long Beach, California
GeoDesign Project: SodoBuild-2-01

GeoDesign, Inc. is pleased to submit this geotechnical engineering report for the proposed hotel development to be constructed at 100 East Ocean Boulevard in Long Beach, California. Our services were performed in general accordance with our proposal dated June 16, 2016.

We appreciate the opportunity to be of service to you. Please contact us if you have questions regarding this report.

Sincerely,

GeoDesign, Inc.

Christopher J. Zadoorian, G.E.
Principal Engineer

cc: Katherine Schultz, GBD Architects (one copy)
Paul Rogness, DCI Consulting Engineers (one copy)
Jose Hernandez, KPFF Consulting Engineers (one copy)

ENT:CJZ
Attachments
One copy submitted (via email only)
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ACRONYMS AND ABBREVIATIONS

1.0 INTRODUCTION

This report summarizes our geotechnical investigation for the proposed hotel development to be constructed at 100 East Ocean Boulevard in Long Beach, California as shown on Figure 1.

The site located at the southeast corner of Pine Avenue and East Ocean Boulevard and is bound on the south by East Seaside Way, on the east an existing tower at 180 East Ocean Boulevard, and to the north by a public park. An existing tunnel structure, the Jergins Tunnel, is located north of the site that crosses beneath East Ocean Boulevard and terminates in the Renaissance Hotel located the northeast corner of Pine Avenue and East Ocean Boulevard as shown on Figure 2.

The ground surface level at the site generally slopes down from north-to-south and ranges from approximately Elevation 19 at the northwest corner of the site to approximately Elevation 6 at the south end of the site.

Plans for the tower located at 180 East Ocean Boulevard were not available at the time we prepared this report, however, based on our observations in the field, the lowest finished floor level of the existing tower is established at approximately Elevation 6. It will ultimately be necessary to obtain the foundation plans for the tower in order to design temporary shoring and permanent walls below grade.

An existing tunnel structure, the Jergins Tunnel, is located north of the site that crosses beneath East Ocean Boulevard and terminates at the Renaissance Hotel located at the northeast corner of Pine Avenue and East Ocean Boulevard.

The ground surface level at the site generally slopes down from north-to-south and ranges from approximately Elevation 16 at the northwest corner of the site to approximately Elevation 6 at the southeast corner.

Paul Rogness of DCI Engineers furnished us with a structural narrative and preliminary foundation plans each dated November 23, 2016 that depict the proposed development. Jeremy Ryan of DCI Engineers furnished us with preliminary foundation loading information and preliminary foundation deformation analysis for our review.

Based on our review of the narrative and foundation plans and our discussions with Mr. Rogness, the proposed development will include the construction of an approximately 310-foot-tall, 26-story hotel building. The proposed development will include one full subterranean parking level with a finished floor elevation of approximately Elevation -3.

The planned foundation system will consist of a 5- to 7.5-foot-thick mat foundation beneath the core of the proposed tower, and spread and continuous footings to support the adjoining podium structures as shown on Figure 2. We anticipate the bottom of the foundations will be established between Elevations -6.0 to -12.5, approximately.

Based on the loading information provided by Mr. Ryan total dead-plus-live load applied pressure mat foundation would range to approximately 6,500 psf and dead-plus live column loads for the podium will be on the order of 1,200 kips to 1,650 kips.

Based on our review of the *Seismic Hazard Evaluation of the Long Beach 7.5-Minute Quadrangle, Los Angeles County, California* (California Geologic Survey [CGS], March 25, 1999), the southern approximately one-half of the site is located within a liquefaction hazard zone as shown on Figure 1.

Our investigation is summarized below followed by our conclusions and recommendations for the design and construction of the proposed development.

Please note that supplemental analysis will be required to develop design recommendations for temporary shoring and permanent walls below grade on the east building side once sufficient information is available for the adjacent tower at 180 East Ocean Boulevard.

Acronyms used herein are defined at the end of this document.

2.0 PURPOSE AND SCOPE

The purpose of our investigation was to determine the subsurface conditions at the site and develop geotechnical design recommendations for the proposed development. Our specific scope of services included the following primary tasks:

- Drilling 5 borings at the site using mud rotary drilling equipment
- Advancing 4 Cone Penetrometer Test (CPT) soundings
- Collecting representative samples from the borings and maintain a log of the soil conditions encountered at the site
- Performing P-S suspension logging to develop a shear-wave velocity profile
- Performing geotechnical laboratory testing on samples collected from the borings
- Evaluating liquefaction potential at the site
- Developing seismic design parameters in accordance with the 2016 CBC
- Developing foundation recommendations for the proposed development
- Developing recommendations for temporary shoring
- Developing recommendations for below-grade building walls
- Developing recommendations for floor slab support
- Developing recommendations for general site flatwork
- Developing recommendations for general site grading and earthwork
- Preparing this report summarizing our investigation and presenting our conclusions and recommendations

3.0 SUBSURFACE INVESTIGATION

3.1 SUBSURFACE EXPLORATIONS

3.1.1 Borings and CPTs

We explored the subsurface conditions at the site by drilling 5 borings (B-1 through B-5) and advancing four CPTS at the locations shown on Figure 2. The borings were drilled to depths ranging from 50.9 to 125.4 feet BGS using mud-rotary drilling equipment and the CPTs were advanced to depths ranging from 6.7 to 50.1 feet BGS. Refusal was encountered at a depth of 6.7 feet in CPT-2 after three attempts.

We maintained a log of the soil conditions encountered in each boring and collected relatively undisturbed and disturbed samples at regular depth intervals.

The logs of the borings, and a detailed description of our drilling and sampling are presented in Appendix A and logs of the CPTs are presented in Appendix B.

3.1.2 Geophysical Testing

Upon completion of drilling in boring B-5, P-S suspension logging was performed by GEOVision, Inc. to estimate the stiffness of the subsurface soil profile of the upper 125 feet.

The suspension logging method uses a 7-meter probe that contains a source and two receivers. The probe is lowered down the drilled hole where the source generates a pressure wave in the drilling fluid within the hole. The pressure wave is converted to seismic P- and S-waves at the boring sidewalls; at each receiver, the P- and S-waves are converted back to pressure waves. The elapsed time between wave arrivals at the receivers is used to determine the average velocity of a 1-meter-high column of soil. The process is repeated for the full depth of the boring to obtain a continuous log of the boring.

Based on the results of shear wave velocity measurements, the average shear wave velocity for the upper 100 feet was approximately 1,070 feet per second (330 meters per second).

The results of the P-S logging are presented in Appendix C.

3.2 SUBSURFACE CONDITIONS

AC pavement ranging from 1 to 3 inches in thickness was encountered at the surface of each boring.

Fill soils ranging from approximately 4.0 to 14.5 feet in thickness were encountered in the borings. The fill soils generally consist of medium dense to very dense silty sand and hard sandy silt and contained various amounts of asphalt and brick fragments. It's likely that existing debris and/or remnants of prior development at the site are the cause of refusal within CPT-2.

Native soil encountered beneath the fill generally consists of medium dense to very dense sand, sand with silt, and silty sand with intermittent layers of very stiff to hard silt and sandy silt approximately 2.5 to 8.5 feet in thickness.

Generalized depictions of the subsurface conditions are presented in Figures 3 to 5.

3.3 GROUNDWATER CONDITIONS

Groundwater was encountered in our explorations at depths ranging from approximately 7.0 to 12.5 feet BGS, corresponding to approximately Elevations -1 to -2.5.

You furnished us with a Phase I Environmental Assessment dated December 2010 prepared by SCS Engineers that included installation of two groundwater monitoring wells at the site in 2004. Based on the well data at the time of installation, groundwater was encountered at approximately 8 feet BGS in MW-1 and at approximately 11 feet BGS in MW-2, corresponding to approximately Elevation -0 to -2.0.

Based on our review of the *Seismic Hazard Evaluation of the Long Beach 7.5-Minute Quadrangle, Los Angeles County, California* (CGS, March 25, 1999), the historical high groundwater level at the site is less than 10 feet BGS which is consistent with the data from our explorations.

For the purposes of this investigation, we assumed a design groundwater level Elevation 2.0.

4.0 GEOTECHNICAL LABORATORY TESTING

We performed geotechnical laboratory testing on samples collected from our investigations to determine strength, consolidation and other pertinent characteristics of the soil. The following tests were performed:

- In-place moisture and dry density
- Atterberg limits
- Grain-size distribution
- Direct shear
- Consolidation

The results of the testing are presented in Appendix A.

5.0 GEOLOGIC AND SEISMIC HAZARDS

Primary geologic and seismic hazards that may impact the development project include liquefaction potential and surface fault rupture. Each is addressed briefly in the following sections.

5.1 LIQUEFACTION POTENTIAL

Based on our review of the Seismic Hazard Zones map of the Long Beach 7.5 minute Quadrangle dated March 25, 1999 by the California Geological Survey (CGS), the southern approximately one-half of the site is located within a liquefaction hazard zone.

Liquefaction generally occurs in saturated, loose to medium dense, granular soil and in saturated, soft to moderately firm silt as a result of strong ground shaking. As the density and/or particle size of the soil increases and as the confinement (overburden pressure)

increases, the potential for liquefaction decreases. Typically, saturated soil within the upper 50 feet of the ground surface or lowest adjacent grade is considered subject to liquefaction.

Our borings included relatively closely spaced sampling intervals for the purpose of performing a detailed liquefaction analysis.

We utilized the procedure outlined in the National Center for Earthquake Engineering Research document titled *Proceedings of the NCEER Workshop of Liquefaction Resistance of Soils* (Youd and Idriss, 1997, updated in 2001).

To evaluate the liquefaction potential of fine-grained soils, we utilized the procedures summarized and/or suggested by Boulanger and Idriss (2006), which include references to the work by Andrus and Martin (2000), Seed et al. (2003), and Bray et al. (2004). In essence, these procedures evaluate whether soils will behave more like clay or more like sand. Clay-like behavior generally precludes liquefaction while sand-like behavior indicates soils may be subject to liquefaction and should be evaluated using the appropriate procedure.

Our determinations for clay- and sand-like behavior were made based on the plasticity data, moisture content, and grain-size distribution data from our laboratory testing.

The groundwater level at the time of our field investigation was generally consistent with the historical high groundwater level in the area; therefore, the current groundwater level data was used for each boring.

The primary seismic input data for a liquefaction analyses includes the pre-dominant earthquake magnitude and peak ground acceleration (PGA). We determined a predominant earthquake magnitude of 6.86 and PGA_M of 0.628 g using the USGS 2008 interactive deaggregation web tool, and the USGS web-based seismic design maps.

The results of our analyses indicate that soils below the planned foundation levels are sufficiently dense and stiff to preclude liquefaction.

5.2 SEISMIC (DRY) SETTLEMENT

Seismic-induced (aka "dry") settlement is generally agreed to occur in loose, clean sand above the water table as the result of strong ground shaking.

The granular soils encountered at the site are sufficient dense to preclude the occurrence of seismic-induced dry settlement.

5.3 SURFACE FAULT RUPTURE

Faults in Southern California are considered active, potentially active, and inactive based on criteria developed by CGS for the Alquist-Priolo Earthquake Fault Zoning Program (Hart, 1999). By definition, an active fault is one that has had surface displacement within Holocene time (approximately the last 11,000 years). A potentially active fault is one that has demonstrated surface displacement of Quaternary age deposits (last 1.6 million years). Inactive faults have not moved in the last 1.6 million years. The primary purpose of the Alquist-Priolo Earthquake Fault

Zoning Program is to identify sites that have a potential for surface rupture due to active faults that are in close proximity to the site. In such cases, a building setback zone is established to mitigate the potential for surface rupture.

The site is not located within a designated fault-study zone and the closest such zone is approximately 2 miles northeast of the site. The potential for ground surface fault rupture at the site is considered to be very low.

6.0 CONCLUSIONS

6.1 GENERAL

The site is free from geologic or seismic hazards that would preclude the proposed development, and the proposed development is considered feasible from a geotechnical perspective.

The site is subject to strong ground shaking that would result from an earthquake occurring on a nearby or distant fault source; however, this hazard is common in Southern California and can be mitigated by implementing the provisions of the California Building Code (CBC).

6.2 FOUNDATIONS

Native soils encountered at the planned foundation levels consist of medium dense to dense silty sand and very stiff to hard sandy silt. These soils are suitable to support the proposed tower on a mat foundation and the adjacent podium on spread footings.

Fill soils are expected to depths of up to approximately 14.5 feet in thickness and may be deeper in areas not explored during our field investigation; however the existing fill will be removed as part of the planned excavation.

If fill materials are encountered below the planned foundation bottom, these materials shall be removed and replaced with lean-mix concrete below the footing as recommended in Section 7.1.

6.3 GROUNDWATER

The planned excavation will extend below the groundwater table; therefore, provisions for temporary dewatering during construction will be required.

Development of groundwater pumping rates and design details for temporary dewatering systems is beyond the scope of this investigation; however, we anticipate that such systems would require a series of well-points and/or internal trenches.

Groundwater discharged during construction will require off-site disposal and typically this is accomplished either through the storm drain system or the sanitary sewer system. In each case, a permit is required through the appropriate regulatory agency.

We can aid in the design of on-site dewatering systems and/or in obtaining necessary discharge permits, if requested.

Provisions to account for groundwater in the permanent design of the proposed below-grade structure walls and floor slab-on-grade are presented in Sections 7.3 and 7.4.

6.4 FLOOR SLABS

The current concept is to utilize a topping slab above the mat foundation and for the remainder of the building footprint to utilize a conventional floor slab-on-grade.

The topping slap will necessarily be supported on compacted fill placed over the mat foundation.

For the remainder of the building footprint, native soils are generally anticipated at the planned floor slab level, however, in boring B-5 and CPT-3 we existing fill appears to extend a few feet below the lowest finished floor level.

Since the majority of the floor slab will be supported on native soils, if existing fill is present in isolated areas it would be prudent to remove the existing fill and replace it as properly compacted fill. However, if existing fill is more than a few feet thick below the bottom of the floor slab, in which case it is less practical to remove and recompact, an alternative solution could be implemented to allow some existing fill to remain in place, as recommended in Section 7.2.

Additionally, the building floor slab will be subject to hydrostatic pressure and waterproofing of the floor slab will be required in accordance with the 2016 CBC.

6.5 SHORING, EXCAVATIONS, AND PERMANENT BELOW-GRADE WALLS

Proposed excavations on the order of 10 to 15 feet BGS will be required to achieve the planned lowest finish floor level and an additional approximately 5 to 10 feet for foundation excavations.

Temporary shoring will be required to support the proposed excavations in conjunction with temporary construction dewatering.

Recommendations for temporary excavations, temporary shoring and permanent below grade walls are presented in Sections 7.3. 7.4 and 7.5, respectively, based on the information available at this time.

Once foundation information for the adjacent tower located at 180 East Ocean Boulevard is available, revised recommendations will be developed that consider the influence of the adjacent foundations, as appropriate. If the adjacent tower is supported on spread footings, it is likely that underpinning will be will be an appropriate solution.

7.0 RECOMMENDATIONS

7.1 FOUNDATIONS

7.1.1 General

The proposed hotel tower may be supported on a mat foundation and the adjacent podium structure may be supported on spread footings established in on-site native soils.

Foundation excavation bottoms should be carefully observed and probed by our technician to confirm undocumented fill, loose, soft, or otherwise unsuitable soils are not present. If unsuitable soils are present, these soils should be removed and replaced with lean-mix concrete.

Recommendations for the tower mat foundation and spread footings are presented below.

7.1.2 Mat Foundation

We performed static settlement analysis for the proposed tower mat foundation based on the preliminary dead-plus live loading information provided by Messrs. Rogness and Ryan and the results of our analysis indicate total static settlement of 1½ inches or less for the mat foundation and differential settlement across the mat of ½ inch or less. We anticipate the majority of static settlement will occur during construction as the dead load is applied.

For foundation deformation evaluation of the mat foundation, a subgrade modulus equal to 120 pci may be used, noting that this value already has considered the effect of the size of the mat foundation. Utilizing a subgrade modulus of 120 pci, we compute dynamic deformation of the foundation soil of approximately 0.4 to 0.5 inches based on applied bearing pressures from two preliminary seismic loading cases provided to us by Mssrs. Rogness and Ryan.

Updated foundation deformation analysis should be performed as part of the design development process to verify that foundation deformation estimates are compatible with geotechnical settlement dynamic settlement estimates.

7.1.3 Spread Footings

The proposed parking podium may be supported on spread footings established in the on-site native soils. Spread footings a minimum of 2 feet wide and established at least 2 feet below the lowest adjacent grade or top of floor slab may be designed using an allowable bearing pressure of 7,000 psf.

The recommended bearing pressures are a net value and apply to the total of dead and long-term live loads and may be increased up to one-third when considering earthquake or wind loads. The weight of the footing and overlying backfill can be neglected when calculating footing loads.

We performed settlement analyses for the proposed podium footings based on the loading information provided by Mssrs. Ryan and Rogness and the results of our analysis indicated total settlement of 1 inch or less for spread footings and differential settlement of ½ inch or less between adjacent spread footings as well as between spread footings and the mat foundation.

7.1.4 Lateral Resistance

For mat and spread footings, lateral loading may be resisted by foundations using an undrained passive pressure of 225 psf per foot of embedment for footings where the concrete is placed directly against the undisturbed native soils.

A coefficient of friction equal to 0.25 may be used when calculating resistance to sliding for foundations bearing on undisturbed native soils, assuming that a waterproofing membrane is present below the bottom of the foundations and/or waste slab, if utilized.

The passive resistance and the frictional resistance may be used in combination without reduction and may also be increased by one-third when considering short-term seismic and wind loading.

7.2 FLOOR SLABS

The proposed building floor slab over the mat foundation will be a topping slab supported and may be supported on properly compacted fill.

For the remainder of the building footprint, the proposed building floor slab may be supported on native soils where present. If existing undocumented fill materials are present, these materials should be removed and replaced as properly compacted fill or, alternatively, replaced to a depth of 8 inches below the bottom of the floor slab with crushed rock placed on a non-woven geotextile fabric.

The bottom of the floor slab excavation should be carefully observed and probed by our technician to confirm undocumented fill, loose, soft, or otherwise unsuitable soils are not present and/or to provide mitigation recommendations in the field.

Satisfactory subgrade support for floor slabs supporting up to an estimated 400 psf areal loading on properly compacted fill and/or stiff or dense native alluvial soil can be obtained.

The building floor slab should be designed to resist and upward pressure resulting from the design groundwater level of Elevation 2.0.

7.3 TEMPORARY EXCAVATIONS AND VERTICAL CUTS

If necessary, temporary, unsurcharged slopes should not exceed a 1.5H:1V gradient when constructed in existing fill and/or native material. Such temporary slopes should not exceed 15 feet in height.

Temporary vertical cuts that will be beneficial for foundation construction may be made into the native material but should not exceed 3 feet in height. Deeper cuts for foundation excavations should be sloped at 1.5H:1V.

Temporary cut slopes should be protected from erosion by directing surface water away by placing sand bags at the top of the slopes and during wet weather, covering the slopes with plastic sheeting.

7.4 TEMPORARY SHORING

7.4.1 Temporary Shoring Design Lateral Earth Pressures

Typically, cantilevered shoring is feasible for retained heights of approximately 15 feet or less, and braced shoring typically becomes economical for retained heights exceeding 15 feet.

Cantilevered shoring should be designed to resist a triangular lateral earth pressure distribution as shown in Figure 6 and internally braced shoring should be designed to resist a trapezoidal lateral earth pressure distribution as shown in Figure 7.

Please note that once foundation information for the adjacent tower located at 180 East Ocean Boulevard is available, revised recommendations will be developed that consider the influence of the adjacent foundations, as appropriate.

The upper 10 feet of the below-grade building walls should be designed to resist a uniform lateral pressure of 100 psf to account for normal traffic loading, where present as shown on Figures 6 and 7.

Where the surface at the top of the shoring is sloped, the recommended lateral earth pressures should be increased as recommended in Table 1.

Table 1. Increase in Lateral Earth Pressures for Retained Slopes

Slope Inclination at Top of Wall (H:V)	Increase in Lateral Earth Pressure (percent)
1:1	200
1.5:1	165
2:1	150

In addition, when developing design drawings for temporary shoring, it would be prudent to consider the location of construction cranes and other potentially heavy equipment or loads that may act against the shoring system.

7.4.2 Soldier Piles

For the design of soldier piles spaced at least 2 diameters on centers, the allowable lateral bearing value (passive value) of the native soil below the level of excavation may be assumed to be 250 psf per foot of depth, up to a maximum of 2,500 psf, assuming that the groundwater table is drawn down to approximately the bottom of excavation.

To develop the full lateral value, provisions should be taken to ensure firm contact between the soldier piles and the undisturbed soil.

If the embedded portion of the soldier pile shaft is filled with lean-mix concrete with a minimum compressive strength of 2,000 psi, then the effective width of the soldier pile shaft for use in developing passive resistance may be assumed to be twice the diameter of the shaft. If the embedded portion of the soldier pile shaft is filled with other material (such as low strength sand-cement slurry), the effective width of the soldier pile should be limited to be the diagonal dimension of the soldier pile beam.

The frictional resistance between the soldier piles and the retained earth may be used in resisting the downward component of the tieback anchor loads. For design, the coefficient of friction

between the soldier piles and the retained earth is 0.4. This value is based on the assumption that uniform full bearing will be developed between the steel soldier beam and the shaft backfill material and the retained earth.

In addition, provided that the portion of the soldier piles below the excavated level is backfilled with structural concrete, the soldier piles below the excavated level may be used to resist downward loads. For resisting the downward loads, the frictional resistance between the concrete soldier piles and the soil below the excavated level may be taken equal to 300 psf for drilled solider piles. For soldier piles that are vibrated into the supporting soil, the frictional resistance between the soldier piles and the soils below the excavated level may be taken as 600 psf.

Where vibratory methods are utilized, the diagonal of the solider beam may be used for the width when computing allowable passive resistance. Pre-drilling, if utilized in conjunction with vibratory methods, should not extend below the bottom of the planned excavation and the diameter of the pre-drilling auger should be less than the beam diagonal.

7.4.3 Timber Lagging

Continuous lagging will be required between the soldier piles. The soldier piles and anchors should be designed for the full anticipated lateral pressure; however, the pressure on the lagging will be less due to arching in the soil. For clear spans of up to 8 feet, we recommend that the lagging be designed for a triangular distribution of earth pressure where the maximum pressure is 400 psf at the mid-line between soldier piles and 0 psf at the soldier piles.

7.4.4 Tiebacks

Tieback friction anchors may be used to resist lateral loads. For design purposes, it may be assumed that the active wedge adjacent to the shoring is defined by a plane drawn at 30 degrees with the vertical through the bottom of the excavation. The anchors should extend at least 20 feet beyond the potential active wedge and to a greater length as necessary to develop the desired capacities.

The capacities of anchors should be determined by testing the initial anchors as outlined below. We anticipate that gravity-filled anchors will be capable of achieving an allowable bond strength of 1 to 3 kips per lineal foot of anchor, depending on the method of construction. A variety of methods is available for construction of anchors. If post-grouted anchors are utilized, we estimate that the anchors will develop resistance on the order of three times the estimated value.

We recommend that the shoring designer and contractor be responsible for selecting the appropriate bond length and installation methods to achieve the required capacity.

Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads. If the anchors are spaced at least 6 feet on-centers, reduction in the capacity of the anchors does not need to be considered due to group action.

The anchors are commonly installed at angles of 15 to 40 degrees below the horizontal; however, in many cases it is necessary to utilize steeper inclinations where adjacent private

property is present. Caving of the anchor holes should be anticipated and provisions made to minimize such caving. The anchors should be filled with concrete placed by pumping from the tip out, and the concrete should extend from the tip of the anchor to the active wedge. To minimize chances of caving, we suggest that the portion of the anchor shaft within the active wedge be backfilled with sand before testing the anchor. This portion of the shaft should be filled tightly and flushed with the face of the excavation. The sand backfill may contain a small amount of cement to allow the sand to be placed by pumping. For 8-inch-diameter or less post-grouted anchors, the anchor may be filled with concrete to the surface of the shoring.

Our representative should select a representative number of the initial anchors for 24 hour, 200 percent tests and 200 percent quick tests. The purpose of the 200 percent test is to verify the friction value assumed in design. The anchors should be tested to develop twice the assumed friction value. Where satisfactory tests are not achieved on the initial anchors, the anchor diameter and/or length should be increased until satisfactory test results are obtained.

For post-grouted anchors where concrete is used to backfill the anchor along its entire length, the test load should be computed as required to develop the appropriate friction along the entire bonded length of the anchor.

We estimate that the influence of the post-grouting and the adjacent soil within the bonded length of the anchors will be less than 5 feet from the anchor.

The total deflection during the 24 hour, 200 percent tests should not exceed 12 inches during loading. The anchor deflection should not exceed 0.75 inch during the 24 hour period, measured after the 200 percent test load is applied. If the anchor movement after the 200 percent load has been applied for six hours is less than 0.5 inch and the movement over the previous four hours has been less than 0.1 inch, the test may be terminated.
For the quick 200 percent tests, the 200 percent test load should be maintained for 30 minutes. The total deflection of the anchor during the quick 200 percent tests should not exceed 12 inches. The deflection after the 200 percent test load has been applied should not exceed 0.75 inch during the 30-minute period. Where satisfactory tests are not achieved on the initial anchors, the anchor diameter and/or length should be increased until satisfactory test results are obtained.

All of the production anchors should be pre-tested to at least 150 percent of the design load. The total deflection during the tests should not exceed 12 inches. The rate of creep under the 150 percent test should not exceed 0.1 inch over a 15 minute period for the anchor to be approved for the design loading.

After a satisfactory test, each production anchor should be locked off at the design load. The locked off load should be verified by rechecking the load in the anchor. If the locked off load varies by more than 10 percent from the design load, the load should be reset until the anchor is locked off within 10 percent of the design load. The installation of the anchors and the testing of the completed anchors should be observed by a representative of our firm.

7.4.5 Raker Bracing

As an alternative to tiebacks, raker bracing may be used to internally brace the soldier piles. If used, raker bracing could be supported laterally by temporary concrete footing (aka deadmen) or by the permanent interior footings. For design of such temporary footings poured with the bearing surface normal to the rakers inclined at 45 to 60 degrees with the vertical, a bearing value of 4,000 psf may be used for footings on the dense or stiff native soil provided the shallowest point of the footing is at least 1 foot below the lowest adjacent grade. To reduce the movement of the shoring, the rakers should be tightly wedged against the footings and/or shoring system.

7.4.6 Monitoring

Some means of monitoring the performance of the shoring system is recommended. The monitoring should consist of periodic surveying of the lateral and vertical locations of the tops of all the soldier piles. When the design of the shoring system has been finalized, we can discuss this further with the design consultants and the contractor.

It is difficult to accurately predict the amount of deflection of a shoring system. It should be realized, however, that some deflection will occur. We estimate that this deflection could be on the order of 1 inch at the top of the shored embankment. If greater deflection occurs during construction, additional bracing may be necessary to minimize settlement of the utilities in the adjacent streets. If it is desired to reduce the deflection of the shoring, a greater active pressure could be used in the shoring design.

7.4.7 Shoring Construction Considerations

Due to the presence of granular soil that may be subject to caving and shallow groundwater, casing and/or drilling mud may be required to prevent caving during the installation of soldier beams and tieback anchors.

Due to the presence of ground water at the site, we recommend placing soldier pile concrete via tremie pipe.

If utilized, vibration for solider piles should not be utilized within 40 feet of existing structures and the peak particle velocity (PPV) should not exceed ½ inch per second. If the PPV velocity is exceeded, the vibration installation operation should be terminated and a mitigation plan should be submitted by the contractor for review and approval prior to resuming vibration.
Pre-drilling, if utilized in conjunction with vibratory methods, should not extend below the bottom of the planned excavation and the diameter of the pre-drilling auger should be less than the beam diagonal.

7.5 PERMANENT BELOW-GRADE BUILDING WALLS

For static conditions, the undrained, below-grade building walls should be designed to resist a trapezoidal-shaped, earth pressure distribution and permanent hydro-static pressure as shown in Figure 8.

For seismic loading conditions, undrained below-grade building walls should be designed to resist a triangular-shaped active lateral earth pressure distribution and permanent hydrostatic pressure, in conjunction with a triangular-shaped seismic lateral earth pressure distribution as shown on Figure 9.

Please note that once foundation information for the adjacent tower located at 180 East Ocean Boulevard is available, revised recommendations will be developed that consider the influence of the adjacent foundations, as appropriate.

The upper 10 feet of the below-grade building walls should also be designed to resist the surcharge pressure resulting from adjacent traffic along Pine Avenue and East Seaside Way as shown on Figure 8 and 9.

Where the surface at the top of the shoring is sloped, the recommended lateral earth pressures should be increased as recommended in Table 1 presented in Section 7.4.

Please note that because the lower portion of the wall will be designed to hydrostatic pressure, conventional wall back-drainage provisions are not required as water that is introduced into the upper soils though irrigation or other means is not anticipated to appreciably raise the groundwater level at the site.

The building walls below grade should be waterproofed to prevent groundwater intrusion into the subterranean level.

7.6 FREE-STANDING RETAINING WALLS

7.6.1 Foundations

If required, free-standing retaining wall foundations should be established on at least 3 feet of properly compacted fill soil and/or the medium dense to dense/ very stiff to hard native soils a minimum of 2 feet below the lowest adjacent grade or floor slab. Wall foundations established in this fashion may be designed using an allowable bearing pressure of 2,500 psf.

To resist lateral loading, a coefficient of friction equal to 0.3 may be used in conjunction with a passive pressure of 300 psf per foot of embedment. The frictional resistance and passive earth pressure may be used in combination and without reduction.

7.6.2 Design Lateral Earth Pressures

Free-standing retaining walls should be designed to resist an equivalent fluid pressure equal to 30 pcf. If the surface at the top of the wall is sloped, the recommended lateral earth pressures should be increased as indicated in Table 1 presented in Section 7.4.

7.6.3 Wall Back-Drainage

Permanent retaining walls should be constructed with adequate back-drainage to prevent the buildup of hydrostatic pressure behind the walls. The installation of drainage boards on the back of the walls, in conjunction with conventional weep holes at the base of the walls, would provide adequate drainage. As an alternative, a collector pipe could be installed at the base of the wall and discharged to a suitable outlet.

7.7 SEISMIC DESIGN

The seismic design for the proposed tower development will be based on the document titled *An Alternative Procedure for Seismic Analysis and Design of Tall Buildings Located in the Los Angeles Region,* 2014 edition, by The Los Angeles Tall Buildings Structural Design Council (LATBSDC). We will perform a site specific ground motion study and develop earthquake time history records as part of that evaluation.

CBC-prescribed seismic design parameters for the podium structure are presented in Table 2.

Table 2. CBC Seismic Design Parameters

Parameter	Short Period (T_s = 0.2 second)	1 Second Period (T_1 = 1.0 second
MCE Spectral Acceleration, S	1.608	0.605
Site Class	D	
Site Coefficient	$F_a = 1.0$	$F_v = 1.5$
Adjusted Spectral Acceleration	$SM_S = 1.608$	$SM_1 = 0.907$
Design Spectral Response Acceleration Parameters	$S_{DS} = 1.072$	$S_{D1} = 0.605$

7.8 SITE PREPARATION

Site preparation for this project will primarily include exposing the bottom of foundations and floor slabs and preparing soil at the bottom of trenches. For foundation support, the exposed bottoms do not require special preparation, except when disturbed by construction activities or when undocumented fill is encountered beneath foundation bottoms. In that case, the undocumented fill should be removed and replaced with lean-mix concrete as recommended in Section 7.1.

Where encountered beneath the podium area floor slab, all undocumented fill, loose, disturbed or otherwise unsuitable soils should be removed and replaced as properly compacted fill, or alternatively, removed for a depth of 8 inches and replaced with ¾-inch minus crushed rock placed on a non-woven geotextile fabric.

It is likely that the placement of a waste-slab will be beneficial for securing and protecting the approved bottom of excavation for either or both foundations and floor slab.

For areas to receive fill and/or beneath other flatwork (walkways and driveways), the upper 6 inches should be scarified and re-compacted to the degree of relative compaction recommended in Section 7.9 of this report.

7.9 GRADING CONSIDERATIONS

7.9.1 General

If not carefully executed, site preparation can result in the presence of disturbed and/or excessively soft soil conditions. This may require additional effort to mitigate or in more extreme cases, if not detected, could result in significant costs to repair damage to flatwork or structures.

Earthwork should be planned and executed to minimize subgrade disturbance. Soil that has been disturbed during site preparation activities and/or soft or loose zones identified during probing should be removed beneath floor slabs.

7.9.2 Compaction

All granular fill material should be compacted to at least 95 percent of the maximum dry density at or near the optimum moisture content, as determined by ASTM D 1557. Cohesive fills should be compacted to at least 90 percent of the maximum dry density, as determined by ASTM D 1557, and moisture conditioned 2 to 4 percent over the optimum moisture content.

Fill material should be placed in loose lifts not exceeding 8 inches in thickness, properly moisture conditioned, and mechanically compacted to the minimum required density. For granular fills, compaction may be achieved using heavy equipment and vibration.

7.9.3 Site Drainage

Adequate site drainage should be maintained at all times. Site drainage should be collected and routed to suitable discharge locations.

7.10 MATERIALS FOR FILL

The fill material should be free of organic matter and other deleterious material and, in general, should consist of particles no larger than 3 inches in largest dimension.

The following sections provide recommendations for the re-use of on-site material in compacted fills and for the use of imported material in required fills.

On-site granular soils are suitable for use in the required fills provided particles larger than 3 inches in largest dimension are removed.

8.0 CONSTRUCTION OBSERVATION

Geotechnical testing and observation during construction is considered to be a continuing part of the geotechnical consultation. To confirm that the recommendations presented herein remain applicable, our representative should be present at the site to provide appropriate observation and testing during the following primary activities:

- Solider pile and tieback installation
- Tieback anchor testing
- Lagging installation
- Installation of wall back-drainage provisions

- Foundation bottom observation and approval
- Placement and compaction of fill material
- Removal of shoring within the public right-of-way upon completion of the project
- De-tensioning of tieback anchors

9.0 LIMITATIONS

We have prepared this report for use by Sodo Builders, LLC, and members of the design and construction team for the proposed development. The data and report can be used for estimating purposes, but our report, conclusions, and interpretations should not be construed as a warranty of the subsurface conditions and are not applicable to other sites.

Soil borings indicate soil conditions only at specific locations and only to the depths penetrated. They do not necessarily reflect soil strata or water level variations that may exist between exploration locations. If subsurface conditions differing from those described are noted during the course of excavation and construction, re-evaluation will be necessary.

The recommendations presented in this report are based on the current site development plan and structural information provided to us by the project team. If design changes are made, we should be retained to review our conclusions and recommendations and to provide a written evaluation or modification.

The scope of our services does not include services related to construction safety precautions, and our recommendations are not intended to direct the contractor's methods, techniques, sequences or procedures, except as specifically described in our report for consideration in design.

Within the limitations of scope, schedule, and budget, our services have been executed in accordance with that degree of skill and care ordinarily exercised by reputable geotechnical consultants practicing in this area at the time this report was prepared. No warranty or other conditions, express or implied, should be understood.

♦ ♦ ♦

We appreciate the opportunity to be of continued service to you. Please call if you have questions concerning this report or if we can provide additional services.

Sincerely,

GeoDesign, Inc.

Eric Torres, P.E.
Staff Engineer

REGISTERED PROFESSIONAL ENGINEER
JOHN W. HALSETH
82559
CIVIL
STATE OF CALIFORNIA

Signed 01/24/2017

John W. Halseth, P.E.
Project Engineer

Christopher J. Zadoorian, G.E.
Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
CHRISTOPHER JOHN ZADOORIAN
No. 2493
GEOTECHNICAL
STATE OF CALIFORNIA

Signed 01/24/2017

FIGURES

VICINITY MAP BASED ON AERIAL PHOTOGRAPH OBTAINED FROM GOOGLE EARTH PRO®

SITE LOCATION
LATITUDE: 33.766265°
LONGITUDE: -118.191967°

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SODOBUILD-2-01	VICINITY MAP	
JANUARY 2017	PROPOSED HOTEL DEVELOPMENT LONG BEACH, CA	FIGURE 1

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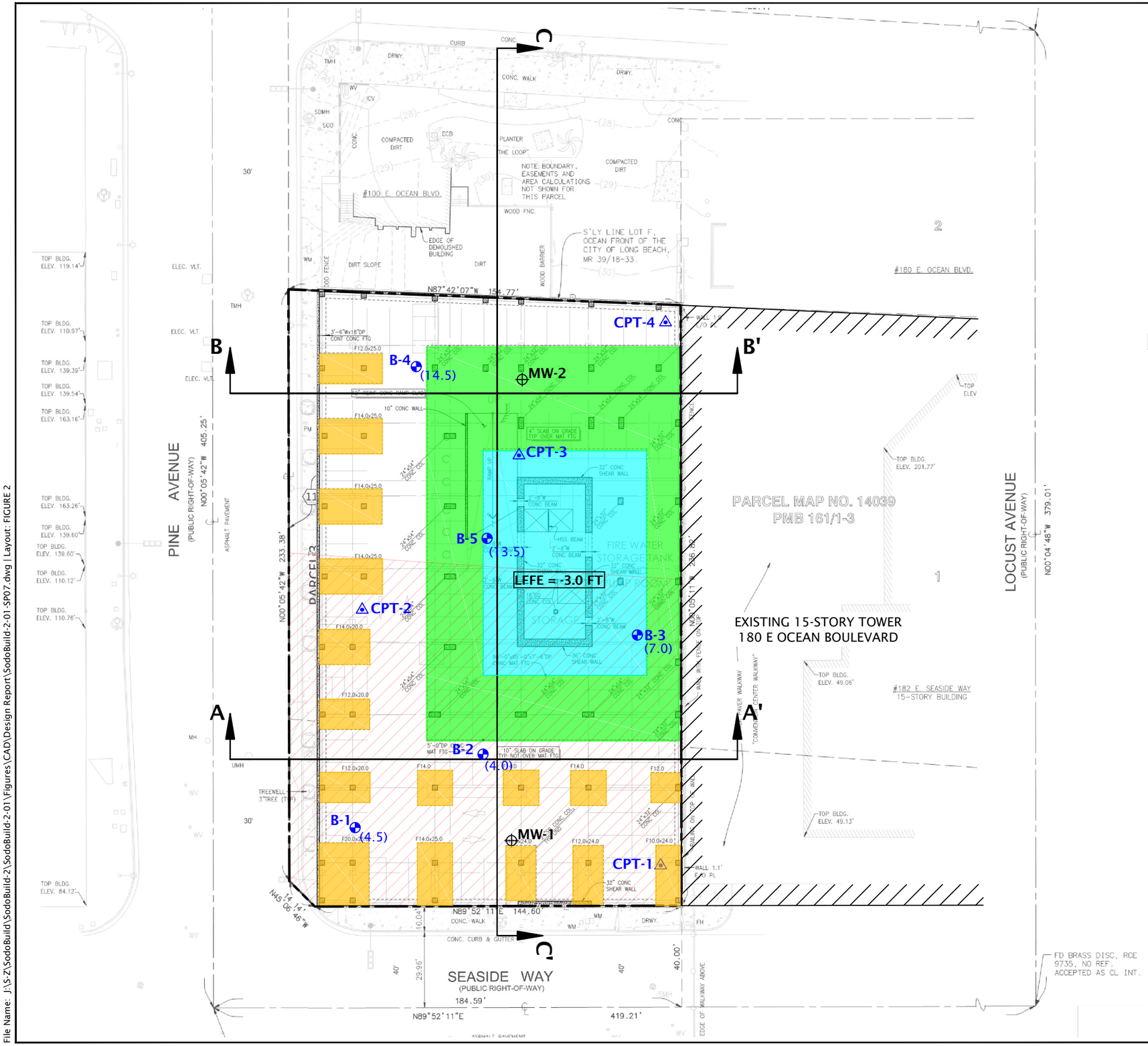
File Name: J:\S-Z\SodoBuild\SodoBuild-2\SodoBuild-2-01\Figures\CAD\Design Report\SodoBuild-2-01-VM01.dwg | Layout: FIGURE 1



LEGEND:

B-1 (4.5)	BORING FILL THICKNESS (FEET)
CPT-1	CONE PENETROMETER TEST
MW-1	GROUNDWATER MONITORING WELL (MACTEC 2004)
	LIMITS OF LIQUEFACTION HAZARD ZONE BASED ON CGS LONG BEACH QUADRANGLE, MARCH 1999
	7.5-FOOT-THICK MAT FOUNDATION
	5-FOOT-THICK MAT FOUNDATION
	SPREAD FOOTINGS
	EXISTING BUILDING
	SITE BOUNDARY
LFFE = -3.0 FT	LOWEST FINISHED FLOOR ELEVATION
A A'	CROSS SECTION

N

0 40 80

(SCALE IN FEET)

NOTES:

1. SITE PLAN BASED ON ABOVE GRADE PARKING IMAGE PREPARED BY GBD ARCHITECTS INCORPORATED.
2. SITE PLAN BASED ON ALTA/NSPS LAND TITLE SURVEY SHEET 1 IMAGE DATED SEPTEMBER 21, 2016 AND PREPARED BY KPFF CONSULTING ENGINEERS.

SITE PLAN

PROPOSED HOTEL DEVELOPMENT
LONG BEACH, CA

FIGURE 2

SODOBUILD-2-01

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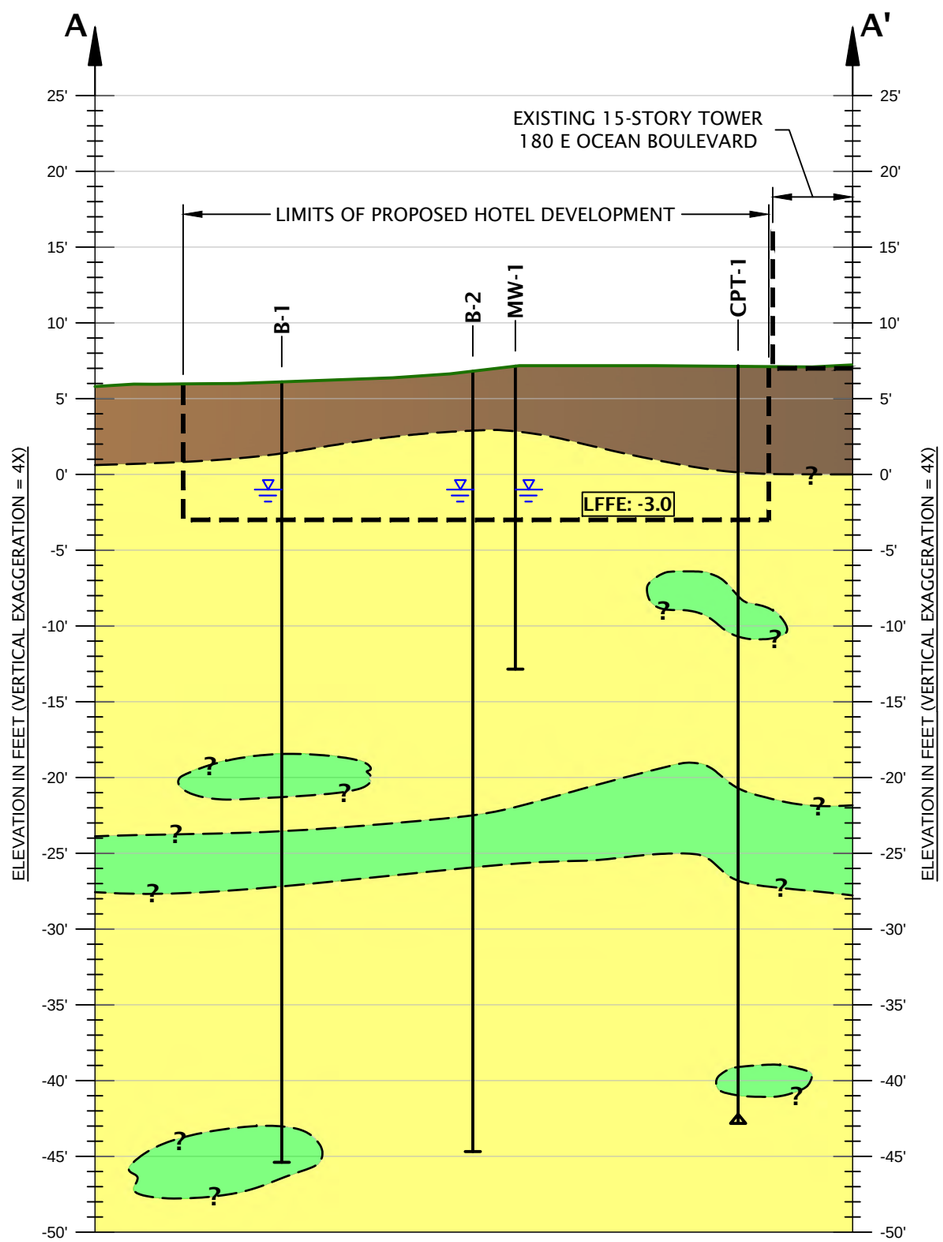
File Name: J:\S-Z\SodoBuild\SodoBuild-2\SodoBuild-2-01\Figures\CAD\Design Report\SodoBuild-2-01-SP07.dwg | Layout: FIGURE 2



LEGEND:

- EXISTING TOPOGRAPHY
- BORING
- CONE PENETROMETER
- INTERPRETED CONTACT
- GROUNDWATER LEVEL
- EXISTING FILL
- MEDIUM DENSE TO VERY DENSE SAND, SAND WITH SILT, AND SILTY SAND
- VERY STIFF TO HARD SILT AND CLAY
- LFFE: -3.0 — PROPOSED LOWEST FINISHED FLOOR ELEVATION (FEET)

0 10 20
(VERTICAL SCALE IN FEET)

0 40 80
(HORIZONTAL SCALE IN FEET)

CROSS SECTION A-A'

PROPOSED HOTEL DEVELOPMENT
LONG BEACH, CA

FIGURE 3

SODOBUILD-2-01

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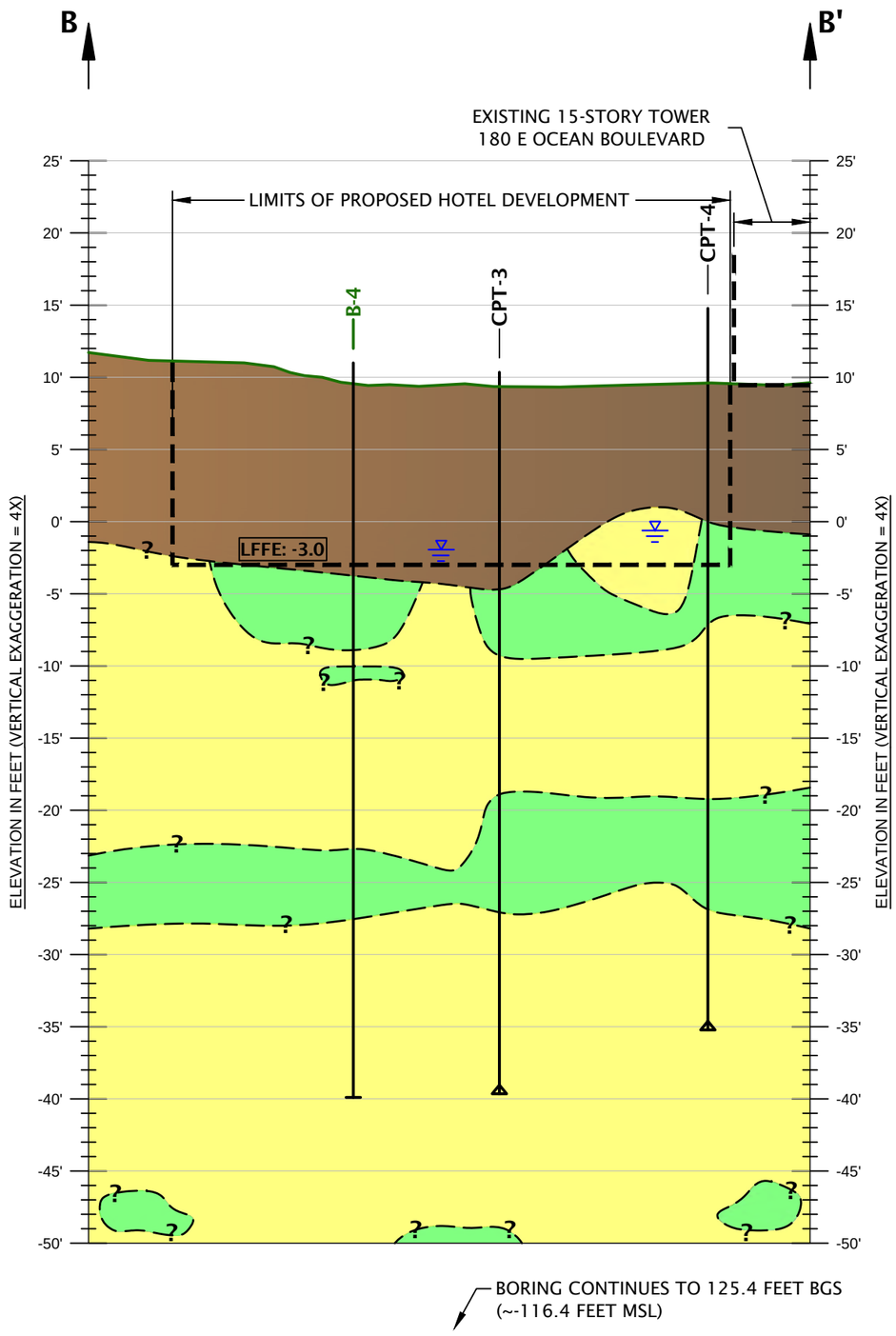
File Name: J:\S-Z\SodoBuild\SodoBuild-2\SodoBuild-2-01\Figures\CAD\Design Report\SodoBuild-2-01-CS06.dwg | Layout: FIGURE 3



LEGEND:

- EXISTING TOPOGRAPHY
- BORING
- CONE PENETROMETER
- INTERPRETED CONTACT
- GROUNDWATER LEVEL
- EXISTING FILL
- MEDIUM DENSE TO VERY DENSE SAND, SAND WITH SILT, AND SILTY SAND
- VERY STIFF TO HARD SILT AND CLAY
- LFFE: -3.0 — PROPOSED LOWEST FINISHED FLOOR ELEVATION (FEET)

0 10 20
(VERTICAL SCALE IN FEET)

0 40 80
(HORIZONTAL SCALE IN FEET)

CROSS SECTION C-C'

FIGURE 4

PROPOSED HOTEL DEVELOPMENT
LONG BEACH, CA

SODOBUILD-2-01

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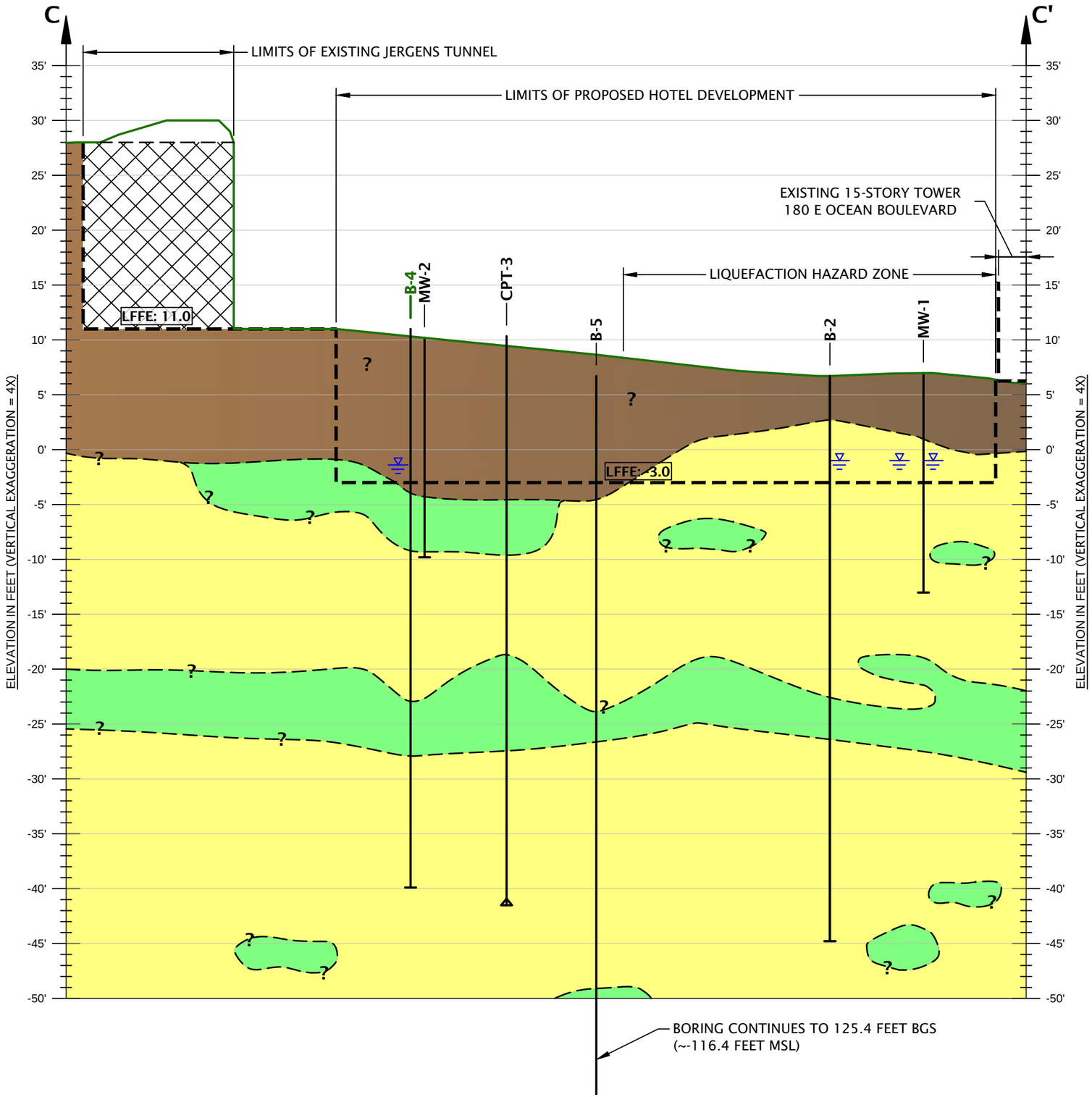
File Name: J:\S-Z\SodoBuild\SodoBuild-2\SodoBuild-2-01\Figures\CAD\Design Report\SodoBuild-2-01-CS06.dwg | Layout: FIGURE 4



LEGEND:

- EXISTING TOPOGRAPHY
- BORING
- CONE PENETROMETER
- INTERPRETED CONTACT
- GROUNDWATER LEVEL
- EXISTING FILL
- MEDIUM DENSE TO VERY DENSE SAND, SAND WITH SILT, AND SILTY SAND
- VERY STIFF TO HARD SILT AND CLAY
- LFFE: -3.0 — PROPOSED LOWEST FINISHED FLOOR ELEVATION (FEET)

0 10 20
(VERTICAL SCALE IN FEET)

0 40 80
(HORIZONTAL SCALE IN FEET)

CROSS SECTION B-B'

FIGURE 5

PROPOSED HOTEL DEVELOPMENT
LONG BEACH, CA

SODOBUILD-2-01

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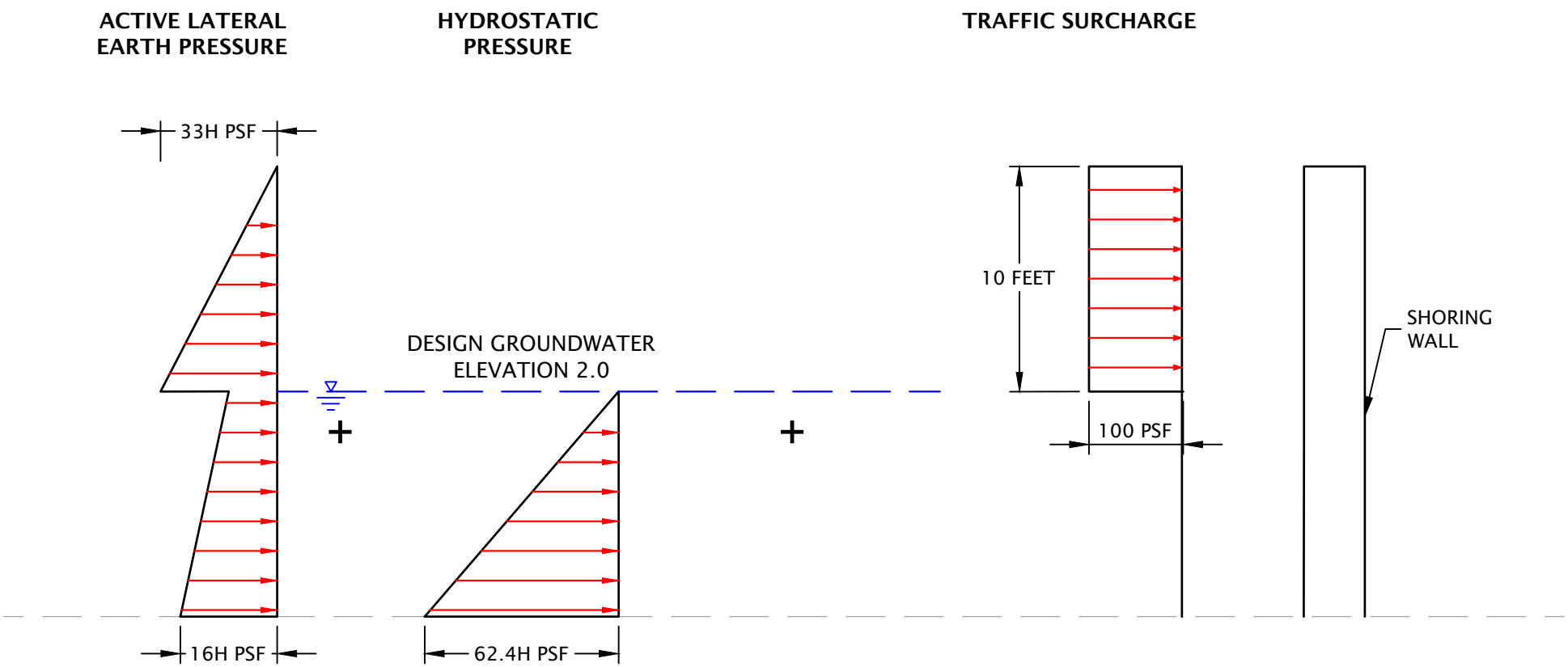
File Name: J:\S-Z\SodoBuild\SodoBuild-2\SodoBuild-2-01\Figures\CAD\Design Report\SodoBuild-2-01-CS06.dwg | Layout: FIGURE 5



INFLUENCE FROM ADJACENT TOWER FOUNDATION NOT INCLUDED

NOT TO SCALE

NOTES:

1. FIGURE SHOULD BE USED IN CONJUNCTION WITH REPORT TEXT.
2. THE LATERAL EARTH PRESSURES ARE UNFACTORED.

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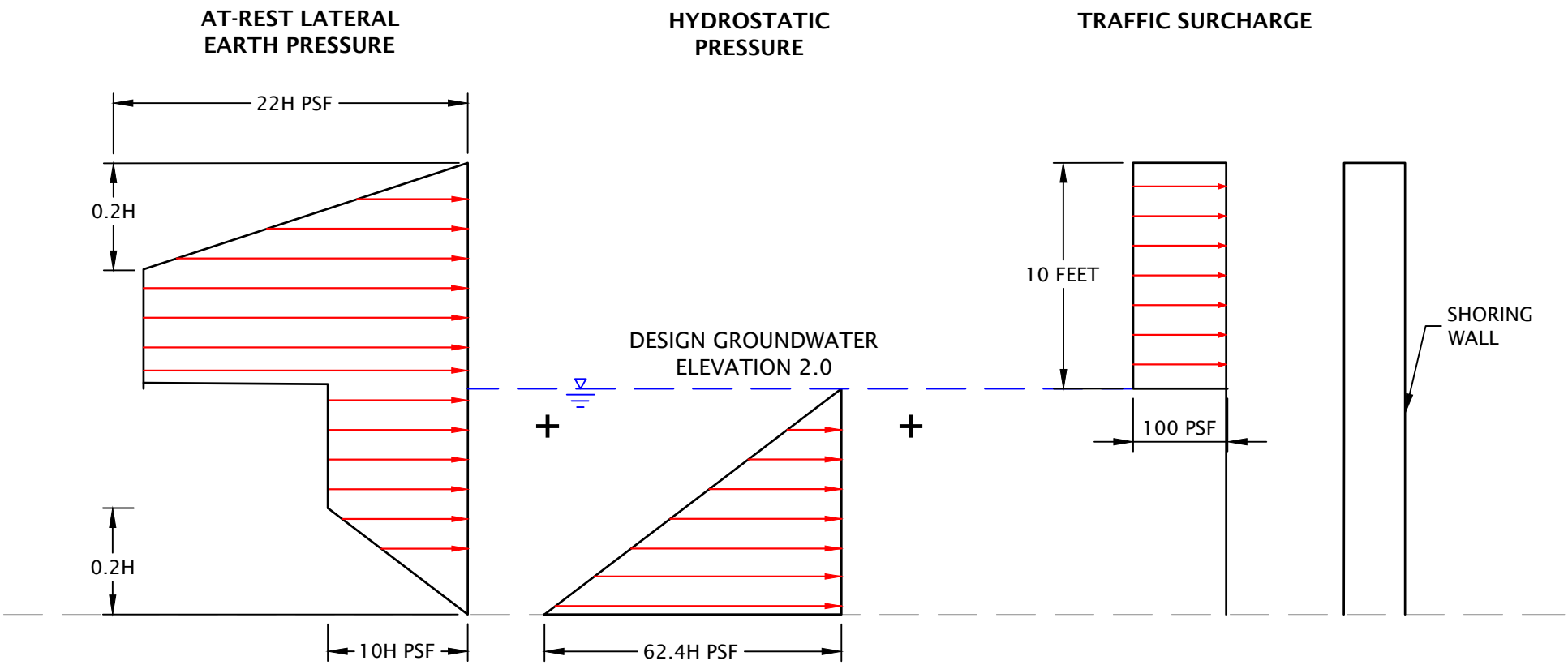
SODOBUILD-2-01	**UNDRAINED CANTILEVER SHORING LATERAL EARTH PRESSURE**	
JANUARY 2017	PROPOSED HOTEL DEVELOPMENT LONG BEACH, CA	**FIGURE 6**



INFLUENCE FROM ADJACENT TOWER FOUNDATION NOT INCLUDED

NOT TO SCALE

NOTES:

1. FIGURE SHOULD BE USED IN CONJUNCTION WITH REPORT TEXT.
2. THE LATERAL EARTH PRESSURES ARE UNFACTORED.

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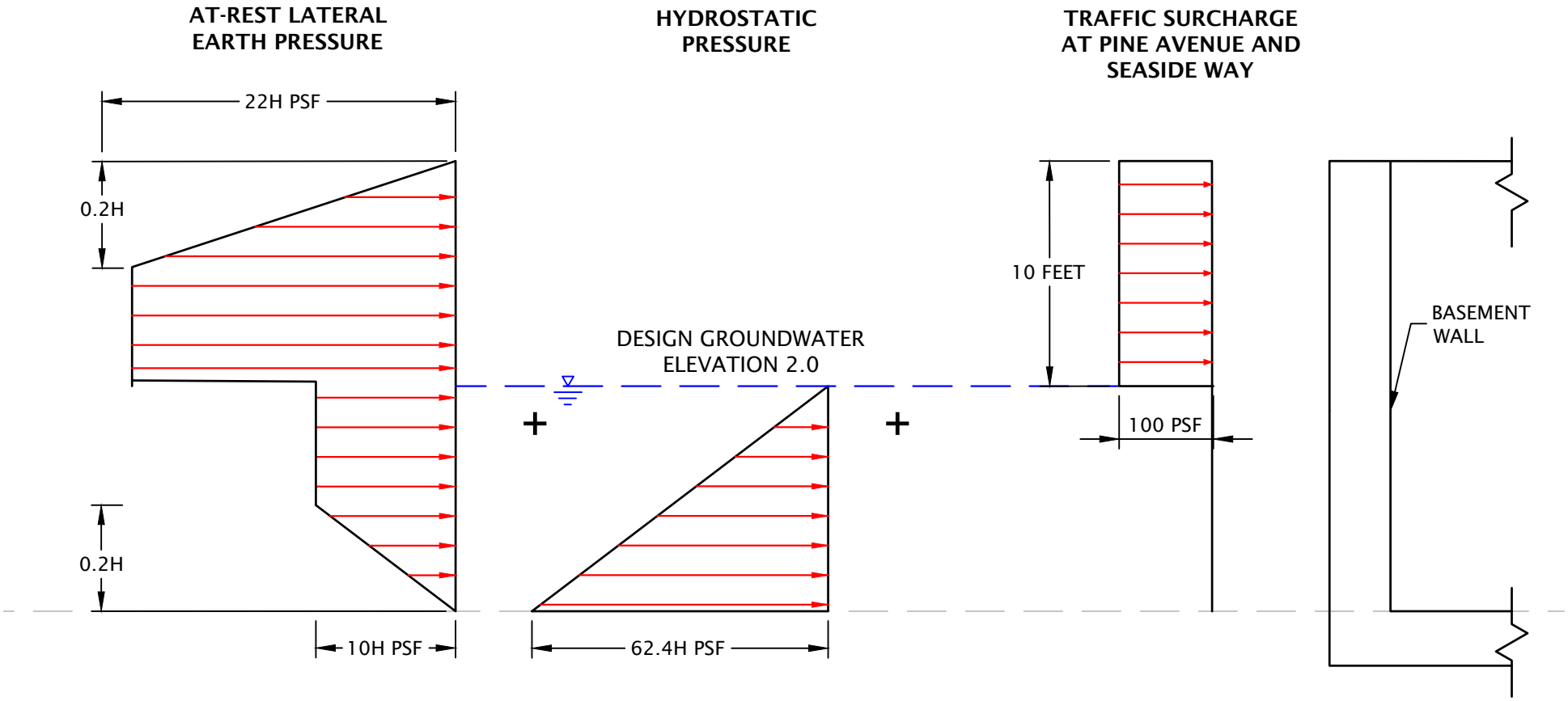
SODOBUILD-2-01	**UNDRAINED INTERNALLY BRACED SHORING LATERAL EARTH PRESSURE**	
JANUARY 2017	PROPOSED HOTEL DEVELOPMENT LONG BEACH, CA	**FIGURE 7**



INFLUENCE FROM ADJACENT TOWER FOUNDATION NOT INCLUDED

NOT TO SCALE

NOTES:

1. FIGURE SHOULD BE USED IN CONJUNCTION WITH REPORT TEXT.
2. THE LATERAL EARTH PRESSURES ARE UNFACTORED.

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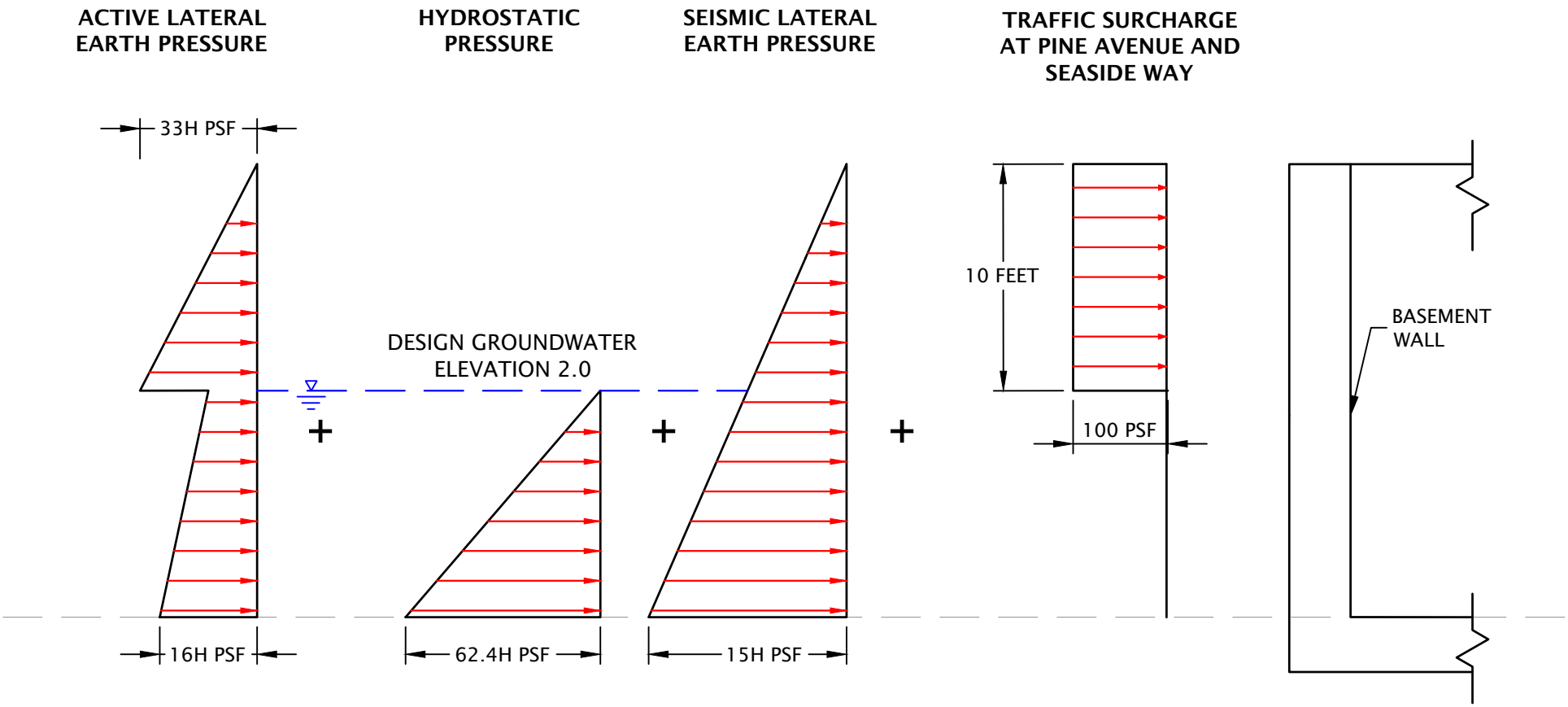
SODOBUILD-2-01	**UNDRAINED AT-REST LATERAL EARTH PRESSURE AND SURCHARGE PERMANENT BELOW-GRADE WALLS**	
JANUARY 2017	PROPOSED HOTEL DEVELOPMENT LONG BEACH, CA	**FIGURE 8**



INFLUENCE FROM ADJACENT TOWER FOUNDATION NOT INCLUDED

NOT TO SCALE

NOTES:

1. FIGURE SHOULD BE USED IN CONJUNCTION WITH REPORT TEXT.
2. THE LATERAL EARTH PRESSURES ARE UNFACTORED.

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SODOBUILD-2-01	**UNDRAINED SEISMIC LATERAL EARTH PRESSURE AND SURCHARGE PERMANENT BELOW-GRADE WALLS**	
JANUARY 2017	PROPOSED HOTEL DEVELOPMENT LONG BEACH, CA	**FIGURE 9**

APPENDIX A

APPENDIX A

SUBSURFACE EXPLORATIONS

GENERAL

We explored the subsurface conditions at the site by drilling 5 borings (B-1 through B-5) to depths ranging from 50.9 to 125.4 feet BGS using mud rotary drilling equipment and advanced four cone penetration test (CPT) soundings at the site to depths ranging from 6.7 to 50.1 feet BGS. Refusal was encountered at a depth of 6.7 feet in CPT-2 after three attempts.

We maintained a log of the soil conditions encountered in each boring and collected relatively undisturbed samples at regular intervals in each boring. SPTs were performed at selected depths.

Upon completion borings were backfilled with a bentonite-cement grout mixture and we restored to surface to match the pre-existing condition.

Drill cuttings from each boring were placed in drums pending the results of chemical testing and subsequently disposed of off-site by a licensed materials hauler.

Logs of the borings are presented in this appendix. The logs of the CPT soundings are presented in Appendix B.

SOIL SAMPLING

Samples were obtained from the borings using modified California split-spoon samplers in general accordance with ASTM D 3550. The split-spoon samplers were driven into the soil with a 140-pound hammer free-falling 30 inches. The samplers were driven 18 inches or to refusal as indicated on the exploration logs. The number of blows required to drive the sampler the final 12 inches (or less if refusal is met) is recorded on the exploration logs presented in this appendix, unless otherwise noted.

In addition, SPTs were performed in the borings in general accordance with ASTM D 1586. The 2-inch-diameter, split-spoon sampler was driven into the soil with a 140-pound hammer free-falling 30 inches. The samplers were driven a total distance of 18 inches or to refusal. The number of blow counts required to drive the sampler the final 12 inches is recorded (or less if refusal is met) on the exploration logs, which are presented in this appendix.

SOIL CLASSIFICATION

The soil samples were classified in accordance with the exploration key and soil classification system sheets for each episode of drilling, which are included in this appendix prior to the exploration logs. The exploration logs indicate the depths at which the soils or their characteristics change, although the change actually could be gradual. If the change occurred between sample locations, the depth was interpreted. Classifications and sampling intervals are presented on the exploration logs included in this appendix.

GEOPHYSICAL TESTING

Suspension P-S velocity logging was performed in our current boring B-5 by GEOVision, Inc. The suspension P-S velocity logging was performed to estimate the stiffness of the subsurface soil profile and was performed for the upper 125 feet in B-5.

The suspension logging method uses a 7-meter probe that contains a source and two receivers. The probe is lowered down the drilled hole where the source generates a pressure wave in the drilling fluid within the hole. The pressure wave is converted to seismic P- and S-waves at the boring sidewalls; at each receiver, the P- and S-waves are converted back to pressure waves. The elapsed time between wave arrivals at the receivers is used to determine the average velocity of a 1-meter-high column of soil. The process is repeated for the full depth of the boring to obtain a continuous log of the boring.

The results of the P-S logging are presented in Appendix C.

LABORATORY TESTING

MOISTURE CONTENT

The natural moisture content of selected samples was obtained from the exploration in general accordance with ASTM D 2216. The natural moisture content is a ratio of the weight of the water to soil in a test sample and is expressed as a percentage. The test results are presented in this appendix.

DRY DENSITY

Selected soil samples were tested to determine the in situ dry density. The tests were performed in general accordance with ASTM D 2937. The dry density is defined as the ratio of the dry weight of the soil sample to the volume of that sample. The dry density typically is expressed in units of pcf. The test results are presented in this appendix.

CONSOLIDATION TESTING

One-dimensional consolidation tests were performed in general accordance with ASTM D 2435 on relatively undisturbed samples obtained from the geotechnical borings. The tests measure the volume change of a soil sample under predetermined loads. The test results are presented in this appendix.

STRENGTH TESTING

Direct shear tests were completed on select samples in general accordance with ASTM D 3080. The test results are presented in this appendix.

ATTERBERG LIMITS

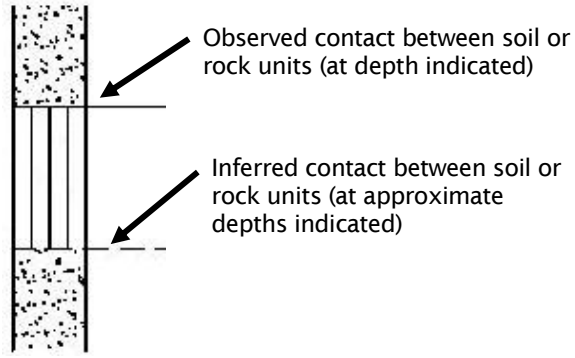
The plastic limit and liquid limit (Atterberg limits) of select samples were determined in accordance with ASTM D 2937. The results of the Atterberg limit tests are included in this appendix.

GRAIN-SIZE DISTRIBUTION

Grain-size distribution was performed in accordance with ASTM D422. The results of the testing are presented in the appendix.

SYMBOL	SAMPLING DESCRIPTION
	Location of sample obtained in general accordance with ASTM D 1586 Standard Penetration Test with recovery
	Location of sample obtained using thin-wall Shelby tube or Geoprobe® sampler in general accordance with ASTM D 1587 with recovery
	Location of sample obtained using Dames & Moore sampler and 300-pound hammer or pushed with recovery
	Location of sample obtained using Dames & Moore and 140-pound hammer or pushed with recovery
	Location of sample obtained using 3-inch-O.D. California split-spoon sampler and 140-pound hammer
	Location of grab sample
	Rock coring interval
	Water level during drilling
	Water level taken on date shown

Graphic Log of Soil and Rock Types



GEOTECHNICAL TESTING EXPLANATIONS

ATT	Atterberg Limits	P	Pushed Sample
CBR	California Bearing Ratio	PP	Pocket Penetrometer
CON	Consolidation	P200	Percent Passing U.S. Standard No. 200 Sieve
DD	Dry Density		
DS	Direct Shear	RES	Resilient Modulus
HYD	Hydrometer Gradation	SIEV	Sieve Gradation
MC	Moisture Content	TOR	Torvane
MD	Moisture-Density Relationship	UC	Unconfined Compressive Strength
NP	Nonplastic	VS	Vane Shear
OC	Organic Content	kPa	Kilopascal

ENVIRONMENTAL TESTING EXPLANATIONS

CA	Sample Submitted for Chemical Analysis	ND	Not Detected
P	Pushed Sample	NS	No Visible Sheen
PID	Photoionization Detector Headspace Analysis	SS	Slight Sheen
		MS	Moderate Sheen
ppm	Parts per Million	HS	Heavy Sheen

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EXPLORATION KEY

TABLE A-1

RELATIVE DENSITY - COARSE-GRAINED SOIL

Relative Density	Standard Penetration Resistance	Dames & Moore Sampler (140-pound hammer)	Dames & Moore Sampler (300-pound hammer)
Very Loose	0 – 4	0 - 11	0 - 4
Loose	4 – 10	11 - 26	4 - 10
Medium Dense	10 – 30	26 - 74	10 - 30
Dense	30 – 50	74 - 120	30 - 47
Very Dense	More than 50	More than 120	More than 47

CONSISTENCY - FINE-GRAINED SOIL

Consistency	Standard Penetration Resistance	Dames & Moore Sampler (140-pound hammer)	Dames & Moore Sampler (300-pound hammer)	Unconfined Compressive Strength (tsf)
Very Soft	Less than 2	Less than 3	Less than 2	Less than 0.25
Soft	2 - 4	3 – 6	2 - 5	0.25 - 0.50
Medium Stiff	4 - 8	6 – 12	5 - 9	0.50 - 1.0
Stiff	8 - 15	12 – 25	9 - 19	1.0 - 2.0
Very Stiff	15 - 30	25 – 65	19 – 31	2.0 - 4.0
Hard	More than 30	More than 65	More than 31	More than 4.0

PRIMARY SOIL DIVISIONS			GROUP SYMBOL	GROUP NAME
COARSE-GRAINED SOIL (more than 50% retained on No. 200 sieve)	GRAVEL (more than 50% of coarse fraction retained on No. 4 sieve)	CLEAN GRAVEL (< 5% fines)	GW or GP	GRAVEL
		GRAVEL WITH FINES (≥ 5% and ≤ 12% fines)	GW-GM or GP-GM	GRAVEL with silt
			GW-GC or GP-GC	GRAVEL with clay
		GRAVEL WITH FINES (> 12% fines)	GM	silty GRAVEL
			GC	clayey GRAVEL
			GC-GM	silty, clayey GRAVEL
	SAND (50% or more of coarse fraction passing No. 4 sieve)	CLEAN SAND (<5% fines)	SW or SP	SAND
		SAND WITH FINES (≥ 5% and ≤ 12% fines)	SW-SM or SP-SM	SAND with silt
			SW-SC or SP-SC	SAND with clay
		SAND WITH FINES (> 12% fines)	SM	silty SAND
			SC	clayey SAND
			SC-SM	silty, clayey SAND
FINE-GRAINED SOIL (50% or more passing No. 200 sieve)	SILT AND CLAY	Liquid limit less than 50	ML	SILT
			CL	CLAY
			CL-ML	silty CLAY
			OL	ORGANIC SILT or ORGANIC CLAY
		Liquid limit 50 or greater	MH	SILT
			CH	CLAY
			OH	ORGANIC SILT or ORGANIC CLAY
HIGHLY ORGANIC SOIL			PT	PEAT

MOISTURE CLASSIFICATION

Term	Field Test
dry	very low moisture, dry to touch
moist	damp, without visible moisture
wet	visible free water, usually saturated

ADDITIONAL CONSTITUENTS

Secondary granular components or other materials such as organics, man-made debris, etc.

Percent	Silt and Clay In: Fine-Grained Soil	Silt and Clay In: Coarse-Grained Soil	Percent	Sand and Gravel In: Fine-Grained Soil	Sand and Gravel In: Coarse-Grained Soil
< 5	trace	trace	< 5	trace	trace
5 – 12	minor	with	5 – 15	minor	minor
> 12	some	silty/clayey	15 – 30	with	with
			> 30	sandy/gravelly	Indicate %

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SOIL CLASSIFICATION SYSTEM

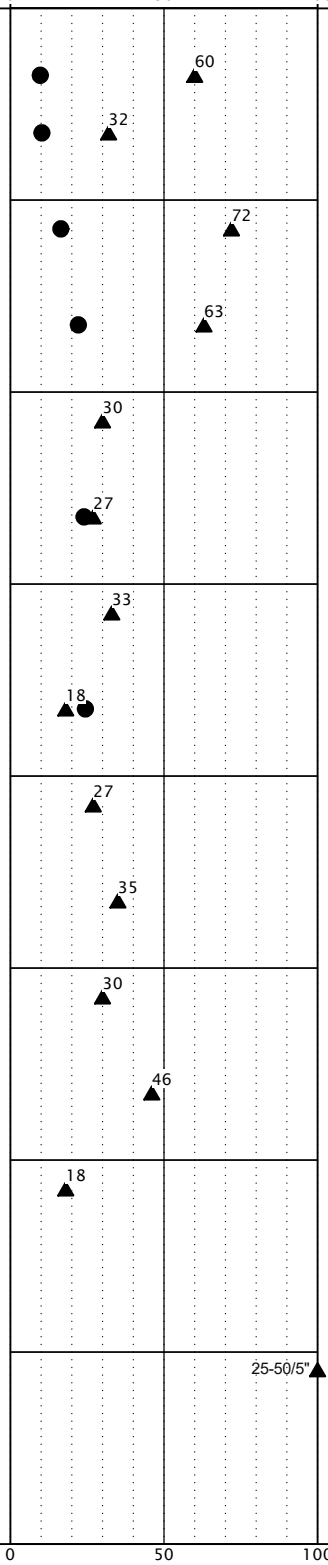
TABLE A-2

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	INSTALLATION AND COMMENTS
0	ASPHALT CONCRETE (3.0 inches).	6.0 / 5.7 / 0.3		
	Hard, gray-brown, sandy SILT (ML), minor asphalt fragments, trace brick fragments; moist, sand is fine - **FILL**.		DD	DD = 126 pcf
			DD DS	DD = 122 pcf
	Medium dense, gray-brown, silty SAND (SM); moist, fine - **FILL**.	2.5 / 3.5 / 1.5 / 4.5		
5	Dense, gray SAND (SP), trace silt; moist, fine.		DD	DD = 97 pcf
	wet at 7.0 feet		DD	DD = 100 pcf
10	Dense, gray SAND with silt (SP-SM); wet, fine.	-3.5 / 9.5		
	Medium dense to dense, gray, silty SAND (SM); wet, fine.	-6.0 / 12.0	P200	P200 = 12%
15	gray-brown, trace shell fragments at 15.0 feet		P200	P200 = 17%
20	gray at 22.5 feet			
25	Very stiff, gray-brown, sandy SILT (ML); moist, sand is fine.	-18.5 / 24.5		
	Dense, gray SAND (SP), trace silt; wet, fine.	-21.0 / 27.0		
30	Very stiff, gray SILT (ML), some clay; moist.	-23.5 / 29.5	ATT PP	PP = 2.5 tsf LL = 54% PL = 31%
	gray-brown at 31.0 feet			
	Dense to very dense, gray, silty SAND (SM), trace clay; wet, fine.	-27.0 / 33.0		
35				
40				

GRAPHIC LOG

SAMPLE

▲ BLOW COUNT
● MOISTURE CONTENT %
RQD% CORE REC%



7.0 feet during drilling

DRILLED BY: SoCal Drilling

LOGGED BY: ENT

COMPLETED: 09/13/16

BORING METHOD: mud rotary (see document text)

BORING BIT DIAMETER: 4 7/8 inches

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BORING B-1

PROPOSED HOTEL DEVELOPMENT
LONG BEACH, CA

FIGURE A-1

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % ▥ RQD% ▨ CORE REC% (0 – 50 – 100)	INSTALLATION AND COMMENTS
40		(continued from previous page)				▲ 36	
45		gray-brown, without clay at 45.0 feet				▲ 58	
			-43.0 / 49.0				
50		Hard, gray-brown SILT (ML), some clay; moist.		ATT PP		● ▲ 43	PP = 3.0 tsf LL = 39% PL = 33%
			-45.5 / 51.5				
55		**Exploration completed at a depth of 51.5 feet.** **Groundwater measured at a depth of 7.0 feet BGS during drilling.** **Backfilled with bentonite chips.**					
60							
65							
70							
75							
80							

DRILLED BY: SoCal Drilling LOGGED BY: ENT COMPLETED: 09/13/16

BORING METHOD: mud rotary (see document text) BORING BIT DIAMETER: 4 7/8 inches

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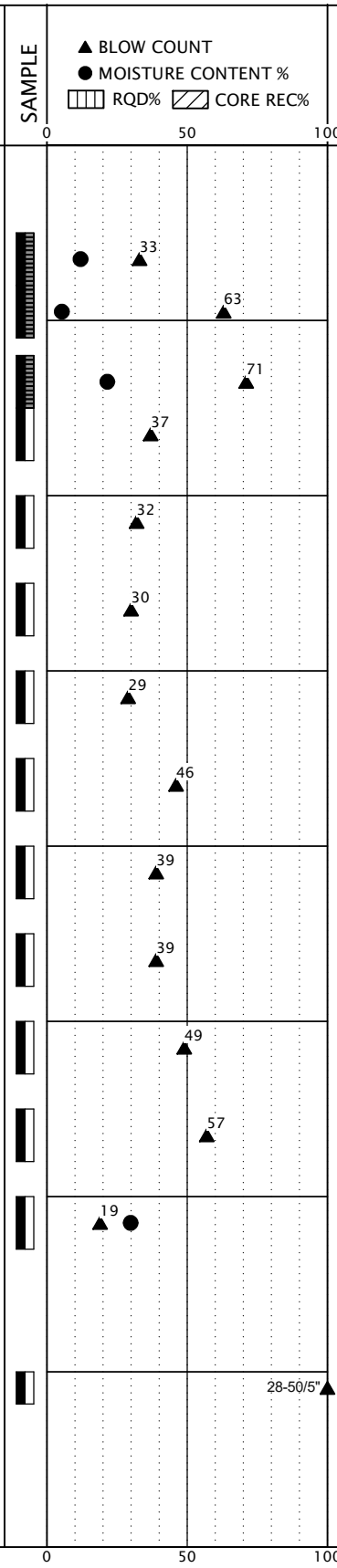
BORING B-1
(continued)

PROPOSED HOTEL DEVELOPMENT
LONG BEACH, CA

FIGURE A-1

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	INSTALLATION AND COMMENTS
0	ASPHALT CONCRETE (3.0 inches).	7.0 / 6.7			
	Medium dense, brown, silty SAND (SM); moist, fine - **FILL**.	0.3			
	Dense, gray SAND (SP), trace silt; moist, fine.	3.0 / 4.0			
5	wet at 8.0 feet				
10	Dense, gray SAND with silt (SP-SM); wet, fine.	-2.5 / 9.5			
	medium dense at 12.5 feet				
15	Medium dense to dense, gray-brown, silty SAND (SM); wet, fine.	-7.5 / 14.5			
20	gray, minor shell fragments at 20.0 feet				
	without shell fragments at 22.5 feet				
25	Dense, gray-brown SAND (SP), trace silt; wet, fine to medium.	-17.5 / 24.5			
	Very dense, gray SAND with silt (SP-SM); wet, fine.	-20.0 / 27.0			
30	Very stiff, gray-brown SILT (ML), some clay; moist.	-22.5 / 29.5			
	Very dense, gray-brown, silty SAND (SM); wet, fine to medium.	-26.0 / 33.0			
35					
40					



DRILLED BY: SoCal Drilling

LOGGED BY: ENT

COMPLETED: 09/13/16

BORING METHOD: mud rotary (see document text)

BORING BIT DIAMETER: 4 7/8 inches

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BORING B-2

PROPOSED HOTEL DEVELOPMENT
LONG BEACH, CA

FIGURE A-2

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % ▥ RQD% ▨ CORE REC% (0 – 50 – 100)	INSTALLATION AND COMMENTS
40		(continued from previous page)				54	
45		fine to coarse at 45.0 feet				68	
50						77	
		Exploration completed at a depth of 51.5 feet.	-44.5 / 51.5				
		Groundwater measured at a depth of 8.0 feet BGS during drilling.					
55		**Backfilled with bentonite chips.**					
60							
65							
70							
75							
80							

DRILLED BY: SoCal Drilling LOGGED BY: ENT COMPLETED: 09/13/16

BORING METHOD: mud rotary (see document text) BORING BIT DIAMETER: 4 7/8 inches

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BORING B-2
(continued)

PROPOSED HOTEL DEVELOPMENT
LONG BEACH, CA

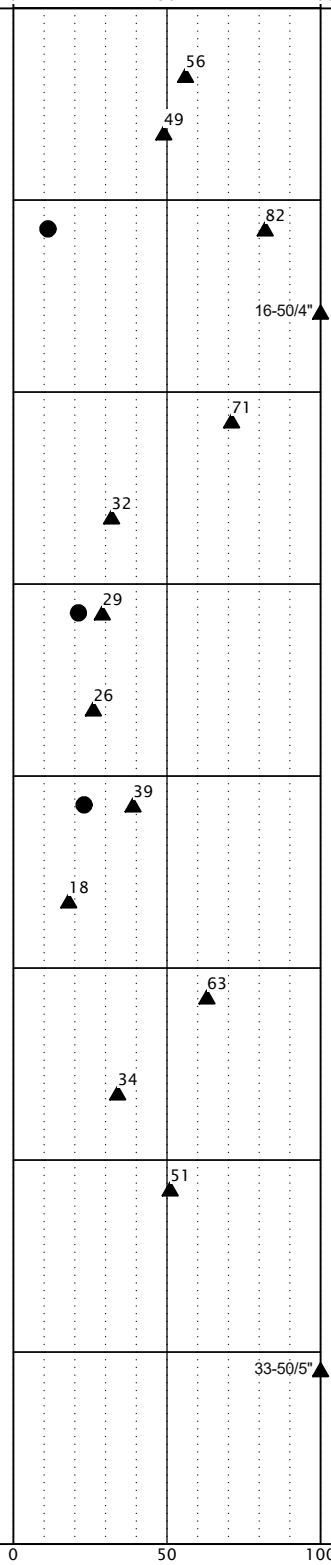
FIGURE A-2

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	INSTALLATION AND COMMENTS
0	ASPHALT CONCRETE (1.0 inch).	8.0 / 7.9 / 0.1		
	Dense, brown, silty SAND (SM), trace gravel; moist, fine, gravel is fine to coarse - **FILL**.			
5	very dense, trace brick fragments at 5.0 feet		DD DS	DD = 121 pcf
	Very dense, gray SAND with silt (SP-SM); wet, fine.	1.0 / 7.0		
	Dense, gray, silty SAND (SM); wet, fine.	-1.0 / 9.0		
10	trace shell fragments at 12.5 feet			
15	Very stiff, gray, sandy SILT (ML); wet, sand is fine.	-6.5 / 14.5	DD DS PP	DD = 106 pcf PP = 2.0 tsf
	Medium dense, gray, silty SAND (SM); wet, fine.	-9.0 / 17.0		
20	Medium dense, gray SAND with silt (SP-SM); wet, fine to medium.	-13.0 / 21.0	DD DS	DD = 99 pcf
	fine to coarse at 22.5 feet			
25	Dense, gray-brown, silty SAND (SM); wet, fine.	-16.5 / 24.5		
	Hard, gray-brown SILT (ML); moist.	-19.0 / 27.0	ATT PP	PP = 4.0 tsf LL = 48% PL = 39%
30			PP	PP = 4.5 tsf
	Very dense, gray-brown, silty SAND (SM); wet, fine to medium.	-25.0 / 33.0		
35				
40				

GRAPHIC LOG

SAMPLE

▲ BLOW COUNT
● MOISTURE CONTENT %
RQD% CORE REC%



9.5 feet, during drilling

DRILLED BY: SoCal Drilling

LOGGED BY: ENT

COMPLETED: 11/09/16

BORING METHOD: mud rotary (see document text)

BORING BIT DIAMETER: 4 7/8 inches

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BORING B-3

PROPOSED HOTEL DEVELOPMENT
LONG BEACH, CA

FIGURE A-3

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % ▥ RQD% ▨ CORE REC% (0 – 50 – 100)	INSTALLATION AND COMMENTS
40		fine to coarse at 40.0 feet				27-50/3" ▲	
		Very dense, gray SAND with silt (SP-SM); wet, fine to medium.	-35.0 / 43.0				
45						35-50/4" ▲	
50						41-50/5" ▲	
		Exploration completed at a depth of 50.9 feet.	-42.9 / 50.9				
		Groundwater measured at a depth of 9.5 feet BGS during drilling.					
55		**Backfilled with bentonite chips.**					
60							
65							
70							
75							
80							

DRILLED BY: SoCal Drilling LOGGED BY: ENT COMPLETED: 11/09/16

BORING METHOD: mud rotary (see document text) BORING BIT DIAMETER: 4 7/8 inches

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BORING B-3
(continued)

PROPOSED HOTEL DEVELOPMENT
LONG BEACH, CA

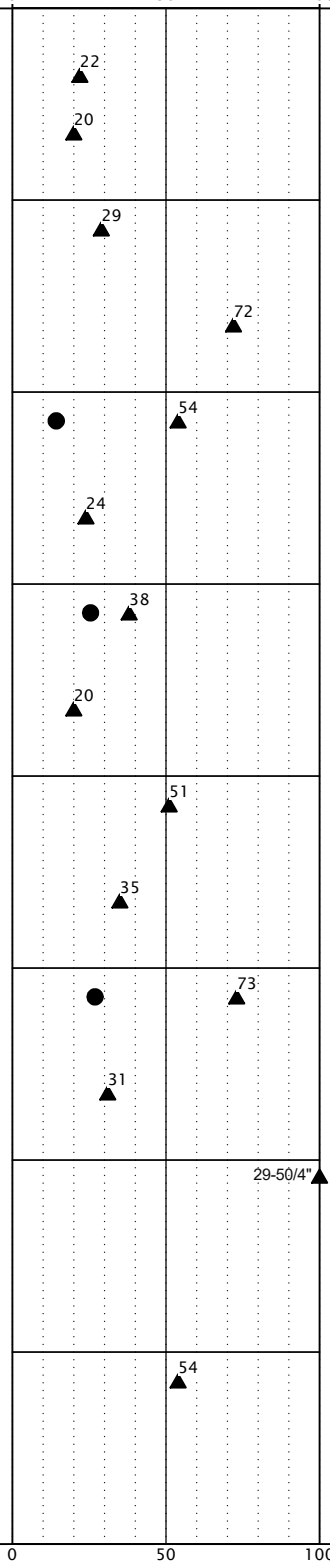
FIGURE A-3

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	INSTALLATION AND COMMENTS
0	ASPHALT CONCRETE (2.0 inches).	11.0 / 10.8		
	Medium dense, brown, silty SAND (SM); moist, fine - **FILL**.	0.2		
	trace brick fragments at 2.5 feet			
5	without brick fragments at 5.0 feet			
	dense, minor brick fragments at 7.5 feet			
10	trace brick fragments at 10.0 feet		DD DS	DD = 117 pcf
	medium dense; wet at 12.5 feet			
15	Very stiff, gray-brown to brown, sandy SILT (ML); moist, sand is fine.	-3.5 / 14.5	DD CON DS PP	DD = 98 pcf PP = 3.5 tsf
			PP	PP = 2.0 tsf
20	Medium dense, gray, silty SAND (SM); wet, fine.	-8.5 / 19.5		
	Hard, brown CLAY (CL); moist.	-10.0 / 21.0		
	Dense, gray SAND with silt (SP-SM); wet, fine to medium.	-10.5 / 21.5		
25	Dense to very dense, gray, silty SAND (SM); wet, fine.	-13.5 / 24.5	DD DS	DD = 95 pcf
30				
35	Hard, gray-brown SILT (ML), trace clay; moist.	-22.5 / 33.5		
40	Very dense, gray SAND with silt (SP-SM); wet, fine to coarse.	-27.5 / 38.5		

GRAPHIC LOG

SAMPLE

▲ BLOW COUNT
● MOISTURE CONTENT %
RQD% CORE REC%



12.5 feet, during drilling

DRILLED BY: SoCal Drilling

LOGGED BY: ENT

COMPLETED: 11/11/16

BORING METHOD: mud rotary (see document text)

BORING BIT DIAMETER: 4 7/8 inches

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BORING B-4

PROPOSED HOTEL DEVELOPMENT
LONG BEACH, CA

FIGURE A-4

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % ▥ RQD% ▨ CORE REC% (0 – 50 – 100)	INSTALLATION AND COMMENTS
40		(continued from previous page)	-29.5 / 40.5				
		Very dense, gray, silty SAND (SM); wet, fine to coarse.				35-50/5"	
45		dense, siltier at 45.0 feet				69	
		fine at 46.0 feet					
50		very dense, gray-brown at 50.0 feet	-39.9 / 50.9			32-50/5"	
		Exploration completed at a depth of 50.9 feet.					
		Groundwater measured at a depth of 12.5 feet BGS during drilling.					
55		**Backfilled with bentonite chips.**					
60							
65							
70							
75							
80							

DRILLED BY: SoCal Drilling LOGGED BY: ENT COMPLETED: 11/11/16

BORING METHOD: mud rotary (see document text) BORING BIT DIAMETER: 4 7/8 inches

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BORING B-4
(continued)

PROPOSED HOTEL DEVELOPMENT
LONG BEACH, CA

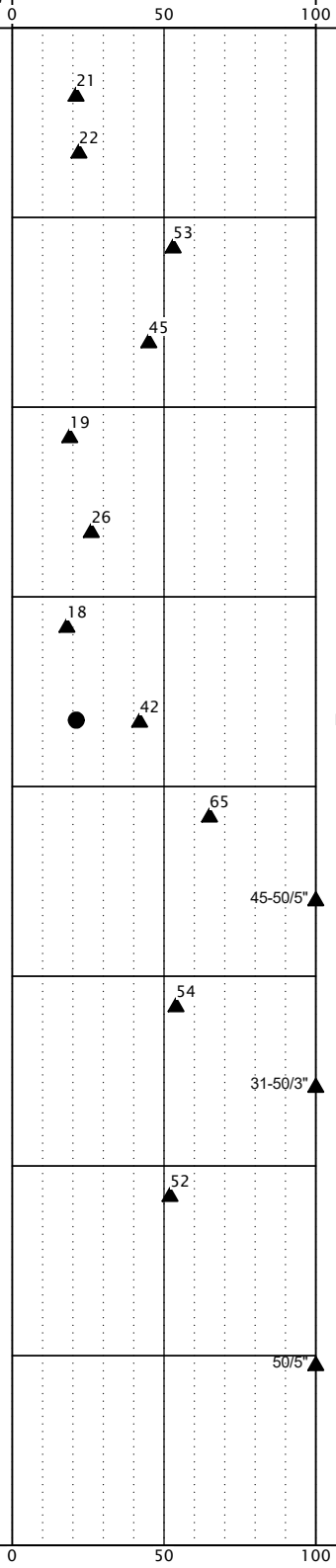
FIGURE A-4

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	INSTALLATION AND COMMENTS
0	ASPHALT CONCRETE (2.3 inches).	9.0 / 0.2		
	Medium dense, brown, silty SAND (SM); moist, fine - **FILL**.	8.8		
	trace gravel and brick fragments; gravel is fine to coarse at 2.5 feet			
5	dense at 5.0 feet			
10	medium dense at 10.0 feet			
	Medium dense, gray SAND with silt (SP-SM), trace shell fragments; wet, fine.	-4.5 / 13.5		
15	Medium dense, brown, silty SAND (SM); wet, fine.	-5.5 / 14.5		
	gray-brown at 17.5 feet		DD CON DS	DD = 108 pcf
20	Very dense, gray SAND with silt (SP-SM); wet, fine.	-10.5 / 19.5		
25	trace shell fragments at 25.0 feet			
	without shell fragments at 27.5 feet			
30	Hard, gray SILT (ML); moist.	-24.0 / 33.0		
35	Very dense, gray-brown, silty SAND (SM); wet, fine to medium.	-26.2 / 35.2		
40				

GRAPHIC LOG

SAMPLE

▲ BLOW COUNT
● MOISTURE CONTENT %
RQD% CORE REC%



11.5 feet, during drilling

DRILLED BY: SoCal Drilling

LOGGED BY: ENT

COMPLETED: 11/10/16

BORING METHOD: mud rotary (see document text)

BORING BIT DIAMETER: 4 7/8 inches

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BORING B-5

PROPOSED HOTEL DEVELOPMENT
LONG BEACH, CA

FIGURE A-5

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	BLOW COUNT / MOISTURE CONTENT % / RQD% / CORE REC% (0–50–100)	INSTALLATION AND COMMENTS
40	brown; fine to coarse at 40.0 feet				36-50/5"	
	Very dense, gray SAND (SP), trace silt; wet, fine to coarse.	-34.0 / 43.0				
45					50/5"	
	Very dense, gray-brown, silty SAND (SM); wet, fine.	-39.0 / 48.0				
50					39-50/4"	
55					45-50/4"	
	Hard, gray-brown SILT (ML), some clay; moist.	-49.0 / 58.0				
60			PP		73	PP = 4.0 tsf
	Very dense, gray-brown, silty SAND with clay (SM); wet, fine.	-52.0 / 61.0				
	Hard, gray-brown SILT (ML); moist.	-54.0 / 63.0				
65			PP		24-50/5"	PP = >4.5 tsf
70	very stiff, dark gray, minor sand; sand is fine at 70.0 feet	-61.5 / 70.5	PP		23-50/4"	PP = 2.0 tsf
	Very dense, dark gray, silty SAND (SM); wet, fine.					
	Very stiff, dark gray, sandy SILT (ML), minor clay; moist, sand is fine.	-64.0 / 73.0				
75			PP		28	PP = 2.5 tsf
	Very dense, gray, silty SAND (SM); wet, fine.	-67.5 / 76.5				
80						

DRILLED BY: SoCal Drilling LOGGED BY: ENT COMPLETED: 11/10/16

BORING METHOD: mud rotary (see document text) BORING BIT DIAMETER: 4 7/8 inches

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BORING B-5
(continued)

PROPOSED HOTEL DEVELOPMENT
LONG BEACH, CA

FIGURE A-5

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (0 – 50 – 100)	INSTALLATION AND COMMENTS
80		(continued from previous page)					
		Very dense, gray SAND with silt (SP-SM); wet, fine.	-74.0 / 83.0				
85						50/5"	
90						50/4"	
		Very dense, gray SAND (SP); wet, fine to medium.	-84.0 / 93.0				
95						50/5"	
		Very dense, dark gray, silty SAND (SM); wet, fine.	-89.0 / 98.0				
100						46-50/4"	
105						25-50/5"	
110						50/5.5"	
115						50/6"	
120							

DRILLED BY: SoCal Drilling LOGGED BY: ENT COMPLETED: 11/10/16

BORING METHOD: mud rotary (see document text) BORING BIT DIAMETER: 4 7/8 inches

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BORING B-5
(continued)

PROPOSED HOTEL DEVELOPMENT
LONG BEACH, CA

FIGURE A-5

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (0 – 50 – 100)	INSTALLATION AND COMMENTS
120		(continued from previous page)				50/4"	
125			-116.4 / 125.4			50/5"	
		Exploration completed at a depth of 125.4 feet.					
		Groundwater measured at a depth of 11.5 feet BGS during drilling.					
130		**Backfilled with bentonite chips.**					
135							
140							
145							
150							
155							
160							

DRILLED BY: SoCal Drilling LOGGED BY: ENT COMPLETED: 11/10/16

BORING METHOD: mud rotary (see document text) BORING BIT DIAMETER: 4 7/8 inches

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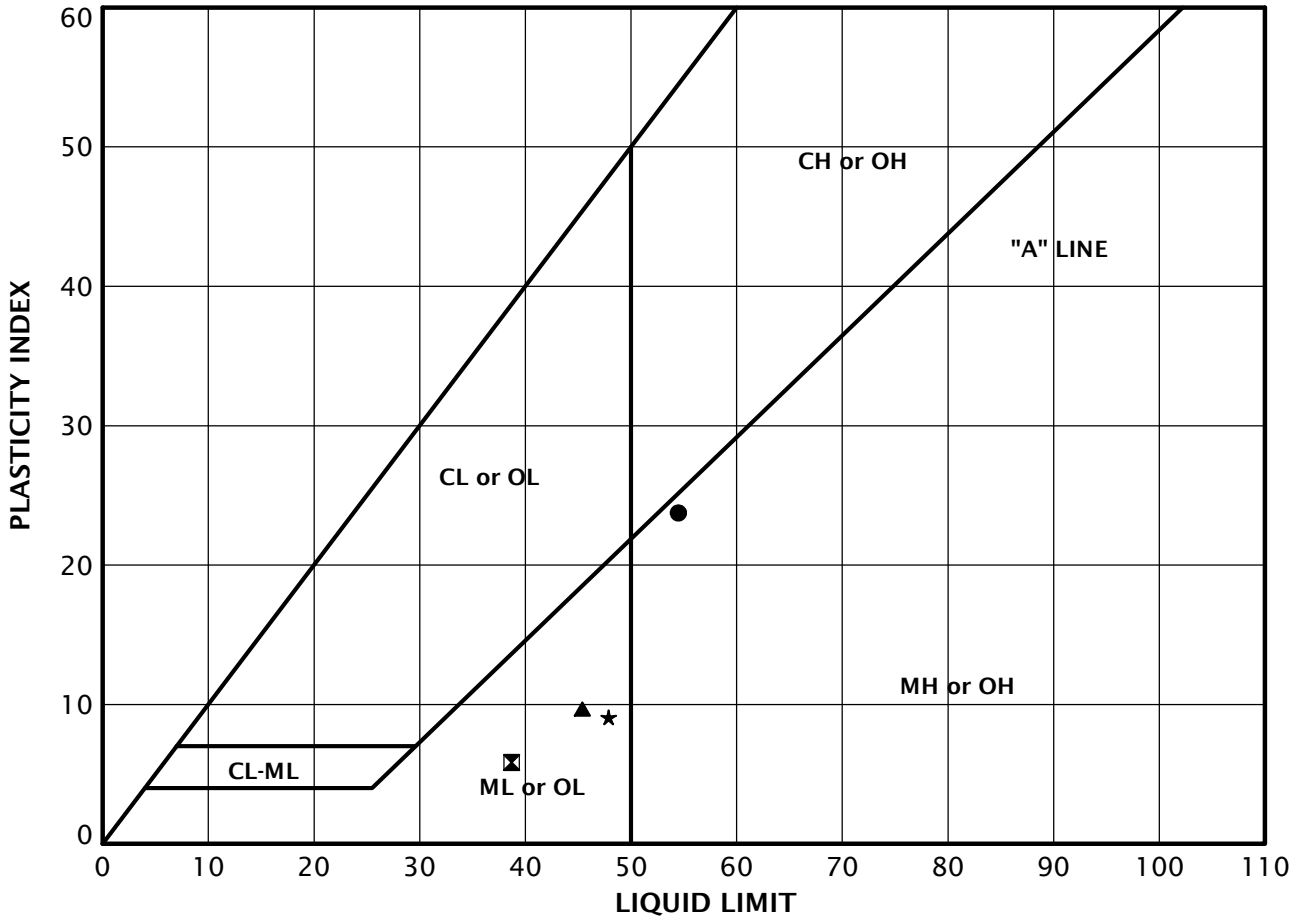
BORING B-5
(continued)

PROPOSED HOTEL DEVELOPMENT
LONG BEACH, CA

FIGURE A-5



KEY	EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	MOISTURE CONTENT (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
●	B-1	30.0		54	31	23
⊠	B-1	50.0	30	39	33	6
▲	B-2	30.0	30	45	36	9
★	B-3	27.5		48	39	9

ATTERBERG_LIMITS 7 SODOBUILD-2-01-B1_5.GPJ GEODESIGN.GDT PRINT DATE: 10/3/18:KT

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Anaheim CA 92806
714.634.3701 www.geodesigninc.com

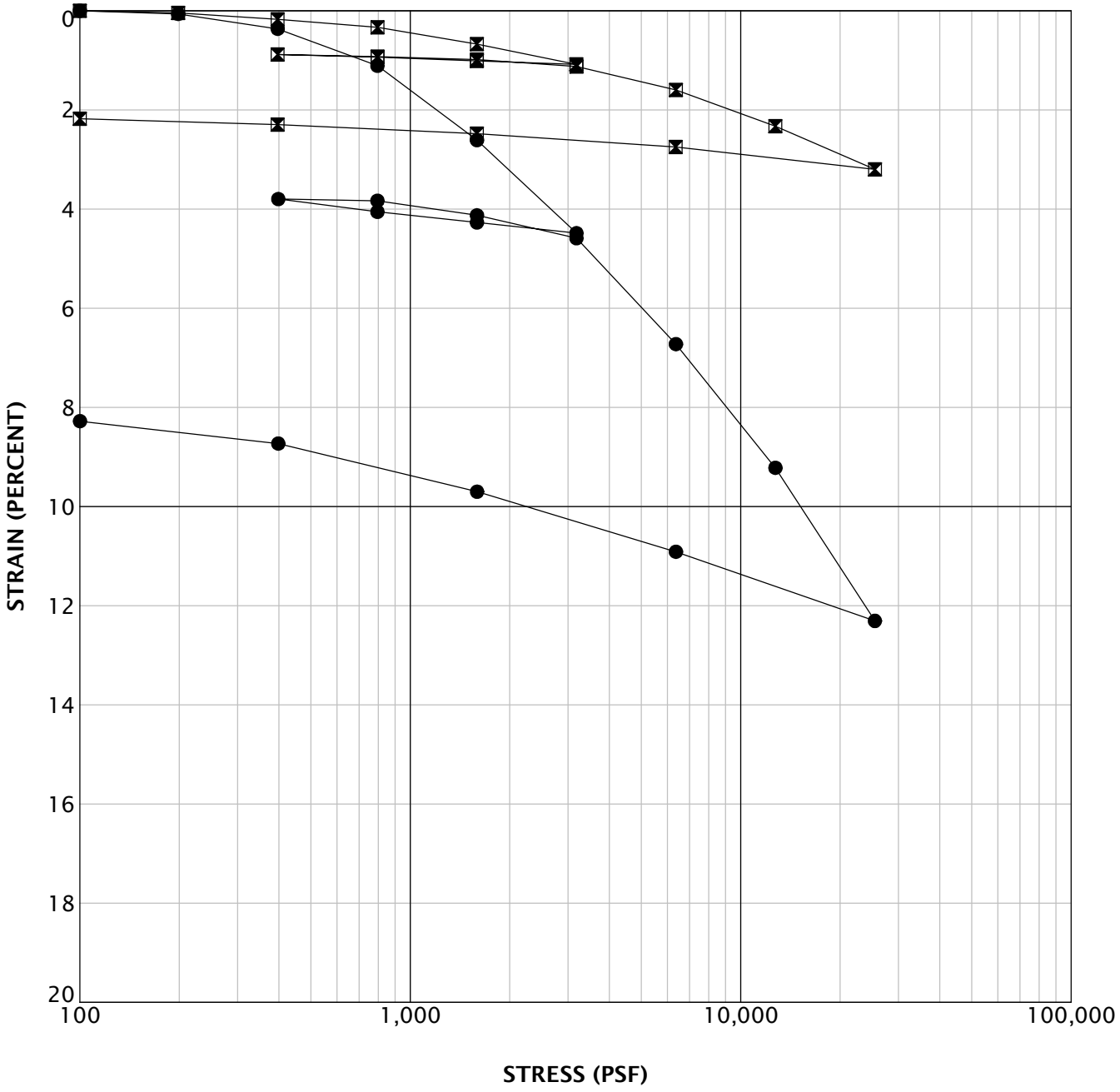
SODOBUILD-2-01

JANUARY 2017

ATTERBERG LIMITS TEST RESULTS

PROPOSED HOTEL DEVELOPMENT
LONG BEACH, CA

FIGURE A-6



KEY	EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)
●	B-4	15.0	25	98
⊠	B-5	17.5	21	108

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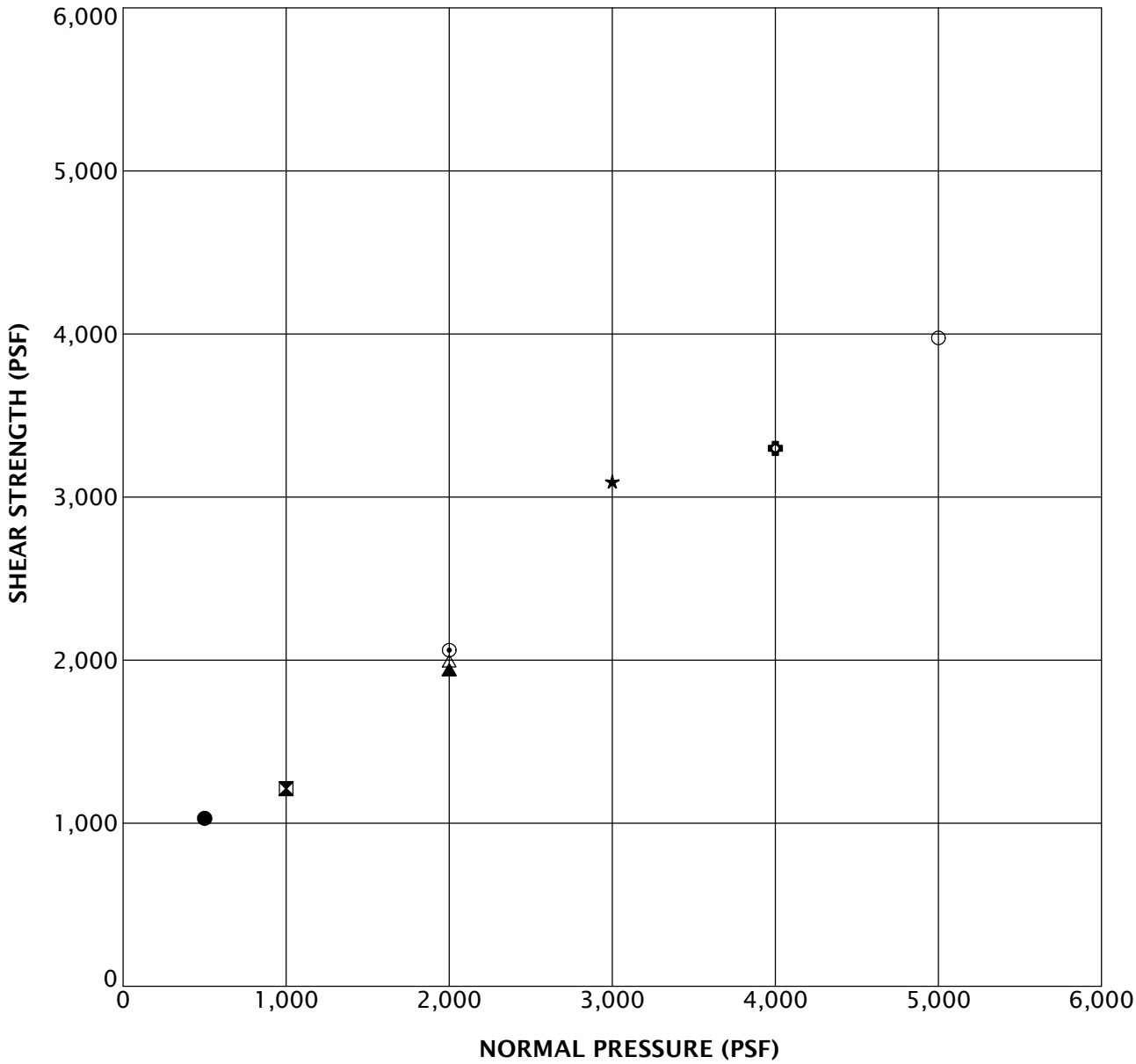
CONSOLIDATION TEST RESULTS

PROPOSED HOTEL DEVELOPMENT
LONG BEACH, CA

FIGURE A-7

CONSOL_STRAIN_100K SODOBUILD-2-01-B1_5.GPJ GEODESIGN.GDT PRINT DATE: 10/3/18:KT



KEY	EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)	SOAKED
●	B-1	2.5	10	119	YES
⊠	B-3	5.0	11	121	YES
▲	B-3	15.0	21	106	YES
★	B-3	20.0	23	99	YES
⊙	B-4	10.0	14	117	YES
✿	B-4	15.0	25	99	YES
○	B-4	25.0	27	95	YES
△	B-5	17.5	21	105	YES

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SODOBUILD-2-01

DIRECT SHEAR TEST RESULTS

JANUARY 2017

PROPOSED HOTEL DEVELOPMENT
LONG BEACH, CA

FIGURE A-8

DIRECT_SHEAR_FAIL_ENV_NO BOX SODOBUILD-2-01-B1_5.GPJ GEODESIGN.GDT PRINT DATE: 10/3/18:KT

SAMPLE INFORMATION			MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)	SIEVE			ATTERBERG LIMITS		
EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	ELEVATION (FEET)			GRAVEL (PERCENT)	SAND (PERCENT)	P200 (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
B-1	1.0	5.0	10	126						
B-1	2.5	3.5	10	122						
B-1	5.0	1.0	16	97						
B-1	7.5	-1.5	22	100						
B-1	12.5	-6.5	24				12			
B-1	17.5	-11.5	24				17			
B-1	30.0	-24.0						54	31	23
B-1	50.0	-44.0	30					39	33	6
B-2	2.5	4.5	12	111						
B-2	4.0	3.0	5	102						
B-2	6.0	1.0	22	101						
B-2	30.0	-23.0	30					45	36	9
B-3	5.0	3.0	11	121						
B-3	15.0	-7.0	21	106						
B-3	20.0	-12.0	23	99						
B-3	27.5	-19.5						48	39	9
B-4	10.0	1.0	14	117						
B-4	15.0	-4.0	25	98						
B-4	25.0	-14.0	27	95						
B-5	17.5	-8.5	21	108						

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SUMMARY OF LABORATORY DATA

PROPOSED HOTEL DEVELOPMENT
LONG BEACH, CA

FIGURE A-9

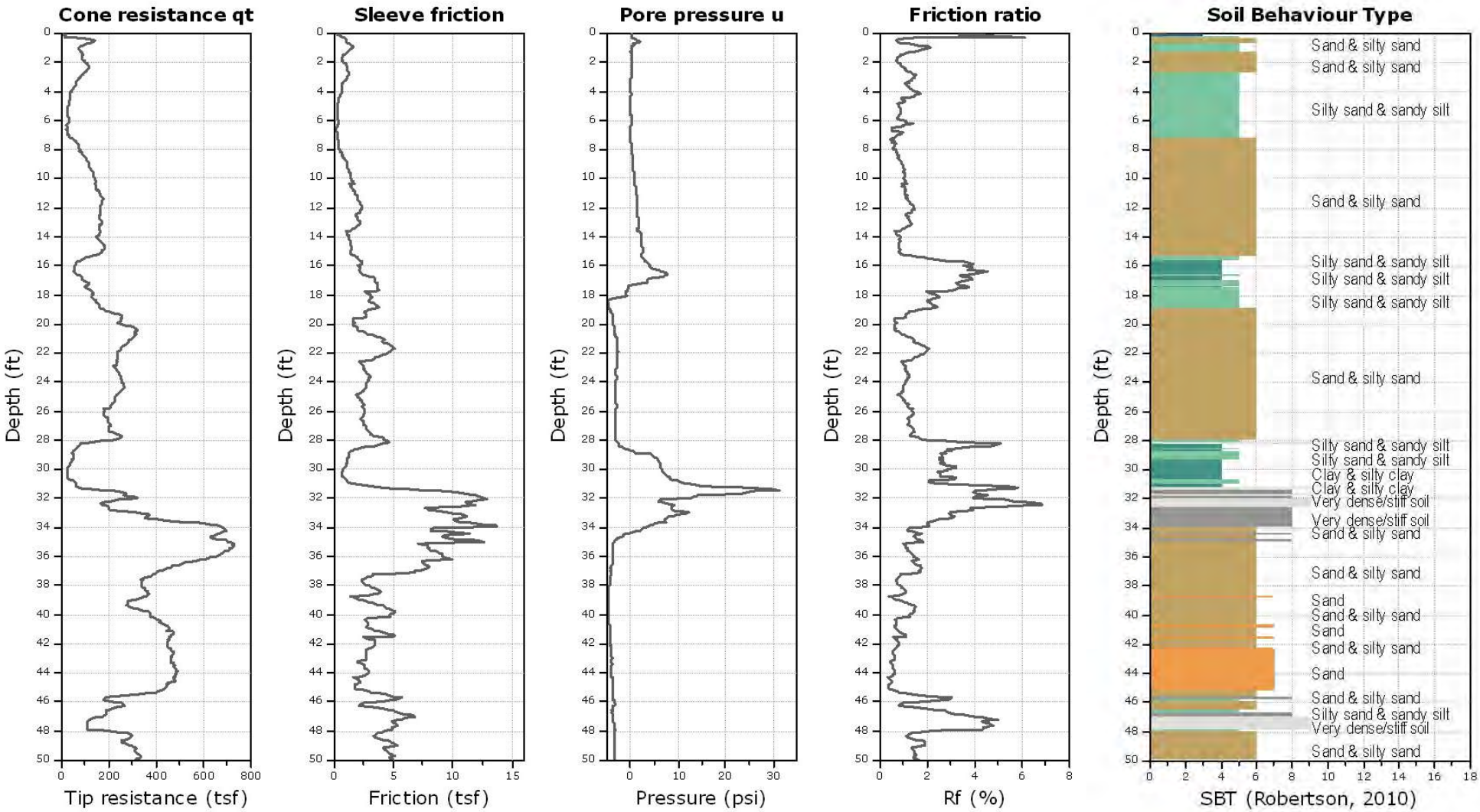
APPENDIX B

Kehoe Testing and Engineering
714-901-7270
rich@kehoetesting.com
www.kehoetesting.com

CPT-1

Project: **GeoDesign, Inc.**
Location: **100 E. Ocean Blvd Long Beach, CA**

Total depth: 50.13 ft, Date: 11/18/2016
Cone Type: Vertek



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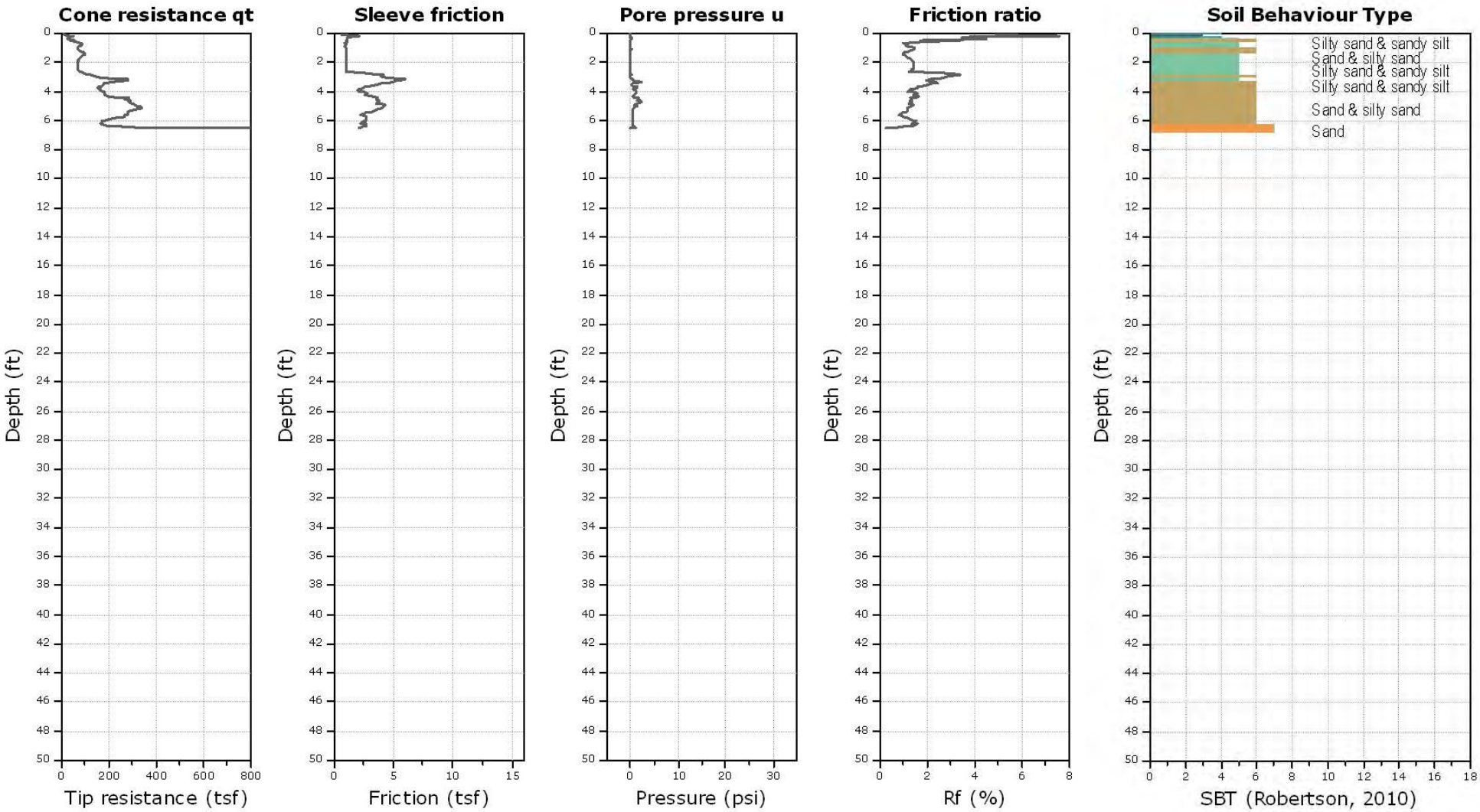
www.kehoetesting.com

CPT-2

Project: **GeoDesign, Inc.**
Location: **100 E. Ocean Blvd Long Beach, CA**

Total depth: 6.54 ft, Date: 11/18/2016
Cone Type: Vertek



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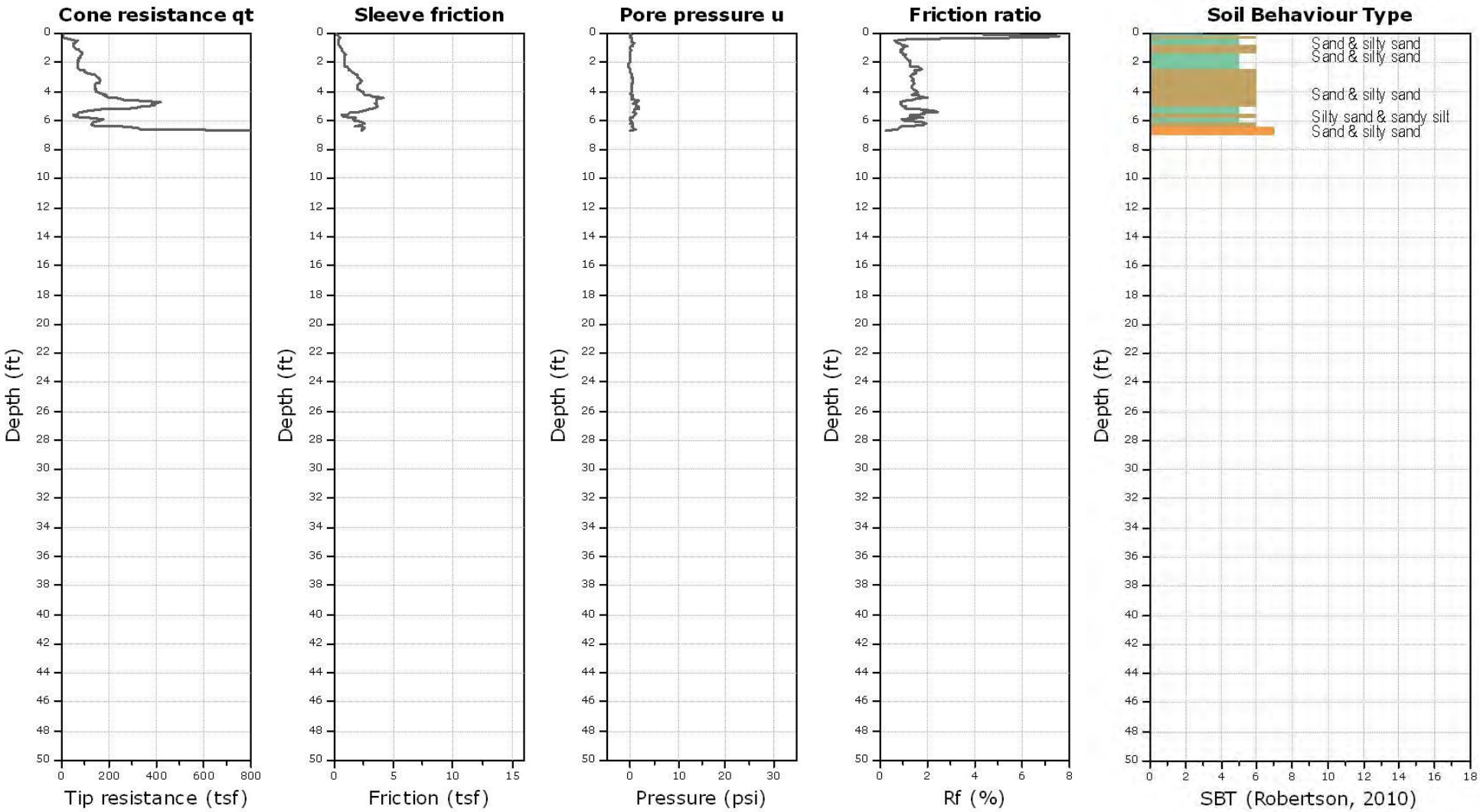
rich@kehoetesting.com
www.kehoetesting.com

CPT-2A

Project: **GeoDesign, Inc.**
Location: **100 E. Ocean Blvd Long Beach, CA**

Total depth: 6.68 ft, Date: 11/18/2016
Cone Type: Vertek



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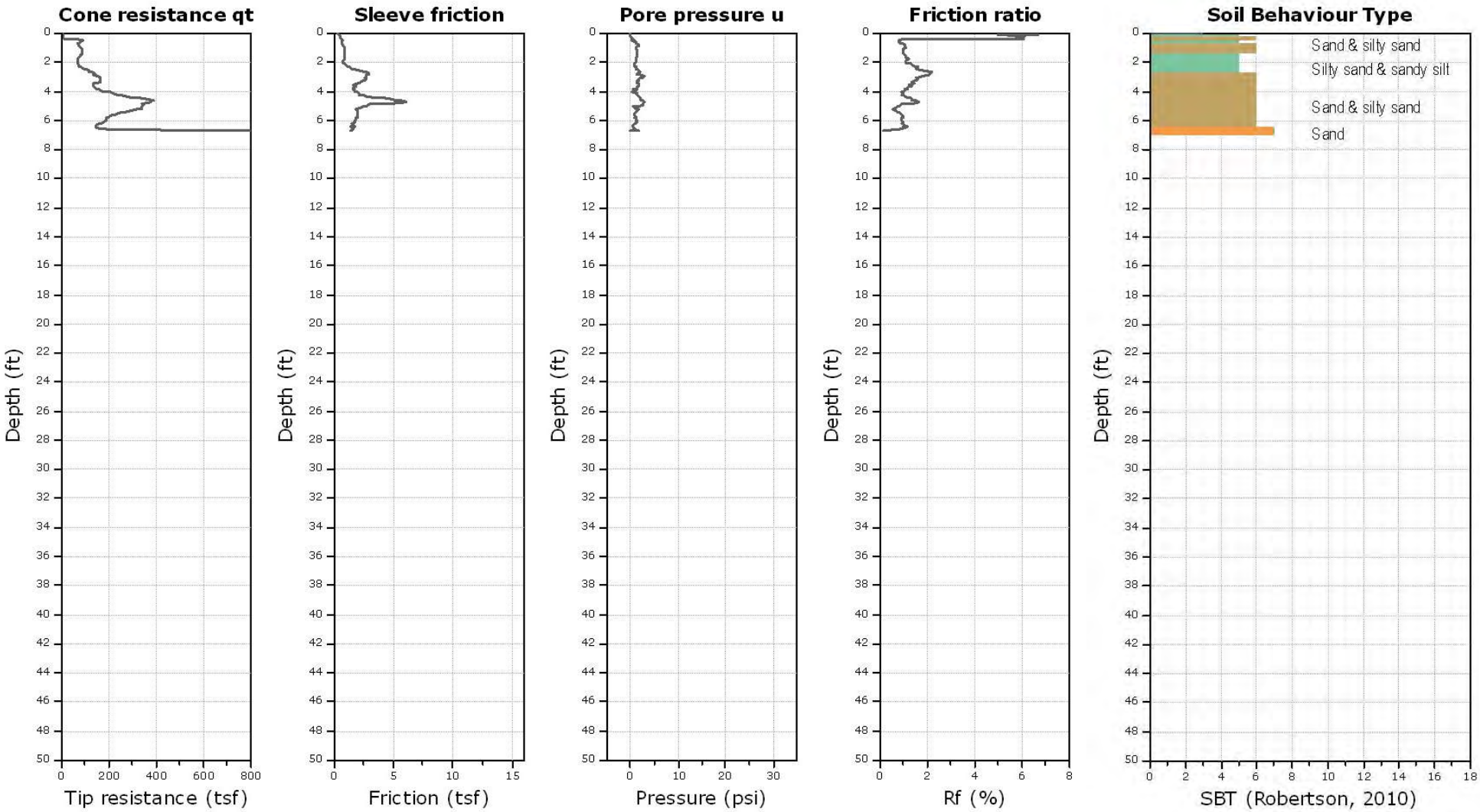
www.kehoetesting.com

CPT-2B

Project: **GeoDesign, Inc.**
Location: **100 E. Ocean Blvd Long Beach, CA**

Total depth: 6.73 ft, Date: 11/18/2016
Cone Type: Vertek



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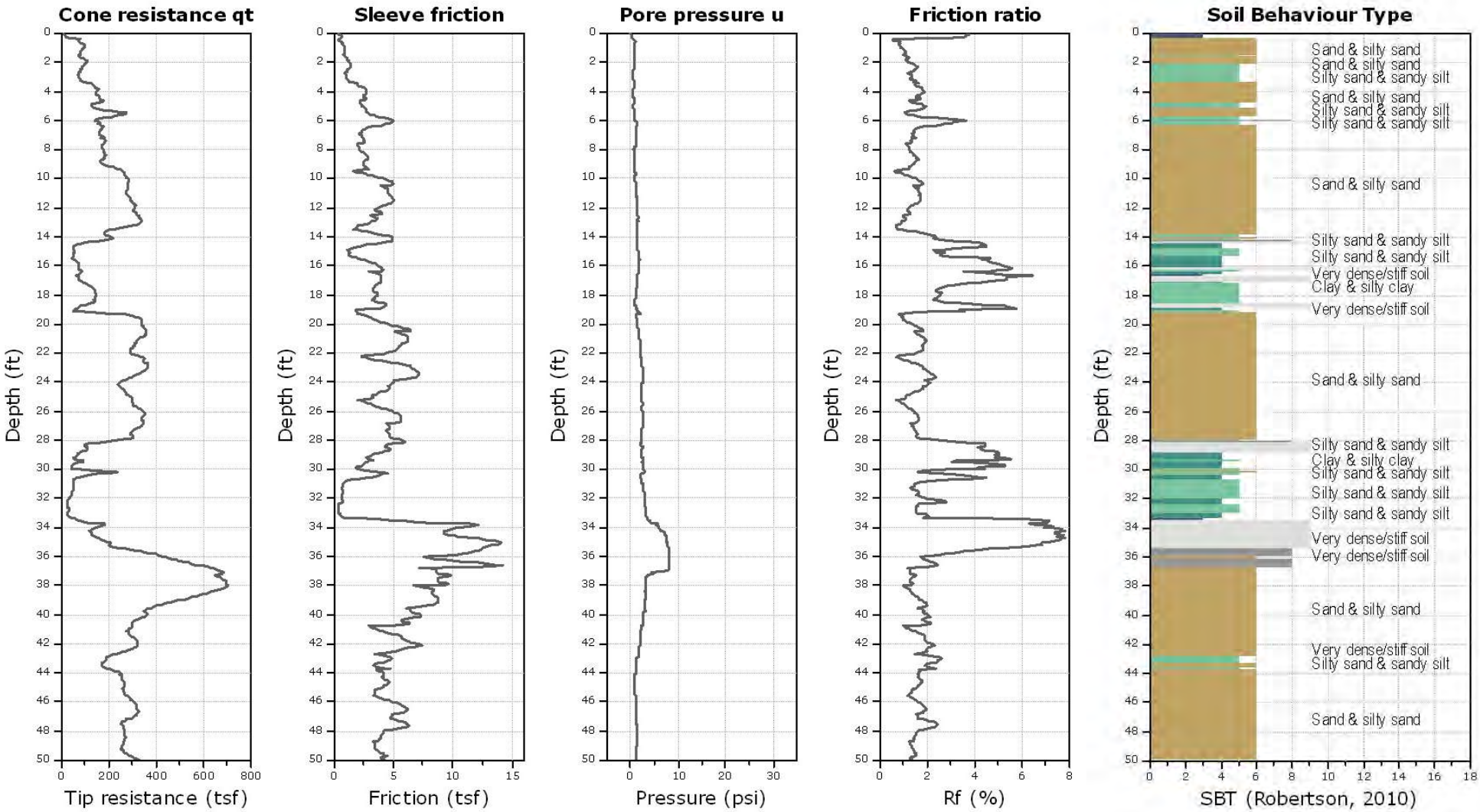
rich@kehoetesting.com
www.kehoetesting.com

CPT-3

Project: **GeoDesign, Inc.**
Location: **100 E. Ocean Blvd Long Beach, CA**

Total depth: 50.10 ft, Date: 11/18/2016
Cone Type: Vertek

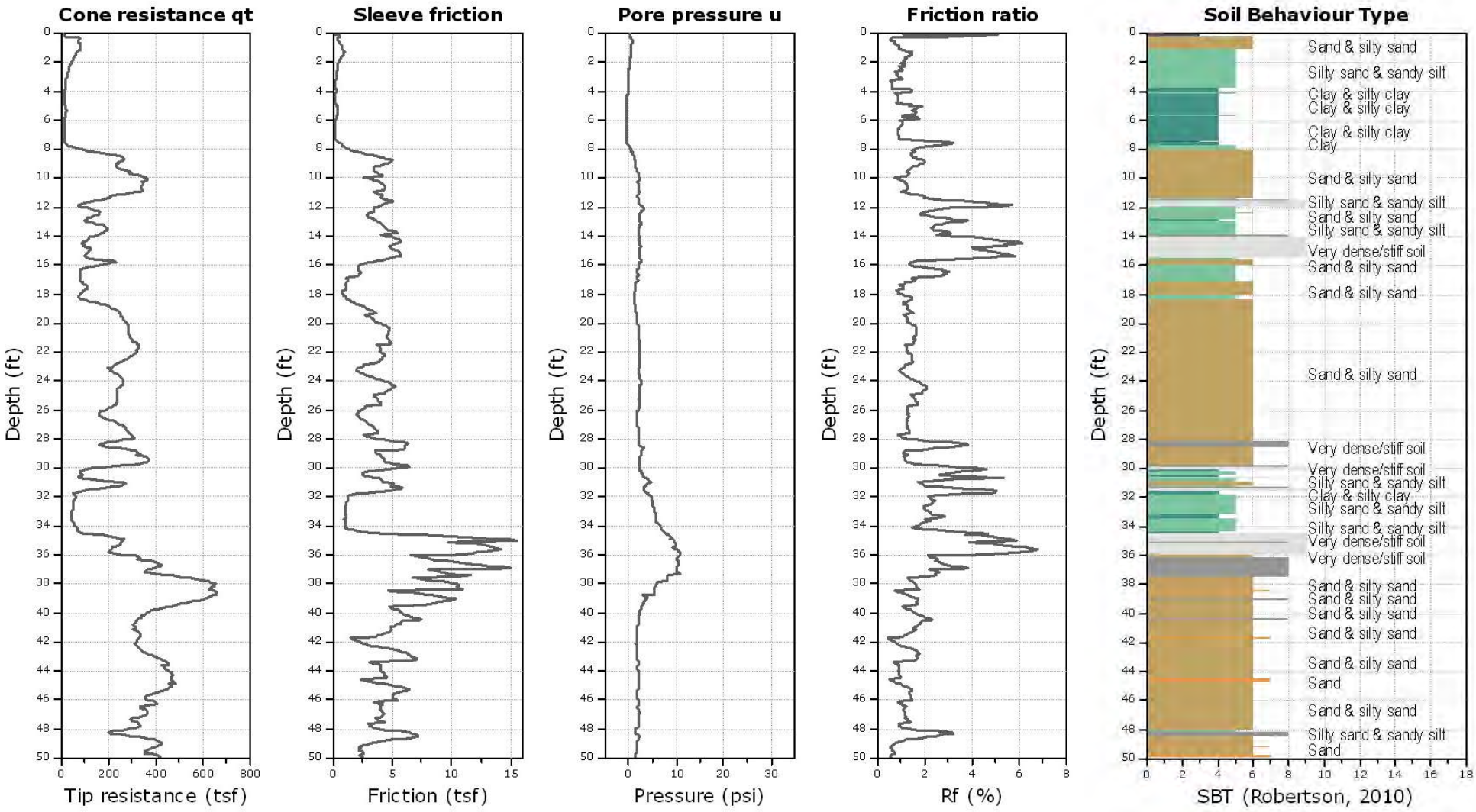


Kehoe Testing and Engineering
714-901-7270
rich@kehoetesting.com
www.kehoetesting.com

CPT-4

Project: **GeoDesign, Inc.**
Location: **100 E. Ocean Blvd Long Beach, CA**

Total depth: 50.10 ft, Date: 11/18/2016
Cone Type: Vertek



APPENDIX C

100 E OCEAN AVENUE

SUSPENSION PS VELOCITIES

BOREHOLE B-5

LONG BEACH, CA

November 23, 2016

Report 16415-01 rev 0

100 E OCEAN AVENUE

SUSPENSION PS VELOCITIES

BOREHOLE B-5

LONG BEACH, CA

Prepared for

GeoDesign, Inc.
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Project 16415

November 23, 2016

Report 16415-01 rev 0

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APPENDICES

APPENDIX A **SUSPENSION VELOCITY MEASUREMENT QUALITY ASSURANCE SUSPENSION SOURCE TO RECEIVER ANALYSIS RESULTS**

APPENDIX B **GEOPHYSICAL LOGGING SYSTEMS - NIST TRACEABLE CALIBRATION RECORDS**

INTRODUCTION

GEO*Vision* acquired borehole geophysical data in one borehole in Long Beach, CA. The work was performed for GeoDesign, Inc. Fieldwork was performed by Jonathan Jordan. Analysis was completed by John Diehl, and report was completed by Emily Feldman, and reviewed by John Diehl, Professional Engineer.

SCOPE OF WORK

This report presents results of Suspension PS velocity data acquired in one borehole on November 10, 2016, as detailed in Table 1. The purpose of these measurements was to supplement stratigraphic information by acquiring shear wave and compressional wave velocities as a function of depth.

The OYO Suspension PS Logging System (Suspension System) was used to obtain in-situ horizontal shear (S_H) and compressional (P) wave velocity measurements in one uncased borehole at 1.6 foot intervals. Measurements followed **GEO***Vision* Procedure for P-S Suspension Seismic Velocity Logging, revision 1.5. Acquired data were analyzed and a profile of velocity versus depth was produced for both S_H and P waves.

A detailed reference for the suspension PS velocity measurement techniques used in this study is:

> Guidelines for Determining Design Basis Ground Motions, Report TR-102293, Electric Power Research Institute, Palo Alto, California, November 1993, Sections 7 and 8.

INSTRUMENTATION

Suspension Velocity Instrumentation

Suspension velocity measurements were performed using the suspension PS logging system, manufactured by OYO Corporation, and their subsidiary, Robertson Geologging. This system directly determines the average velocity of a 3.3-foot high segment of the soil column surrounding the borehole of interest by measuring the elapsed time between arrivals of a wave propagating upward through the soil column. The receivers that detect the wave, and the source that generates the wave, are moved as a unit in the borehole producing relatively constant amplitude signals at all depths.

The suspension system probe consists of a combined reversible polarity solenoid horizontal shear-wave source (S_H) and compressional-wave source (P), joined to two biaxial receivers by a flexible isolation cylinder, as shown in Figure 1. The separation of the two receivers is 3.3 feet, allowing average wave velocity in the region between the receivers to be determined by inversion of the wave travel time between the two receivers. The total length of the probe as used in these surveys is approximately 25 feet, with the center point of the receiver pair 12.5 feet above the bottom end of the probe.

The probe receives control signals from, and sends the digitized receiver signals to, instrumentation on the surface via an armored multi-conductor cable. The cable is wound onto the drum of a winch and is used to support the probe. Cable travel is measured to provide probe depth data using a sheave of known circumference fitted with a digital rotary encoder.

The entire probe is suspended in the borehole by the cable, therefore, source motion is not coupled directly to the borehole walls; rather, the source motion creates a horizontally propagating impulsive pressure wave in the fluid filling the borehole and surrounding the source. This pressure wave is converted to P and S_H-waves in the surrounding soil and rock as it passes through the casing and grout annulus and impinges upon the wall of the borehole. These waves propagate

through the soil and rock surrounding the borehole, in turn causing a pressure wave to be generated in the fluid surrounding the receivers as the soil waves pass their location. Separation of the P and S_H-waves at the receivers is performed using the following steps:

1. Orientation of the horizontal receivers is maintained parallel to the axis of the source, maximizing the amplitude of the recorded S_H -wave signals.
2. At each depth, S_H-wave signals are recorded with the source actuated in opposite directions, producing S_H-wave signals of opposite polarity, providing a characteristic S_H-wave signature distinct from the P-wave signal.
3. The 6.3 foot separation of source and receiver 1 permits the P-wave signal to pass and damp significantly before the slower S_H-wave signal arrives at the receiver. In faster soils or rock, the isolation cylinder is extended to allow greater separation of the P- and S_H-wave signals.
4. In saturated soils, the received P-wave signal is typically of much higher frequency than the received S_H-wave signal, permitting additional separation of the two signals by low pass filtering.
5. Direct arrival of the original pressure pulse in the fluid is not detected at the receivers because the wavelength of the pressure pulse in fluid is significantly greater than the dimension of the fluid annulus surrounding the probe (feet versus inches scale), preventing significant energy transmission through the fluid medium.

In operation, a distinct, repeatable pattern of impulses is generated at each depth as follows:

1. The source is fired in one direction producing dominantly horizontal shear with some vertical compression, and the signals from the horizontal receivers situated parallel to the axis of motion of the source are recorded.
2. The source is fired again in the opposite direction and the horizontal receiver signals are recorded.

3. The source is fired again and the vertical receiver signals are recorded. The repeated source pattern facilitates the picking of the P and S_H-wave arrivals; reversal of the source changes the polarity of the S_H-wave pattern but not the P-wave pattern.

The data from each receiver during each source activation is recorded as a different channel on the recording system. The Suspension PS system has six channels (two simultaneous recording channels), each with a 1024 sample record. The recorded data are displayed as six channels with a common time scale. Data are stored on disk for further processing.

Review of the displayed data on the recorder or computer screen allows the operator to set the gains, filters, delay time, pulse length (energy), and sample rate to optimize the quality of the data before recording. Verification of the calibration of the Suspension PS digital recorder is performed every twelve months using a NIST traceable frequency source and counter, as presented in Appendix B.

MEASUREMENT PROCEDURES

Suspension Velocity Measurement Procedures

The borehole was logged uncased and filled with fresh water mud. Measurements followed the **GEO***Vision* Procedure for P-S Suspension Seismic Velocity Logging, revision 1.5. Prior to the logging run, the probe was positioned with the top of the probe even with a stationary reference point. The electronic depth counter was set to the distance between the mid-point of the receiver and the top of the probe, minus the height of the stationary reference point, if any. Measurements were verified with a tape measure, and calculations recorded on a field log.

The probe was lowered to the bottom of the borehole, stopping at 1.6 foot intervals to collect data, as summarized in Table 2. At each measurement depth the measurement sequence of two opposite horizontal records and one vertical record was performed. Gains were adjusted as required. The data from each depth were viewed on the computer display, checked, and saved to disk before moving to the next depth.

Upon completion of the measurements, the probe was returned to the surface and the zero depth indication at the depth reference point was verified prior to removal from the borehole.

DATA ANALYSIS

Suspension Velocity Analysis

Using the proprietary OYO program PSLOG.EXE version 1.0, the recorded digital waveforms were analyzed to locate the most prominent first minima, first maxima, or first break on the vertical axis records, indicating the arrival of P-wave energy. The difference in travel time between receiver 1 and receiver 2 (R1-R2) arrivals was used to calculate the P-wave velocity for that 1.0 meter segment of the soil column. When observable, P-wave arrivals on the horizontal axis records were used to verify the velocities determined from the vertical axis data. The time picks were then transferred into a Microsoft Excel® template to complete the velocity calculations based on the arrival time picks made in PSLOG. The Microsoft Excel® analysis files accompany this report.

The P-wave velocity over the 6.3-foot interval from source to receiver 1 (S-R1) was also picked using PSLOG, and calculated and plotted in Microsoft Excel®, for quality assurance of the velocity derived from the travel time between receivers. In this analysis, the depth values as recorded were increased by 4.8 feet to correspond to the mid-point of the 6.33-foot S-R1 interval. Travel times were obtained by picking the first break of the P-wave signal at receiver 1 and subtracting the calculated and experimentally verified delay, in milliseconds, from source trigger pulse (beginning of record) to source impact. This delay corresponds to the duration of acceleration of the solenoid before impact.

As with the P-wave records, the recorded digital waveforms were analyzed to locate clear S_H-wave pulses, as indicated by the presence of opposite polarity pulses on each pair of horizontal records. Ideally, the S_H-wave signals from the 'normal' and 'reverse' source pulses are very nearly inverted images of each other. Digital Fast Fourier Transform – Inverse Fast Fourier Transform (FFT – IFFT) lowpass filtering was used to remove the higher frequency P-wave signal from the S_H-wave signal. Different filter cutoffs were used to separate P- and S_H-waves at different depths, ranging from 600 Hz in the slowest zones to 4000 Hz in the regions of highest velocity. At each depth, the filter frequency was selected to be at least twice the fundamental frequency of the S_H-wave signal being filtered.

Generally, the first maxima were picked for the 'normal' signals and the first minima for the 'reverse' signals, although other points on the waveform were used if the first pulse was distorted. The absolute arrival time of the 'normal' and 'reverse' signals may vary by +/- 0.2 milliseconds, due to differences in the actuation time of the solenoid source caused by constant mechanical bias in the source or by borehole inclination. This variation does not affect the R1-R2 velocity determinations, as the differential time is measured between arrivals of waves created by the same source actuation. The final velocity value is the average of the values obtained from the 'normal' and 'reverse' source actuations.

As with the P-wave data, S_H-wave velocity calculated from the travel time over the 6.33-foot interval from source to receiver 1 was calculated and plotted for verification of the velocity derived from the travel time between receivers. In this analysis, the depth values were increased by 4.8 feet to correspond to the mid-point of the 6.33-foot S-R1 interval. Travel times were obtained by picking the first break of the S_H-wave signal at the near receiver and subtracting the calculated and experimentally verified delay, in milliseconds, from the beginning of the record at the source trigger pulse to source impact.

Poisson's Ratio, **ν**, was calculated in the Microsoft Excel® template using the following formula:

$$\boldsymbol{\nu} = \frac{\left(\frac{v_s}{v_p}\right)^2 - 0.5}{\left(\frac{v_s}{v_p}\right)^2 - 1.0}$$

Data and analyses were reviewed by a **GEO***Vision* Professional Geophysicist or Engineer as a component of the in-house data validation program.

Figure 2 shows an example of R1 - R2 measurements on a sample filtered suspension record. In Figure 2, the time difference over the 3.3 foot interval of 1.88 milliseconds for the horizontal signals is equivalent to an S_H-wave velocity of 1745 feet/second. Whenever possible, time differences were determined from several phase points on the S_H-waveform records to verify the data obtained from the first arrival of the S_H-wave pulse. Figure 3 displays the same record before filtering of the S_H-waveform record with a 1400 Hz FFT - IFFT digital lowpass filter, illustrating the presence of higher frequency P-wave energy at the beginning of the record, and distortion of the lower frequency S_H-wave by residual P-wave signal.

RESULTS

Suspension Velocity Results

Suspension R1-R2 P- and S_H-wave velocities for borehole B-5 are plotted in Figure 4, respectively. Suspension velocity data are also presented in Table 3. The Microsoft Excel® analysis file accompanies this report.

P- and S_H-wave velocity data from R1-R2 analysis and quality assurance analysis of S-R1 data are plotted together in Figure A-1 in Appendix A to aid in visual comparison. It should be noted that R1-R2 data are an average velocity over a 3.3-foot segment of the soil column; S-R1 data are an average over 6.3 feet, creating a significant smoothing relative to the R1-R2 plots. The S-R1 velocity data are also presented in Table A-1 and included in the Microsoft Excel® analysis file, which also includes Poisson's Ratio calculations, tabulated data and plots.

SUMMARY

Discussion of Suspension Velocity Results

Suspension PS velocity data are ideally collected in uncased fluid filled boreholes drilled with rotary wash methods, as was the borehole for this project. Overall, Suspension PS velocity data quality is judged on 5 criteria, as summarized below.

	Criteria	B-5
1	Consistent data between receiver to receiver (R1 – R2) and source to receiver (S – R1) data.	Yes.
2	Consistency between data from adjacent depth intervals.	Yes
3	Consistent relationship between P-wave and S_H -wave (excluding transition to saturated soils)	Yes Saturation occurs at about 16ft BGS
4	Clarity of P-wave and S_H-wave onset, as well as damping of later oscillations.	This is excellent data
5	Consistency of profile between adjacent borings, if available.	Not applicable

Quality Assurance

These borehole geophysical measurements were performed using industry-standard or better methods for measurements and analyses. All work was performed under **GEO***Vision* quality assurance procedures, which include:

- Use of NIST-traceable calibrations, where applicable, for field and laboratory instrumentation
- Use of standard field data logs
- Use of independent verification of velocity data by comparison of receiver-to-receiver and source-to-receiver velocities
- Independent review of calculations and results by a registered professional engineer, geologist, or geophysicist.

Suspension Velocity Data Reliability

P- and S_H-wave velocity measurement using the Suspension Method gives average velocities over a 3.3-foot interval of depth. This high resolution results in the scatter of values shown in the graphs. Individual measurements are very reliable with estimated precision of +/- 5%. Depth indications are very reliable with estimated precision of +/- 0.2 feet. Standardized field procedures and quality assurance checks contribute to the reliability of these data.

CERTIFICATION

All geophysical data, analysis, interpretations, conclusions, and recommendations in this document have been prepared under the supervision of and reviewed by a **GEO***Vision* California Professional Geophysicist.

Prepared by

11/23/2016

Emily Feldman — Date
Senior Staff Geophysicist
GEO*Vision* Geophysical Services

Reviewed and approved by

11/23/16

John G. Diehl — Date
California Professional Engineer 30362
GEO*Vision* Geophysical Services

∗ This geophysical investigation was conducted under the supervision of a California Professional Geophysicist using industry standard methods and equipment. A high degree of professionalism was maintained during all aspects of the project from the field investigation and data acquisition, through data processing, interpretation and reporting. All original field data files, field notes and observations, and other pertinent information are maintained in the project files and are available for the client to review for a period of at least one year.

A professional geophysicist's certification of interpreted geophysical conditions comprises a declaration of his/her professional judgment. It does not constitute a warranty or guarantee, expressed or implied, nor does it relieve any other party of its responsibility to abide by contract documents, applicable codes, standards, regulations or ordinances.

Table 1. Borehole locations and logging dates

BOREHOLE	DATES	COORDINATES (DEGREES) [1]		ELEVATION (TOP OF WELL CASING) [1]
DESIGNATION	LOGGED	LATITUDE	LONGITUDE	(FEET)
B-5	11/10/2016	33.766375 N	118.1920137 W	-

[1] Survey location approximated using cellular GPS

Table 2. Logging dates and depth ranges

BOREHOLE NUMBER	TOOL AND RUN NUMBER	DEPTH RANGE (feet)	OPEN HOLE (FEET)	SAMPLE INTERVAL (FEET)
B-5	SUSPENSION DOWN 01	1.64 – 111.5	125	1.6

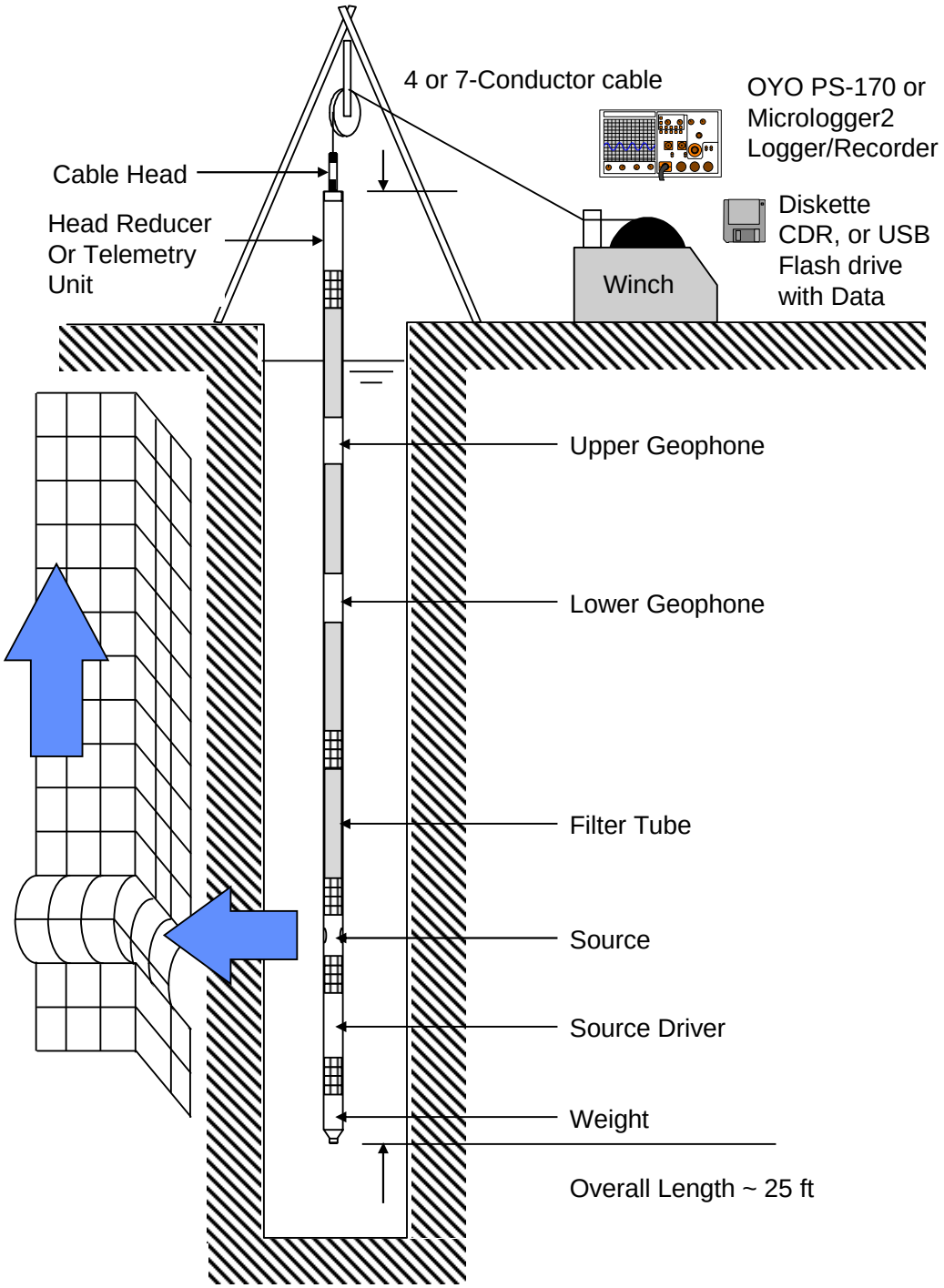

Figure 1: Concept illustration of P-S logging system

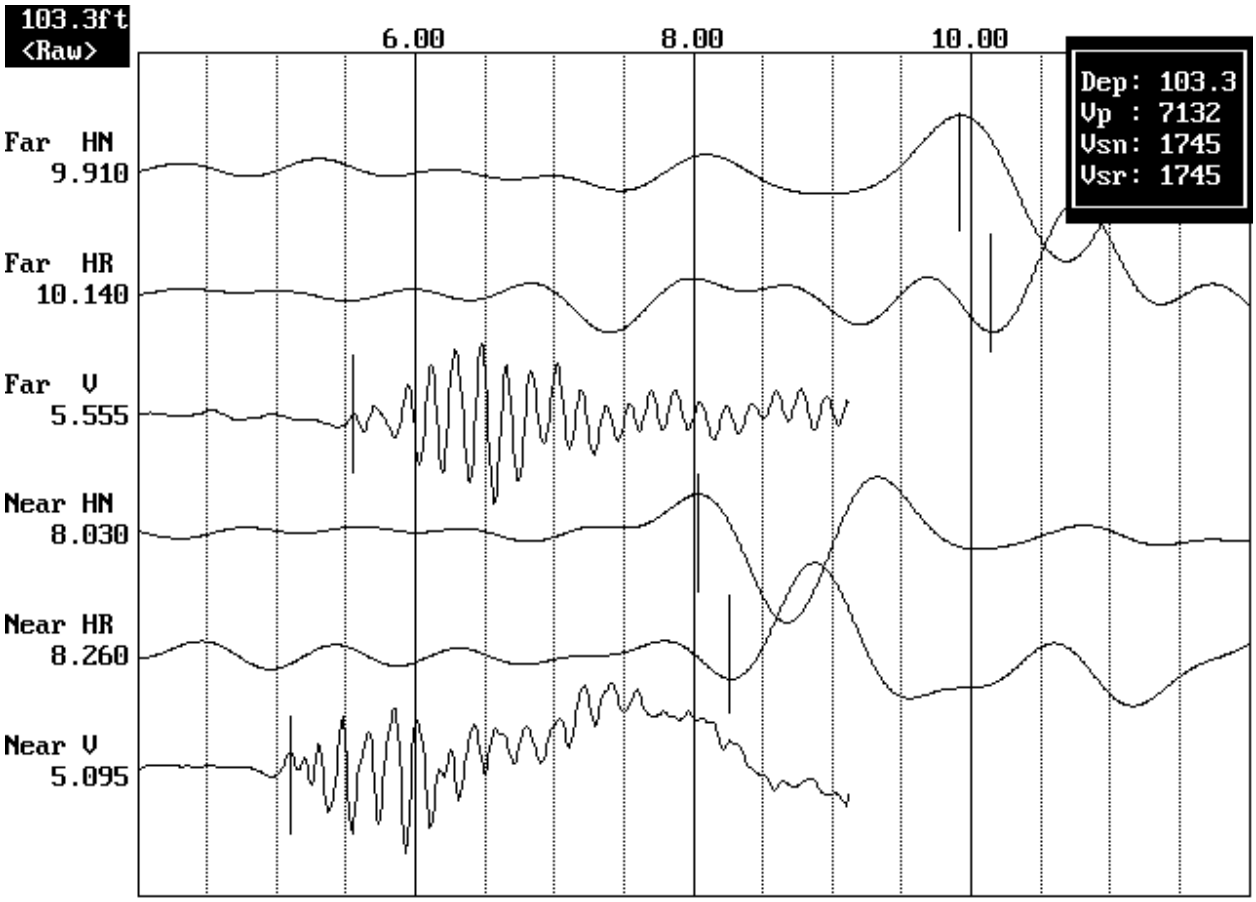

Figure 2: Example of filtered (1400 Hz lowpass) suspension record

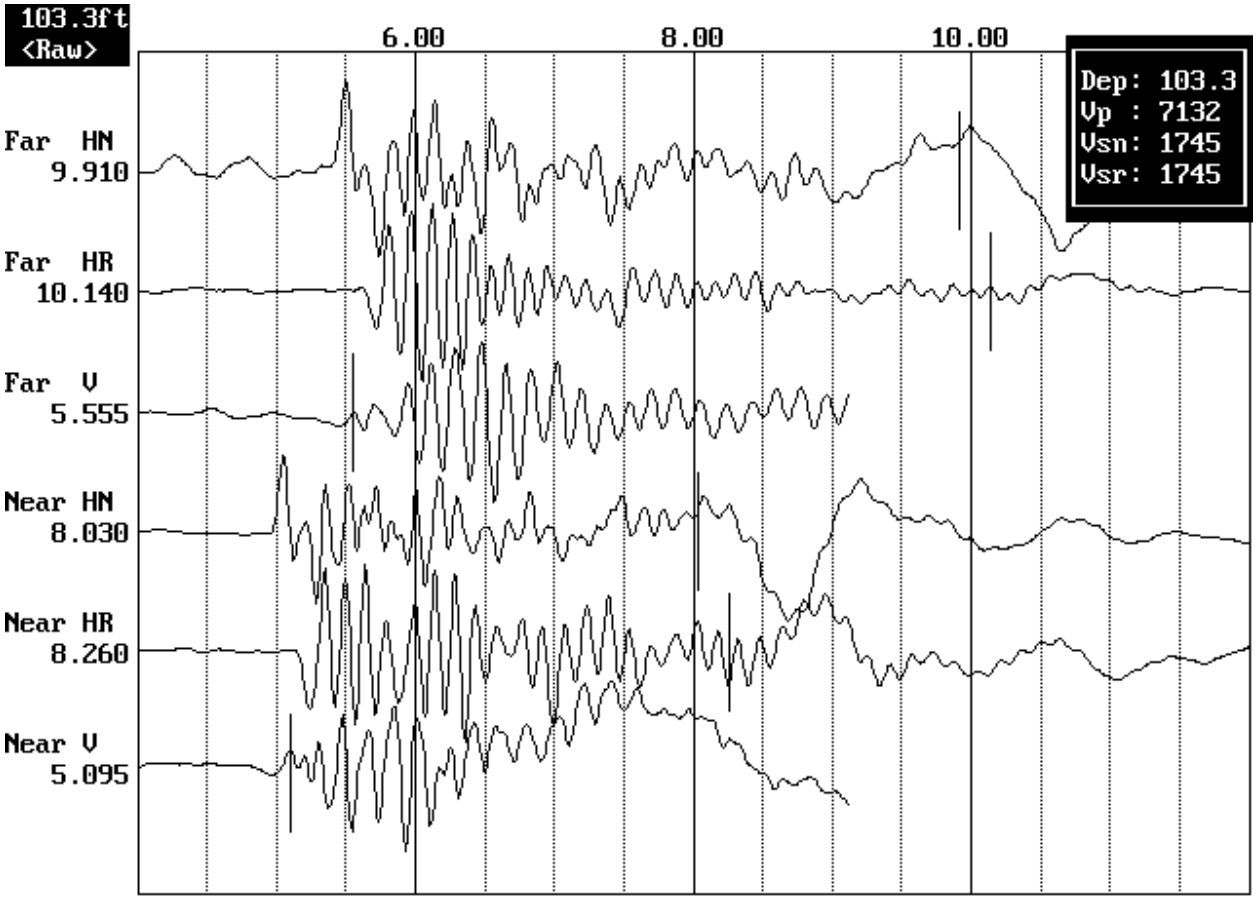

Figure 3. Example of unfiltered suspension record

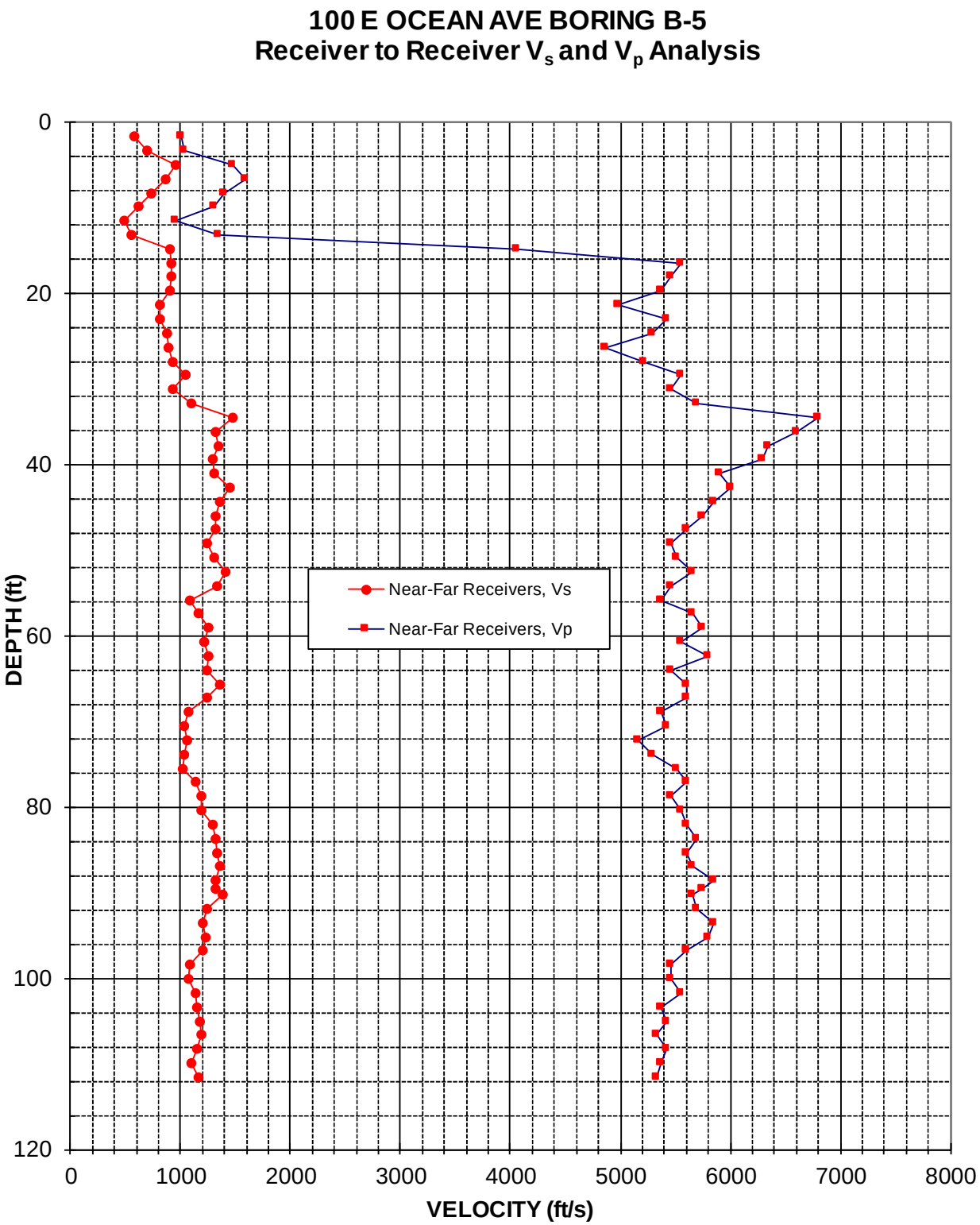

Figure 4: Borehole B-5, Suspension R1-R2 P- and S_H-wave velocities

Table 3. Borehole B-5, Suspension R1-R2 depths and P- and S_H-wave velocities

Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio Based on Receiver-to-Receiver Travel Time Data - Borehole B-5

American Units				Metric Units			
Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio	Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio
	V_s	V_p			V_s	V_p	
(ft)	(ft/s)	(ft/s)		(m)	(m/s)	(m/s)	
1.6	580	1010	0.26	0.5	180	310	0.26
3.3	700	1030	0.07	1.0	210	310	0.07
4.9	960	1470	0.13	1.5	290	450	0.13
6.6	870	1590	0.29	2.0	260	490	0.29
8.2	740	1390	0.31	2.5	220	430	0.31
9.8	620	1310	0.36	3.0	190	400	0.36
11.5	480	960	0.33	3.5	150	290	0.33
13.1	550	1340	0.40	4.0	170	410	0.40
14.8	910	4070	0.47	4.5	280	1240	0.47
16.4	920	5560	0.49	5.0	280	1690	0.49
18.0	920	5460	0.49	5.5	280	1670	0.49
19.7	900	5380	0.49	6.0	270	1640	0.49
21.3	810	4980	0.49	6.5	250	1520	0.49
23.0	810	5420	0.49	7.0	250	1650	0.49
24.6	880	5290	0.49	7.5	270	1610	0.49
26.3	880	4870	0.48	8.0	270	1480	0.48
27.9	920	5210	0.48	8.5	280	1590	0.48
29.5	1050	5560	0.48	9.0	320	1690	0.48
31.2	930	5460	0.49	9.5	280	1670	0.49
32.8	1100	5700	0.48	10.0	330	1740	0.48
34.5	1470	6800	0.48	10.5	450	2070	0.48
36.1	1320	6600	0.48	11.0	400	2010	0.48
37.7	1340	6350	0.48	11.5	410	1940	0.48
39.4	1300	6290	0.48	12.0	400	1920	0.48
41.0	1300	5900	0.47	12.5	400	1800	0.47
42.7	1440	6010	0.47	13.0	440	1830	0.47
44.3	1360	5850	0.47	13.5	410	1780	0.47
45.9	1320	5750	0.47	14.0	400	1750	0.47
47.6	1320	5600	0.47	14.5	400	1710	0.47
49.2	1240	5460	0.47	15.0	380	1670	0.47
50.9	1300	5510	0.47	15.5	400	1680	0.47
52.5	1410	5650	0.47	16.0	430	1720	0.47
54.1	1330	5460	0.47	16.5	400	1670	0.47
55.8	1090	5380	0.48	17.0	330	1640	0.48
57.4	1160	5650	0.48	17.5	350	1720	0.48
59.1	1260	5750	0.47	18.0	380	1750	0.47

Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio
Based on Receiver-to-Receiver Travel Time Data - Borehole B-5

American Units				Metric Units			
Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio	Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio
	V_s	V_p			V_s	V_p	
(ft)	(ft/s)	(ft/s)		(m)	(m/s)	(m/s)	
60.7	1220	5560	0.47	18.5	370	1690	0.47
62.3	1250	5800	0.48	19.0	380	1770	0.48
64.0	1240	5460	0.47	19.5	380	1670	0.47
65.6	1360	5600	0.47	20.0	410	1710	0.47
67.3	1240	5600	0.47	20.5	380	1710	0.47
68.9	1070	5380	0.48	21.0	330	1640	0.48
70.5	1030	5420	0.48	21.5	310	1650	0.48
72.2	1050	5170	0.48	22.0	320	1580	0.48
73.8	1040	5290	0.48	22.5	320	1610	0.48
75.5	1020	5510	0.48	23.0	310	1680	0.48
77.1	1140	5600	0.48	23.5	350	1710	0.48
78.7	1190	5460	0.47	24.0	360	1670	0.47
80.4	1190	5560	0.48	24.5	360	1690	0.48
82.0	1290	5600	0.47	25.0	390	1710	0.47
83.7	1320	5700	0.47	25.5	400	1740	0.47
85.3	1330	5600	0.47	26.0	400	1710	0.47
86.9	1360	5650	0.47	26.5	410	1720	0.47
88.6	1320	5850	0.47	27.0	400	1780	0.47
89.6	1320	5750	0.47	27.3	400	1750	0.47
90.2	1380	5650	0.47	27.5	420	1720	0.47
91.9	1240	5700	0.47	28.0	380	1740	0.47
93.5	1200	5850	0.48	28.5	370	1780	0.48
95.1	1230	5800	0.48	29.0	370	1770	0.48
96.8	1210	5600	0.48	29.5	370	1710	0.48
98.4	1090	5460	0.48	30.0	330	1670	0.48
100.1	1070	5460	0.48	30.5	330	1670	0.48
101.7	1140	5560	0.48	31.0	350	1690	0.48
103.4	1150	5380	0.48	31.5	350	1640	0.48
105.0	1170	5420	0.48	32.0	360	1650	0.48
106.6	1190	5330	0.47	32.5	360	1630	0.47
108.3	1150	5420	0.48	33.0	350	1650	0.48
109.9	1100	5380	0.48	33.5	340	1640	0.48
111.6	1160	5330	0.48	34.0	350	1630	0.48

APPENDIX A

SUSPENSION VELOCITY MEASUREMENT QUALITY ASSURANCE SUSPENSION SOURCE TO RECEIVER ANALYSIS RESULTS

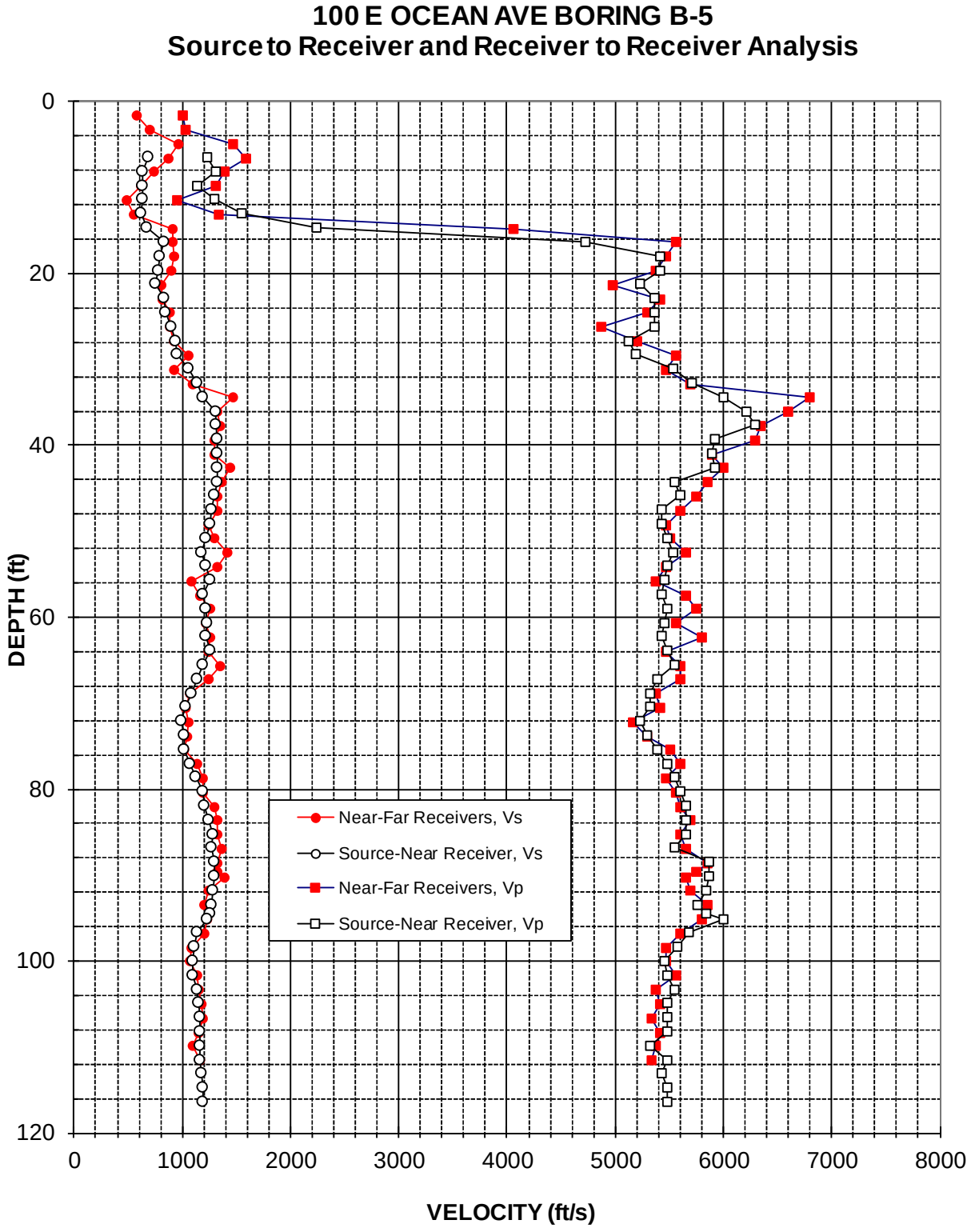

Figure A-1: Borehole B-5, Suspension S-R1 P- and S_H-wave velocities

Table A-1. Borehole B-5, S - R1 quality assurance analysis P- and S_H-wave data

Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio Based on Source-to-Receiver Travel Time Data - Borehole B-5

American Units				Metric Units			
Depth at Midpoint Between Source and Near Receiver	Velocity		Poisson's Ratio	Depth at Midpoint Between Source and Near Receiver	Velocity		Poisson's Ratio
	V_s	V_p			V_s	V_p	
(ft)	(ft/s)	(ft/s)		(m)	(m/s)	(m/s)	
6.5	690	1230	0.27	2.0	210	380	0.27
8.1	640	1310	0.35	2.5	190	400	0.35
9.8	640	1130	0.27	3.0	190	350	0.27
11.4	630	1290	0.35	3.5	190	390	0.35
13.0	610	1550	0.41	4.0	190	470	0.41
14.7	670	2240	0.45	4.5	200	680	0.45
16.3	830	4720	0.48	5.0	250	1440	0.48
18.0	790	5410	0.49	5.5	240	1650	0.49
19.6	780	5410	0.49	6.0	240	1650	0.49
21.2	750	5230	0.49	6.5	230	1590	0.49
22.9	830	5360	0.49	7.0	250	1640	0.49
24.5	850	5360	0.49	7.5	260	1640	0.49
26.2	890	5360	0.49	8.0	270	1640	0.49
27.8	940	5130	0.48	8.5	290	1560	0.48
29.4	950	5190	0.48	9.0	290	1580	0.48
31.1	1060	5530	0.48	9.5	320	1690	0.48
32.7	1130	5700	0.48	10.0	350	1740	0.48
34.4	1190	6000	0.48	10.5	360	1830	0.48
36.0	1310	6210	0.48	11.0	400	1890	0.48
37.6	1310	6300	0.48	11.5	400	1920	0.48
39.3	1320	5920	0.47	12.0	400	1800	0.47
40.9	1330	5890	0.47	12.5	410	1790	0.47
42.6	1320	5920	0.47	13.0	400	1800	0.47
44.2	1320	5550	0.47	13.5	400	1690	0.47
45.8	1300	5600	0.47	14.0	400	1710	0.47
47.5	1270	5430	0.47	14.5	390	1660	0.47
49.1	1250	5430	0.47	15.0	380	1660	0.47
50.8	1220	5480	0.47	15.5	370	1670	0.47
52.4	1180	5530	0.48	16.0	360	1690	0.48
54.0	1220	5480	0.47	16.5	370	1670	0.47
55.7	1250	5460	0.47	17.0	380	1660	0.47
57.3	1190	5430	0.47	17.5	360	1660	0.47
59.0	1210	5480	0.47	18.0	370	1670	0.47
60.6	1240	5460	0.47	18.5	380	1660	0.47
62.2	1220	5430	0.47	19.0	370	1660	0.47
63.9	1250	5480	0.47	19.5	380	1670	0.47
65.5	1190	5550	0.48	20.0	360	1690	0.48
67.2	1140	5390	0.48	20.5	350	1640	0.48
68.8	1080	5320	0.48	21.0	330	1620	0.48

Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio
Based on Source-to-Receiver Travel Time Data - Borehole B-5

American Units				Metric Units			
Depth at Midpoint Between Source and Near Receiver	Velocity		Poisson's Ratio	Depth at Midpoint Between Source and Near Receiver	Velocity		Poisson's Ratio
	V_s	V_p			V_s	V_p	
(ft)	(ft/s)	(ft/s)		(m)	(m/s)	(m/s)	
70.5	1030	5320	0.48	21.5	310	1620	0.48
72.1	990	5230	0.48	22.0	300	1590	0.48
73.7	1010	5300	0.48	22.5	310	1610	0.48
75.4	1020	5390	0.48	23.0	310	1640	0.48
77.0	1070	5480	0.48	23.5	330	1670	0.48
78.7	1120	5550	0.48	24.0	340	1690	0.48
80.3	1190	5600	0.48	24.5	360	1710	0.48
81.9	1200	5650	0.48	25.0	370	1720	0.48
83.6	1250	5650	0.47	25.5	380	1720	0.47
85.2	1280	5650	0.47	26.0	390	1720	0.47
86.9	1270	5550	0.47	26.5	390	1690	0.47
88.5	1300	5860	0.47	27.0	400	1790	0.47
90.1	1290	5860	0.47	27.5	390	1790	0.47
91.8	1280	5830	0.47	28.0	390	1780	0.47
93.4	1270	5750	0.47	28.5	390	1750	0.47
94.4	1250	5830	0.48	28.8	380	1780	0.48
95.1	1230	6000	0.48	29.0	380	1830	0.48
96.7	1130	5680	0.48	29.5	350	1730	0.48
98.3	1110	5580	0.48	30.0	340	1700	0.48
100.0	1100	5460	0.48	30.5	340	1660	0.48
101.6	1100	5480	0.48	31.0	340	1670	0.48
103.3	1140	5550	0.48	31.5	350	1690	0.48
104.9	1150	5480	0.48	32.0	350	1670	0.48
106.5	1160	5480	0.48	32.5	350	1670	0.48
108.2	1170	5480	0.48	33.0	360	1670	0.48
109.8	1160	5320	0.47	33.5	350	1620	0.47
111.5	1160	5480	0.48	34.0	350	1670	0.48
113.1	1180	5430	0.48	34.5	360	1660	0.48
114.7	1190	5480	0.48	35.0	360	1670	0.48
116.4	1190	5480	0.48	35.5	360	1670	0.48

APPENDIX B

BOREHOLE GEOPHYSICAL LOGGING SYSTEMS - NIST TRACEABLE CALIBRATION RECORDS

MICRO PRECISION CALIBRATION INC.

MICRO PRECISION CALIBRATION, INC
2165 N. Glassell St.,
Orange, CA 92865
714-901-5659

ilac-MRA ANAB
Calibration CERT # AC-1969

Certificate of Calibration

Date: Jul 14, 2016 **Cert No. 222200812421146**

Customer:
GEOVISION
1124 OLYMPIC DRIVE
CORONA CA 92881

Work Order #: N/A

MPC Control #:	AM6767	Serial Number:	160023
Asset ID:	160023	Department:	N/A
Gage Type:	LOGGER	Performed By:	TYLER MCKEEN
Manufacturer:	OYO	Received Condition:	IN TOLERANCE
Model Number:	3403	Returned Condition:	IN TOLERANCE
Size:	N/A	Cal. Date:	July 14, 2016
Temp/RH:	72.0°F / 54.0%	Cal. Interval:	12 MONTHS
		Cal. Due Date:	July 14, 2017

Calibration Notes:
See attached data sheet for calculations. (1 Page)

Calibrated IAW customer supplied data form Rev 2.1
Frequency measurement uncertainty = 0.0005 Hz
Unit calibrated with Laptop Panasonic Model CF-29,s/n: 4FKSA41798
Calibrated To 4:1 Accuracy Ratio

This Calibration has been performed in conformance with, and complies to all requirements as set forth in S&ME purchase order SCP-0022, Dated July 13, 2016

Standards Used to Calibrate Equipment

I.D.	Description.	Model	Serial	Manufacturer	Cal. Due Date	Traceability #
T1100	UNIVERSAL COUNTER	53131A	3546A09912	HEWLETT PACKARD	Feb 2, 2017	222008122827657
DB8748	GPS TIME AND FREQUENCY RECEIVER	58503A	3625A01225	HEWLETT PACKARD	Jun 17, 2017	222008122553843
AM4000	WAVEFORM GENERATOR	33250A	MY40000703	AGILENT	Jul 8, 2017	222200812420653

Calibrating Technician: TYLER MCKEEN

QC Approval: Jim Williams

The reported expanded uncertainty of measurement is stated as the standard uncertainty of measurement multiplied by the coverage factor k=2, which for normal distribution corresponds to a coverage probability of approximately 95%. The standard uncertainty of measurement has been determined in accordance with EA's Publication and NIST Technical Note 1297, 1994 Edition. Services rendered comply with ISO 17025:2005, ANSI/NCSL Z540-1, MPC Quality Manual, MPC CSD and with customer purchase order instructions.

Calibration cycles and resulting due dates were submitted/approved by the customer. Any number of factors may cause an instrument to drift out of tolerance before the next scheduled calibration. Recalibration cycles should be based on frequency of use, environmental conditions and customer's established systematic accuracy. The information on this report, pertains only to the instrument identified.

All standards are traceable to SI through the National Institute of Standards and Technology (NIST) and/or recognized national or international standards laboratories. Services rendered include proper manufacturer's service instruction and are warranted for no less than thirty (30) days. This report may not be reproduced in part or in a whole without the prior written approval of the issuing MPC lab.

MICRO PRECISION CALIBRATION INC.

MICRO PRECISION CALIBRATION, INC
2165 N. Glassell St.,
Orange, CA 92865
714-901-5659

Calibration CERT # AC-1969

Certificate of Calibration

Date: Jul 14, 2016

Cert No. 222200812421146

Procedures Used in this Event

Procedure Name	Description
GEOVISION SEISMIC	Suspension PS Seismic Logger/Recorder Calibration Procedure

Calibrating Technician: TYLER MCKEEN

QC Approval: Jim Williams

The reported expanded uncertainty of measurement is stated as the standard uncertainty of measurement multiplied by the coverage factor k=2, which for normal distribution corresponds to a coverage probability of approximately 95%. The standard uncertainty of measurement has been determined in accordance with EA's Publication and NIST Technical Note 1297, 1994 Edition. Services rendered comply with ISO 17025:2005, ANSI/NCSL Z540-1, MPC Quality Manual, MPC CSD and with customer purchase order instructions.

Calibration cycles and resulting due dates were submitted/approved by the customer. Any number of factors may cause an instrument to drift out of tolerance before the next scheduled calibration. Recalibration cycles should be based on frequency of use, environmental conditions and customer's established systematic accuracy. The information on this report, pertains only to the instrument identified.

All standards are traceable to SI through the National Institute of Standards and Technology (NIST) and/or recognized national or international standards laboratories. Services rendered include proper manufacturer's service instruction and are warranted for no less than thirty (30) days. This report may not be reproduced in part or in a whole without the prior written approval of the issuing MPC lab.

GEOVision geophysical services

SUSPENSION PS SEISMIC LOGGER/RECORDER CALIBRATION DATA FORM

INSTRUMENT DATA

System mfg.:	OYO / Robertson Geologging	Model no.:	3403
Serial no.:	160023	Calibration date:	7/14/16
By:	Emily Feldman	Due date:	7/14/17
Counter mfg.:	Hewlett Packard	Model no.:	53131A
Serial no.:	3546A09912	Calibration date:	2/02/16
By:	Micro Precision	Due date:	2/02/17
Signal generator mfg.:	Agilent	Model no.:	33250A
Serial no.:	MY40000703	Calibration date:	7/08/16
By:	Micro Precision	Due date:	7/08/17
Laptop controller mfg.:	Panasonic	Model no.:	CF-29 Tough book
Serial no.:	4FKSA41798	Calibration date:	N/A

SYSTEM SETTINGS:

Gain:	~~x10~~ EFF 7/14/16 2
Filter	Open (Low pass 10k)
Range:	200 to 50 µs
Delay:	0.3 msec
Stack (1 std)	1
System date = correct date and time	7/14/16, 1300 hrs

PROCEDURE:

Set sine wave frequency to target frequency with amplitude of approximately 0.25 volt peak
Note actual frequency on data form.
Set sample period and record data file to disk. Note file name on data form.
Pick duration of 9 cycles using PSLOG.EXE program, note duration on data form, and save as .sps file. Calculate average frequency for each channel pair and note on data form.

Average frequency must be within +/- 1% of actual frequency at all data points.

Maximum error ((AVG-ACT)/ACT*100)% As found 0.22% As left 0.22%

Target Frequency (Hz)	Actual Frequency (Hz)	Sample Period (microS)	File Name C001	Time for 9 cycles Hn (msec)	Average Frequency Hn (Hz)	Time for 9 cycles Hr (msec)	Average Frequency Hr (Hz)	Time for 9 cycles V (msec)	Average Frequency V (Hz)
50.00	50.00	200	001	180.2	50.06	179.8	50.06	180.4	49.89
100.0	100.0	100	002	90.0	100.0	90.0	100.0	89.9	100.1
200.0	200.0	50	003	45.0	200.0	45.05	199.8	44.95	200.2
500.0	500.0	20	004	18.02	499.5	18.00	500.0	18.00	~~200.~~ 500.0 EFF 7/14/16
1000	1000	10	005	9.01	998.9	9.01	998.9	9.00	1000
2000	2000	5	006	4.50	2000	4.495	~~2002.2~~ ~~2000~~ EF 7/14/16 2002	4.5	2000

Calibrated by: Name 7/14/16 Date Tyler McKeen Signature

Witnessed by: Emily Feldman Name 7/14/16 Date Signature

Suspension PS Seismic Recorder/Logger Calibration Data Form Rev 2.1 February 7, 2012

ACRONYMS AND ABBREVIATIONS

ACRONYMS AND ABBREVIATIONS

AC	asphalt concrete
ASCE	American Society of Civil Engineers
ASTM	American Society for Testing and Materials
BGS	below ground surface
CBC	California Building Code
CBR	California Bearing Ratio
CGS	California Geological Survey
CPT	cone penetration test
g	gravitational acceleration (32.2 feet/second^2)
H:V	horizontal to vertical
MCE	maximum considered earthquake
PCC	portland cement concrete
pcf	pounds per cubic foot
pci	pounds per cubic inch
psf	pounds per square foot
psi	pounds per square inch
SPT	standard penetration test