# APPENDIX E

## Geology and Soils

# APPENDIX E.1

## Geotechnical Investigations

# REPORT OF GEOTECHNICAL ENGINEERING SERVICES

For

# PROPOSED HIGH-RISE TOWER – SITE 2
1105 South Olive Street
Los Angeles, California

*Prepared For:*

**MREG 1105 Olive LLC**
**1150 South Olive Street**
**Los Angeles, CA 90015**

*Prepared By:*

**Langan Engineering and Environmental Services, Inc.**
**18575 Jamboree Road, Suite 150**
**Irvine, California 92612**

**December 15, 2020**
**Langan Project No.: 700070601**

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Technical Excellence
Practical Experience
Client Responsiveness

December 15, 2020

Andrew Dutton and Kevin Lindquist
MREG 1105 Olive LLC
1150 South Olive Street, Suite 2250
Los Angeles, CA 90015

**Preliminary Geotechnical Report**
**Proposed High-Rise Residential Tower Development – Site 2**
**1105 South Olive Street**
**Los Angeles, California**
**Langan Project: 700070601**

Dear Mr. Dutton, Mr. Elliott, and Mr. Wareham,

Langan Engineering & Environmental Services, Inc. is pleased to submit this geotechnical investigation report for the proposed high-rise residential tower development (a.k.a. Site 2) to be constructed at 1105 South Olive Street in Los Angeles, California.

Our services were performed in general accordance with our most recent proposal dated November 13, 2020 and our agreement for professional services that was executed on December 7, 2020.

◆ ◆ ◆

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

Sincerely,
Langan Engineering & Environmental Services, Inc.

Chris Zadoorian, G.E.
Associate

AJA:SR:SHW:CJZ:
Document ID: C:\Users\CZadoorian\Desktop\Projects\DTLA-Site-2\report\700070601-Site2-121420-geor-cjz.docx

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# 1.0 INTRODUCTION

## 1.1 GENERAL

This preliminary report was prepared for the proposed high-rise residential tower development to be constructed at 1105 South Olive Street (also known as Site 2) in Los Angeles, California. The approximately 0.8-acre site is located west of the intersection of South Olive Street and West 11th Street as shown on Figure 1.

## 1.2 EXISTING SITE CONDITIONS AND FEATURES

### 1.2.1 General

The site is bound by West 11th Street on the northeast; an existing six-level, at-grade parking garage on the southwest; Margo Street (an alleyway) on the northwest; and South Olive Street on the southeast as shown on Figure 2.

Existing structures are also located along the northwest side of the alleyway, including the 7-story Grand Lofts building located at 1100 South Grand Avenue and the 37-story high-rise tower and 6-story contiguous parking garage, collectively referred to as AVEN, located at 1120 South Grand Avenue. Each adjacent structure is described briefly herein.

### 1.2.2 Six-Level Parking Garage (1127 to 1143 South Olive Street)

According to as-built plans dated December 11, 1964 prepared by William L Pereira & Associates, the existing six-level parking garage is supported on a combined foundation system that consists of CIDH shafts and deepened spread footings.

The drilled shafts were installed along the north and south building grid lines (Grid Lines A and J) and deepened spread footings were used for the remainder of the parking garage. Based on our review of the as-built plans, drilled shafts along the north building grid line (Grid Line J) consist of 24-inch-diameter CIDH shafts that are 30 and 37 feet in length below the bottom of the pile cap.

The lowest finish floor level of the existing parking garage is established at approximately Elevation 248.4 as shown on Figures 2 through 4.

### 1.2.3 Seven-Story Grand Lofts (1100 South Grand Avenue)

According to construction plans dated January 28, 2004 prepared by Killefer Flammang Architects, the existing Grand Lofts building is seven stories above grade constructed over one subterranean level that is established at approximately Elevation 234.3 as shown on Figures 2 and 6.

The existing building is established on spread and continuous footings.

### 1.2.4 37-Story Tower and Contiguous 6-Story Parking Garage (1120 South Grand Avenue)

A 37-story tower and contiguous six-story parking garage are located at 1120 South Grand Avenue, (the development is also known as AVEN). You furnished us with the design geotechnical report dated February 19, 2015 prepared by GeoDesign, Inc. (GDI) for the AVEN tower and parking garage and several addenda.

Based on the information from the prior report the existing AVEN tower is supported on a mat foundation and the contiguous parking garage is supported on spread and continuous footings designed for an allowable bearing pressure 12,000 psf.

The lowest finish floor level of the tower and parking garage are each established at approximately Elevation 224 as shown on Figures 2 and 5.

## 1.3 HISTORICAL SITE DEVELOPMENT

Sanborn maps and aerial photographs dating back to 1888 provided general background information regarding the historical site development. Based on this information, initial residential development at the site occurred prior to 1906 and consisted of dwellings and flats. By 1950 commercial developments were present at the south side of the site and the north side was apparently used for parking.

A one-story building appears at the site between 1947 and 1956 and the adjacent six-level parking garage, located on the southwest side of the site, is present after 1964. The adjacent seven-story Grand Lofts building was constructed in 1923, appearing as the Western Auto Supply Building at that time.

The site remained relatively unchanged with respect to development from 1964 through approximately 2016 when the one-story building was demolished, the parking lot was expanded to the south, and the adjacent 37-story AVEN tower was constructed.

It is likely that remnants of the prior developments will be encountered during construction.

## 1.4 PROPOSED DEVELOPMENT

You furnished us with conceptual plans dated April 10, 2020 prepared by CallisonRTKL for the proposed development.

Based on our review of the conceptual plans and the draft structural engineering basis of design, the proposed development will include construction of a 51-story, approximately 603-foot-tall residential tower over six subterranean parking levels. The lowest finished floor level for the subterranean parking will be approximately 60 feet below ground surface (BGS), corresponding to approximately Elevation 185.

Preliminary foundation loading information was not available at the time we prepared this report; however, based on similar projects, we anticipate the average applied bearing pressure of the tower will be on the order of 10,000 psf and typical dead-plus-live loading on the podium columns will be on the order of approximately 500 to 1,500 kips.

We also anticipate that the tower will be supported on a mat foundation that may be on the order of 10 feet thick, so the bottom of the mat foundation will be established at approximately Elevation 175.

Temporary excavations up to approximately 70 feet deep will be required for the proposed development and as a result temporary shoring will be required. The temporary shoring design and construction will include provisions for support of adjacent structures.

The site is also located in an LADBS-designated methane zone. As such, methane mitigation provisions will be required in accordance with LADBS Ordinance No. 175790. The required level of methane mitigation will be based on the results of a soil-gas survey.

On-site groundwater infiltration will be implemented as part of Standard Urban Storm Water Mitigation Plan measures. We anticipate that stormwater infiltration will be accomplished through the use of deep drywells, likely within the building footprint, if feasible.

## 1.5 SEISMIC DESIGN APPROACH

It's our understanding that the proposed residential tower will be designed using a non-prescriptive seismic design approach as permitted in Section 104.11 *Alternative materials, design and methods of construction and equipment* of the 2019 California Building Code (CBC). This approach allows the use of alternative methods to determine the design lateral forces on the tower in lieu of code-prescriptive CBC provisions. LADBS approved the CBC provision in LADBS Information Bulletin P/BC 2017-123 titled *Alternative Design Procedure for Seismic Analysis and Design of Tall Buildings and Buildings Utilizing Complex Structural Systems.*

In general, the alternative design approach will result in a building that remains serviceable when subjected to frequent earthquakes and a building that does not experience collapse during an extremely rare seismic event.

Seismic design for the parking garage will be performed in general accordance with LABC-prescriptive methods and/or utilizing site specific ground motions in accordance with ASCE-7-16, Chapter 11-4.8.

Site specific ground motion studies in accordance with ASCE-7-16 will be performed during the design development phase of the project along with earthquake acceleration time histories for use in the alternative design procedure.

## 1.6 PRIOR GEOTECHNICAL INVESTIGATION

### 1.6.1 Prior On-Site Borings and Laboratory Testing

We were furnished with a prior geotechnical investigation report dated June 1, 2018, prepared by GDI. GDI's explorations at the site included six borings (Borings B-1 through B-6) at the approximate locations shown on Figure 2.

The results of pertinent prior data is summarized herein followed by our conclusions and recommendations for the proposed development.

### 1.6.2 Nearby P-S Suspension Logging

As part of prior investigations at the two sites located in close proximity, GDI performed P-S suspension logging to determine the soil profile type at each site.

P-S suspension logging was performed in prior GDI boring B-9 to a depth of approximately 153 feet BGS as part of a prior investigation for the completed 37-story tower located at 1120 South Grand Avenue.

P-S suspension logging was also performed in prior GDI boring B-8 to a depth of approximately 193 feet BGS for a proposed high-rise tower planned at 1100 Olive Street. The locations of the prior P-S suspension logging are shown on Figure 2.

### 1.6.3 Statement of Responsibility for Prior Data

We have reviewed the data presented in the GDI report and nearby P-S logging and have determined that the information presented is suitable for use in developing the conclusions and preliminary geotechnical design recommendations presented herein. As such, we assume the professional responsibility for the use and interpretation of the prior GDI data.

## 2.0 SITE EXPLORATION AND SUBSURFACE CONDITIONS

## 2.1 SUBSURFACE CONDITIONS

GDI drilled a total of six borings (B-1 through B-6) to depths between approximately 75 and 125 feet BGS using hollow-stem auger equipment. The locations of the explorations are shown on Figure 2.

Asphaltic concrete (AC), 2 to 4 inches in thickness, was logged in the borings, except for B-5 where a 4-inch thick Portland cement concrete (PCC) slab was encountered. Crushed rock base was encountered underlying the AC pavement ranging in thickness from 16.0 to 22.0 inches in borings

B-3 and B-6, and a 4-inch-thick PCC slab was encountered in boring B-4 beneath the AC pavement. Base materials were not logged beneath the AC pavement in borings B-1 and B-2.

Beneath AC and PCC pavement sections, fill soil was encountered in borings B-4 and B-6 to depths of 9.0 and 8.0 feet BGS, respectively. The fill material consists of medium dense, silty sand with gravel and medium stiff, sandy silt with organics. The fill is most likely related to the prior episodes of development at the site and will be removed as part of the planned excavations.

In the remaining borings, the upper approximately 8.0 to 10.5 feet of native soil encountered beneath the pavement section consists of medium stiff to very stiff, silty and clayey soils and medium dense, silty sand or clayey sand.

Native soil encountered below the fill and/or upper medium stiff and medium dense native soils consists of dense to very dense, silty sand and poorly-graded sand with gravel and cobbles to depths between approximately 23 and 28 feet BGS.

Alternating layers of dense to very dense sand and silty sand and very stiff to hard, sandy silt, clayey silt, and silty clay were encountered in the borings below the dense sand with gravel and cobble layer.

The general site stratigraphy as interpreted from the material encountered in the GDI borings is presented on Figures 3 through 6.

Logs of prior GDI borings are presented in Appendix A.

As noted in 1.6.2, GDI also drilled two relatively deep borings at adjacent sites and performed P-S logging in those borings. The locations of the prior nearby borings are referenced on Figure 2.

## 2.2 GROUNDWATER CONDITIONS

### 2.2.1 General

The distinction between the groundwater table and groundwater seepage is significant for sites in downtown Los Angeles as intermittent and typically discontinuous, relatively shallow zones of groundwater seepage are often present. Intermittent zones of groundwater seepage would require mitigation during the construction phase of the project if encountered within the depth of the proposed excavation; however, in general, this condition would not require any permanent design considerations. Groundwater seepage, the regional groundwater table, and the historical high groundwater level are each discussed below.

### 2.2.2 Groundwater Seepage

Groundwater seepage was not observed in the explorations at the site; however, it has been our experience that groundwater seepage, though present, is not always evident in hollow-stem auger borings. In addition, the frequency and intensity of groundwater seepage varies seasonally, typically in proportion to the seasonal rainfall.

In this part of Los Angeles, perched groundwater seepage on silt and clay layers is typically sporadic, and in many cases, explorations that are in close proximity may encounter highly variable groundwater conditions. It is also typical for perched water to dissipate relatively quickly once encountered.

### 2.2.3 Groundwater Table

Based on the data from borings in the site vicinity, the groundwater table at the site is below the maximum depths explored (125.5 feet BGS). Data from GDI's prior boring B-8, performed at 1100 South Olive Street (also known as Site 3), groundwater was logged at a depth of 130 feet BGS.

### 2.2.4 Historical High Groundwater Level

Based on our review of the Seismic Hazard Zone Report for the Hollywood 7.5-Minute Quadrangle (CGS, 1998), the historical high groundwater level is approximately 110 feet BGS.

## 2.3 FIELD PERCOLATION TESTING

GDI performed field percolation testing in general conformance with the Borehole Percolation Test Procedure outlined in the *Guidelines for Design, Investigation and Reporting Low Impact Development Stormwater Infiltration* (Section GS200.2 of County of Los Angeles Administrative Manual, December 2014) in borings B-5 and B-6 at depths of 75.5 and 76.0 feet BGS, respectively.

The test procedure consisted of drilling 8-inch-diameter boreholes to the corresponding test depths below the existing ground surface; placing a 2-inch-diameter, perforated pipe in the holes; and backfilling the annulus with clean gravel to avoid caving in the test zone.

To perform the necessary testing, a 2-inch-diameter PVC pipe was installed within the hollow-stem auger simultaneously as the auger was being withdrawn from the hole. The lower 5 feet of the PVC pipe was screened and an end cap was installed at the bottom of the pipe. To prevent the caving of the boring side wall, filter pack gravel was placed around the PVC pipe as the hollow-stem auger was withdrawn.

The testing consisted of introducing water to the subsurface soil through the PVC pipe and measuring the rate of infiltration. Prior to the start of the test, pre-soaking was performed in each boring. During the two-hour pre-soaking period, the water level in each test dropped more than 12 inches within 30 minutes or less; therefore, pre-soaking was considered complete in accordance with the City of Los Angeles Low Impact Development guidelines.

Field percolation testing was initiated following the completion of the pre-soak process. Water was added and the water level drop was recorded for 10-minute intervals until stabilized rates were obtained after the fourth and fifth intervals, respectively. A stabilized rate is considered to be reached when the highest and lowest readings from three consecutive readings were within 10 percent of each other.

A reduction factor was used to adjust the infiltration rate to account for the discharge of water through gravel pack and into the sides of the borehole.

The testing indicates a design infiltration rate for the on-site soil of approximately 19 inches per hour at a depth of 75 feet BGS. The results of percolation testing are presented in Appendix B.

After the completion of the percolation test, the PVC pipe was removed from the boring and the boring was backfilled with the soil cuttings.

## 2.4 P-S SUSPENSION LOGGING

P-S suspension logging was conducted in boring B-9 from GDIs prior investigation performed at 1120 South Grand Avenue and in boring B-8 from GDIs prior investigation performed at 1100 South Olive Street. The testing was performed by GEOVision, Inc. of Corona, California.

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 performed using P-S logging techniques, the average shear wave velocity for the upper 100 feet ranged from approximately 1,540 feet per second (470 meters per second) at 1120 South Grand Avenue to 1,630 feet per second (500 meters per second) at 1100 Olive Street.

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

## 3.0 PRIOR LABORATORY TESTING

Geotechnical laboratory testing was performed on select samples from the prior investigations and included the following:

- In-place moisture and density
- Percent Passing #200
- Consolidation
- Direct shear

Results of the geotechnical testing from the investigation performed are presented in Appendix A.

## 4.0 GEOLOGIC AND SEISMIC HAZARDS

### 4.1 GEOLOGIC SETTING

The site is located within the Peninsular Ranges geomorphic province, which is characterized by northwest/southeast-trending alignments of mountains and hills and intervening basins, reflecting the influence of northwest-trending major faults and folds controlling the general geologic structural fabric of the region. This province extends northwest from Baja California into the Los Angeles Basin and west into the offshore area, including the Catalina and Channel islands. The Los Angeles Basin lies in the northern part of the Peninsular Ranges province. The Newport-Inglewood fault zone, a northwest-trending structural zone expressed at the surface by a series of discontinuous low hills, is located approximately 6 miles west-southwest of the site. The relationship of the site to local geologic features is shown on Figure 7.

### 4.2 SOIL

Based on soil mapping available from USDA (2017), the site is located in an area indicated as "urban land with slopes of 0 to 5%." The site is developed with a parking lot. Past agricultural use of the site is not documented. Site improvements are anticipated to include drainage controls and protective features to minimize soil erosion. The potential for erosion is considered low.

### 4.3 GEOLOGIC MATERIALS

The geologic units in the site region are also presented on Figure 7. As shown on published geologic mapping (Lamar, 1970; Dibblee, 1991; Campbell et al, 2014), the site is underlain by Holocene and Pleistocene age alluvium.

The soil encountered in current borings at the site and prior nearby borings are generally consistent with the mapped and published descriptions. As discussed previously, the soil encountered at the site includes areas of fill associated with prior site development underlain by native soil. The native soil includes dense to very dense sand, gravel, and silty sand and stiff to hard, clayey sediments. The alluvium of the site region is underlain at depth by siltstone bedrock of the Puente Formation. Bedrock was not encountered within the maximum 125½-foot depth boring explored in the prior GDI geotechnical investigation.

## 4.4 MINERAL RESOURCES

The aggregate resource potential for the area of the site is addressed in a report titled Mineral Land Classification of the Greater Los Angeles Area, Part IV: Classification of Sand and Gravel Resource Areas, San Gabriel Production-Consumption Region (CGS, 1982). The report addresses the sand and gravel resource potential according to the presence or absence of significant sand and gravel deposits for use in construction-grade aggregate. The resource quality of surrounding lands is reported according to the following MRZ classification system:

- MRZ-1: Areas where adequate information indicates that significant mineral deposits are not present or where it is judged that little likelihood exists for their presence
- MRZ-2: Areas where adequate information indicates mineral deposits are present or where it is judged that a high likelihood for their presence exists
- MRZ-3: Areas containing mineral deposits, the significance of which cannot be evaluated from available data
- MRZ-4: Areas where available information is inadequate for assignment to any other MRZ

The site is situated in primarily developed terrain underlain by consolidated sediments. Economically significant sources of aggregate material were not observed within the site. The site is placed within MRZ-1 defined as "adequate information indicates that significant mineral deposits are not present."

As the project area is not presently used for mineral resource extraction and does not contain identified sources of aggregate materials, the proposed project will not result in the loss of availability of any known mineral resources. Therefore, significant impacts are not anticipated.

## 4.5 LANDSLIDES

According to the County of Los Angeles GIS database and the CGS Seismic Hazard Zone map for the Hollywood quadrangle, the site is not located within an area identified as having a potential for slope instability. The site is situated in level terrain that lacks significant natural relief or slopes. The potential for landslide or slope instability is considered low.

## 4.6 FAULTS

### 4.6.1 General

The tectonics of the Southern California area are dominated by the interaction of the North American tectonic plate and the Pacific tectonic plate, which are sliding past each other in a transform motion. Although some of the motion may be accommodated by rotation of crustal blocks such as the Transverse Ranges geomorphic province (Dickinson, 1996), the San Andreas fault zone is thought to represent the major surface expression of the tectonic boundary and is thought to be accommodating most of the transform motion between the Pacific plate and North American plate. Some of the plate motion is accommodated along other northwest-trending, strike-slip faults that are related to the San Andreas system, such as the San Jacinto, Newport-Inglewood, Elsinore-Whittier, Palos Verdes, and offshore faults. Figure 8 shows the regional faults with respect to the site location.

The site does not lie within or immediately adjacent to an Alquist-Priolo Earthquake Fault Zone designated by the State of California to include traces of suspected active faulting. Alquist-Priolo Earthquake Fault Zones have been established for regional faults, including portions of the Newport-Inglewood fault zone and Hollywood fault zone located approximately 6 miles west-southwest and 4.8 miles north-northwest of the site, respectively. The site is not located within a preliminary Fault Rupture Study Area designated by City of Los Angeles (2017). The potential for fault surface rupture to occur at the site is considered very low.

### 4.6.2 Hollywood Fault

The Hollywood fault is located approximately 4.8 miles north-northwest of the site. The Hollywood fault is an oblique, left-lateral, reverse fault that places crystalline basement rock of the Santa Monica Mountains over alluvial fan deposits of the northern Los Angeles Basin. Subsurface and geomorphic investigations indicate that the fault extends along the southern flank of the Santa Monica Mountains, from the Los Angeles River to northwestern Beverly Hills. A magnitude 6.4 capability is postulated for the fault based on the fault length and estimated slip rates (Field et al., 2008). An alignment of bedrock outcrops along Sunset Boulevard, previously thought to represent the surface trace of the fault, was found to be a paleo seacliff with the active trace of the fault being located farther to the south (Lindvall et al., 2001). The Hollywood fault is included in a state-designated zone to mitigate surface rupture effects in the built environment.

While some literature references the Hollywood-Santa Monica fault, these references are describing the combined faults along the Santa Monica Mountain front. These faults are treated as different faults and are modelled as separate faults in terms of characteristic magnitudes, distances from a site, and subsurface geometry. These fault-specific characteristics are used in determining the level of ground shaking at a site. The reason that the faults are modelled separately is that there is a separation between the faults at the extension of the northwest-trending Newport-Inglewood fault.

For the purposes of determining the potential for ground surface rupture at a site, neither the Hollywood fault nor the Santa Monica fault are relevant due to the distance from each fault to the site.

### 4.6.3 Newport-Inglewood Fault

The Newport-Inglewood fault zone is a system of northwest-trending, right-lateral, en echelon faults located approximately 6 miles west-southwest of the site. The discontinuous surface expression along a series of aligned hills and topographic rises suggests a youthful stage of development for this zone. The Newport-Inglewood fault extends offshore and trends into the Rose Canyon fault system, toward the south. The 1933 Long Beach earthquake is attributed to a segment of the Newport-Inglewood fault. A magnitude 7.4 capability is postulated based on fault length and a scenario rupture along the combined Newport-Inglewood and Rose Canyon fault systems.

### 4.6.4 Raymond Fault

The Raymond fault is located approximately 5.3 miles north of the site. The Raymond fault is an approximately 23-kilometer-long feature exhibiting south-facing scarps and predominant left lateral motion (Weaver and Dolan, 2000). The Raymond fault is active, with trenching studies that have shown the most recent rupture on the Raymond fault to have occurred 1,000 to 2,000 years ago (Weaver and Dolan, 2000). A potential for magnitude 6.7 earthquakes is postulated for the Raymond fault based on the dimensions of the fault plane area. Portions of the Raymond fault are included within state-designated Alquist-Priolo Earthquake Fault Zones.

### 4.6.5 Verdugo Fault

The Verdugo fault is located approximately 7.3 miles north-northeast of the site. The Verdugo fault is a northeast-dipping, reverse fault that trends along base of the San Rafael Hills and Verdugo Mountains and merges southeasterly to the Raymond fault zone (Weber et al., 1980). Probable magnitudes between 6.0 and 6.8 are estimated based on fault length.

### 4.6.6 San Andreas Fault

The Mojave segment of the San Andreas fault zone is located along the northern margin of the San Gabriel Mountains, approximately 35 miles northeast of the site. The San Andreas fault is thought to represent the major surface expression of the tectonic boundary between the Pacific plate and the North American plate. A magnitude 7.4 earthquake is estimated for the Mojave segment of the San Andreas fault based on magnitude-length relations.

### 4.6.7 Blind Thrust Faults

The Los Angeles Basin is underlain locally by a system of buried thrust faults that terminate at a depth of approximately 3 kilometers. These buried thrust faults include the Puente Hills thrust (PHT) system and the Upper Elysian Park fault.

The PHT is a system of buried thrust fault ramps that extend from beneath Los Angeles to the Puente Hills of eastern Los Angeles County and Orange County. Identified by subsurface data, including seismic reflection profiles, petroleum well data, and precisely located seismicity, the PHT is expressed at the surface as a series of contractional folds. Fault segments designated for the PHT include the Los Angeles, Santa Fe Springs, and Coyote Hills segments (Shaw and Shearer, 1999).

The Los Angeles segment of the PHT underlies downtown Los Angeles at a depth of approximately 4 kilometers.

This buried fault system is estimated to be capable of producing earthquakes with magnitudes of 6.5 to 6.6 on individual segments or a magnitude 7.1 earthquake as a group (Shaw et al., 2002). The Santa Fe Springs segment of the PHT is postulated to be the source of the 1987 magnitude 5.9 Whittier Narrows earthquake. A study using borehole data collected from sediments overlying the central segment of the PHT indicates that subtle folding extends to the near-surface locally and reveals four events in the past 11,000 years (Dolan et al., 2003).

The active Upper Elysian Park fault is located approximately 4.8 kilometers from the site as measured to the closest portion of the fault plane. The Upper Elysian Park thrust is a blind thrust fault located above the Los Angeles segment of the PHT system. The Elysian Park anticline and associated escarpments (MacArthur Park, Coyote Pass) and Montebello anticline provide evidence of recent activity on this fault. The vertically projected plane boundaries of this fault, as depicted by Shaw et al. (2002), are located northeast of the site. The plane of this structure plunges to the north-northeast at angles between 45 and 60 degrees (Oskin et al., 2000). This buried fault is postulated to be capable of producing earthquakes with magnitudes of 6.2 to 6.7.

These faults do not present a surface rupture hazard although they are capable of producing strong ground shaking as evidenced by the 1987 magnitude 5.9 Whittier Narrows earthquake.

## 4.7 HISTORICAL EARTHQUAKES

The site is located within the seismically active southern California region. Table 1 summarizes the historical seismic events in the site region. The locations of historical seismic events of magnitude 5.0 or greater that have occurred since 1800 are shown on Figure 8. Some of the most significant seismic events in the region are provided in Table 1 below.

**Table 1. Summary of Historical Earthquakes**

| Event I.D. | Date | Magnitude | Distance from Site (miles) | Direction from Site |
|---|---|---|---|---|
| Hector Mine | 10/16/1999 | 7.1 | 120 | ENE |
| Big Bear | 6/28/1992 | 6.4 | 83 | E |
| Landers | 6/28/1992 | 7.3 | 105 | E |
| Upland | 2/28/1990 | 5.4 | 33 | ENE |
| Sierra Madre | 6/28/1991 | 5.8 | 23 | NE |
| Chino Hills | 7/29/2008 | 5.4 | 29 | ESE |
| Whittier Narrows | 10/1/1987 | 5.9 | 11 | E |
| Sylmar | 2/9/1971 | 6.6 | 27 | NNW |
| Tehachapi | 7/21/1952 | 7.3 | 44 | NW |
| Northridge | 1/17/1994 | 6.7 | 20 | NW |
| Long Beach | 3/10/1933 | 6.4 | 31 | SE |

The Long Beach, Whittier Narrows, Sylmar, and Northridge earthquakes attest to the potential for future seismic events in the southern California region to produce strong ground shaking. Any of the active faults of the region are capable of producing strong ground shaking during earthquakes. Construction according to applicable building codes can mitigate or lessen the potential for damage to site facilities.

## 4.8 GEOTECHNICAL HAZARDS

### 4.8.1 Fault Rupture

The site does not lie within or immediately adjacent to an Alquist-Priolo Earthquake Fault Zone, designated by the State of California to include traces of suspected active faulting. The site is not included in a city-designated fault hazard zone. The potential for surface fault rupture at the site is considered to be very low.

### 4.8.2 Strong Ground Motions

The site is located within a seismically active region; therefore, strong ground shaking may occur during the design life of the proposed project. However, this hazard is common in Southern California and can be mitigated by designing the proposed structures in accordance with the current Los Angeles Building Code (LABC).

### 4.8.3 Slope Stability and Landslides

According to City of Los Angeles Navigate LA (2017) and CGS (1998), the site is not located within an area identified as having a potential for slope instability. Significant natural slopes are not present on the site and the potential for slope instability and/or landslides is very low.

### 4.8.4 Erosion

The site is mantled by artificial fill soil overlying native sediments with moderate fines. The site has been developed with buildings and associated flatwork since at least 1948. The planned development is anticipated to include improvements that will conceal site soil; therefore, the potential for erosion is considered very low.

### 4.8.5 Liquefaction and Seismically Induced Settlement

Liquefaction is a process in which strong ground shaking causes saturated soil to lose its strength and behave as a fluid. Ground failure associated with liquefaction can result in severe damage to structures. The geologic conditions for increased susceptibility to liquefaction are (1) shallow groundwater (generally less than 50 feet in depth), (2) the presence of unconsolidated, sandy alluvium, typically Holocene in age, and (3) strong ground shaking. All three of these conditions must be present for liquefaction to occur.

The site is not located within an area identified as having a potential for liquefaction by CGS (1999). Based on an historical high groundwater depth and the dense nature of the native soil at the site, liquefaction is not considered a hazard at the site.

### 4.8.6 Tsunamis, Inundation, Seiche, and Flooding

The site is not located in a coastal area; therefore, tsunamis are not considered a hazard at the site. According to the County of Los Angeles General Plan (1990), the site is not located within a potential inundation area for seismically induced dam/reservoir failure.

A portion of the site is located in an area mapped as having a one percent chance of flooding to a depth of less than one foot (Zone X), based on a review of FEMA FIRM panel 06037C1617G (2018).

### 4.8.7 Subsidence

Land subsidence may be induced from withdrawal of oil, gas, or water from wells. Based on a search of the CalGEM (formerly known as Division of Oil, Gas, and Geothermal Resources [DOGGR]) GIS Well Finder online tool the site is located within the Los Angeles Downtown oil/gas field. Several plugged wells lie within a half mile of the site. According to our review of the available information from CalGEM, the likelihood of land subsidence caused by oil or gas withdrawal from oil wells is low.

### 4.8.8 Expansive and Corrosive Soil

Plasticity index values available from USDA (2017) indicate the soil at the site is non-plastic. The soil at the site is generally considered non-expansive based on the reported plasticity index values. Samples collected from the current and prior nearby borings did not exhibit expansive characteristics. Chemical testing performed for a prior site investigation indicate a "negligible" anticipated exposure to sulfate attack.

### 4.8.9 Oil Wells and Methane Gas

As stated above, the site is located within the boundaries of the Los Angeles Downtown oil field. The closest well is located approximately a quarter mile south-southwest of the site. This consists of a plugged and abandoned drill hole designated as API No. 0403730413.

The site is located in a “methane zone” designated by the City of Los Angeles Navigate LA (2017). This zone identifies areas that have a potential for accumulation of methane and other volatile gases to occur in subsurface strata. As such, methane mitigation provisions will be required in accordance with LADBS Ordinance No. 175790. The required level of methane mitigation will be based on the results of a soil gas survey.

### 4.8.10 Volcanic Eruption

The site is not located in an area of recent volcanic eruption. The potential for volcanic activity at the site is very low.

### 4.8.11 Radon Gas

The site is not located in an area of high radon potential for indoor areas (CGS, 2005). The potential for radon gas accumulation is considered low.

### 4.8.12 Off-Site Impacts

Potential geotechnical impacts to off-site areas are not anticipated due to requirements regarding grading permitting, erosion control, and avoidance of non-permitted disturbance to off-site areas required by local regulations.

## 5.0 CONCLUSIONS

### 5.1 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 Los Angeles and can be mitigated by following LABC seismic design requirements.

The site is also located within a LADBS-designated methane zone and appropriate methane mitigation provisions are required in accordance with the LADBS Ordinance.

### 5.2 FOUNDATIONS

The soil anticipated at the foundation level of proposed development generally consists of very dense sand and very stiff to hard silt and clay. These soils are suitable for supporting the proposed tower on a mat foundation and the proposed podium on a mat foundation or spread and continuous footings.

### 5.3 FLOOR SLABS

We anticipate that the tower floor slab will be established above the top of the mat foundation; therefore, the slab in this area can be supported on properly compacted fill soil. Beyond the tower mat foundation limits, the floor slab may be established on the stiff to hard and dense to very dense native soil present at the site.

### 5.4 SHORING, EXCAVATIONS, AND PERMANENT BELOW-GRADE WALLS

Temporary shoring consisting of soldier piles or sheet piles with tiebacks will be required to provide support for the mass excavation that may extend on the order of 70 feet BGS.

Temporary shoring may encounter gravel and cobbles typically in the upper approximately 10 to 28 feet. In addition, drilling could encounter localized zones of perched water on fine-grained layers.

The planned excavations will extend below the three adjacent buildings located at 1127 to 1143 Olive Street, 1120 South Grand Avenue, and 1100 South Grand Avenue. As such, surcharge loading from the adjacent building foundations will result on temporary shoring and permanent below-grade building walls.

Prior to the start of construction, tieback locations and inclinations should be checked to verify that they do not interfere with existing foundations or buried utilities. Remnants of prior developments may also be present at the site within the planned mass excavation.

## 5.5 STORMWATER INFILTRATION

Stormwater infiltration is feasible at the site provided infiltration is performed in a manner that does not adversely impact the performance of the foundation system and is in conformance with City of Los Angeles Bureau of Sanitation guidelines and regulations. Deep drywells are feasible within the footprint of the proposed tower and podium, provided they are designed as described in this report.

## 5.6 GROUNDWATER SEEPAGE

Groundwater seepage was not encountered in the explorations at the site; however, it has been observed in the general site vicinity and could be present during excavation, particularly where fine-grained soil is present. If encountered, the volume of seepage may initially be high but is expected to dissipate relatively quickly.

While we do not anticipate significant quantities of groundwater, regulatory provisions to assure proper handling and disposal of water from the site may also be required, depending on the intensity of the seepage. Regulatory considerations and provisions related to handling and disposal of groundwater seepage should be evaluated by the team a sufficient time prior to the start of construction.

Although groundwater seepage may impact temporary excavations, it will not impact permanent design considerations.

## 5.7 ON-SITE MATERIAL

Large-sized particles will be generated from the on-site excavations, particularly within the upper 10 to 25 feet. These large-sized particles will generally be difficult to re-use on site without processing to meet the specifications presented herein for fill material. The remainder of on-site materials are suitable for re-use in required fills.

PCC and AC generated from the required demolition are suitable for re-use in compacted fills provided the concrete and AC are processed and placed as recommended herein.

# 6.0 RECOMMENDATIONS

## 6.1 FOUNDATIONS

### 6.1.1 General

The tower may be supported on a mat foundation and the podium and parking garage may be supported on spread footings each established in the dense to very dense and stiff to hard native soil present at the planned foundation levels.

Foundation loading information was not available at the time this report was prepared; therefore, preliminary recommendations are presented below based on assumed foundation loading. Once specific foundation loading is available, we will need to re-evaluate the recommendations presented below and issue an addendum either confirming or updating these preliminary foundation recommendations.

### 6.1.2 Mat Foundation

Based on similar projects, we anticipate the average applied bearing pressure on the order of 12,000 psf may be applied by the mat foundation. Noting that the planned excavation will result in a pressure released on the order 7,200 psf, the net average applied bearing pressure from the mat foundation will be on the order of 4,800 psf.

We estimate that total settlement of the tower mat foundation will be on the order of 1.5 inches or less and that differential settlement of the mat foundation will be on the order of ½ inch or less.

For preliminary design of the mat foundation, a soil modulus of subgrade reaction equal to 130 pci for the stiff to hard and dense to very dense native soil may be used to compute the deformation of the mat foundation.

The recommended modulus value should be used to evaluate the deformation of the mat foundation for compatibility with the settlement estimates presented above.

### 6.1.3 Spread Footings

The proposed podium and parking garage may be supported on spread and continuous footings established in the dense to very dense and stiff to hard native soil. Spread and continuous 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 10,000 psf and can be increased by 500 psf for each additional foot of width and/or embedment depth to a maximum value of 15,000 psf. The additional 500 psf increase is assumed to begin below 2 feet from the lowest adjacent grade or floor level.

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.

Based on data available from similar projects, we anticipate dead-plus-live column loading for the proposed parking garage will be on the order of 500 to 1,500 kips. Based on these assumed values, we estimate total settlement for the proposed spread footings to be on the order of 1 inch or less and differential settlement across the parking garage and tower building footprint to be on the order of ½ inch or less, as the influence of the mat foundation would also contribute to settlement of the spread footings.

### 6.1.4 Lateral Resistance

For mat foundations and spread and continuous spread footings, lateral loading may be resisted by foundations using an ultimate passive pressure of 600 psf for footings where the concrete is placed directly against the undisturbed stiff or dense alluvial soil.

An ultimate coefficient of friction equal to 0.6 may be used when calculating resistance to sliding for foundations bearing on undisturbed stiff to hard or dense to very dense native soil. However, if a methane barrier is placed beneath the foundation, the ultimate coefficient of friction shall be reduced to 0.35.

The ultimate passive resistance and the ultimate frictional resistance may be used in combination provided the ultimate passive resistance is reduced to by 0.5 to account for the magnitude of deformation required to mobilize the full passive resistance when considering short-term seismic and wind loading.

### 6.1.5 On-Site Infiltration

On-site stormwater infiltration should be performed at least 40 feet below the bottom of the mat foundation and at least 30 feet below the bottom of the footings and may extend to at least a depth of 115 feet BGS, which is at least 10 feet above the seasonal average groundwater level based on the data from our current and prior borings at the site.

## 6.2 PERMANENT BELOW-GRADE WALLS

### 6.2.1 Design Lateral Earth Pressures

For static conditions, drained below-grade building walls should be designed to resist a trapezoidal-shaped at-rest lateral earth pressure distribution equal to 28H psf as shown on Figure 10.

For seismic loading conditions, drained below-grade building walls should be designed to resist a triangular-shaped active lateral earth pressure distribution equal to 35H psf in conjunction and a triangular-shaped seismic lateral earth pressure distribution equal to 15H psf as shown on Figure 11.

The upper 10 feet of the below-grade building walls should also be designed to resist a uniform lateral pressure of 100 psf to account for normal traffic loading as shown on Figures 10 and 11.

The load combination (active and seismic earth pressure) and the shape of the seismic pressure distribution are each based on *Seismic Earth Pressures on Cantilevered Retaining Structures* (Atik and Sitar, 2010) and *Seismic Earth Pressures: Fact or Fiction* (Lew, Sitar, and Atik, 2010).

Although not currently planned, if the surface at the top of the wall is sloped, the recommended lateral earth pressures should be increased as indicated in Table 2.

**Table 2. Permanent Below-Grade Walls – Lateral Earth Pressures**

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

### 6.2.2 Surcharge Loading from Adjacent Building Foundations

The planned excavations will extend below the foundations for the adjacent buildings and to aid in preliminary design, we developed preliminary surcharge pressures in accordance with the procedure outlined in NAVFAC DM 7.2 Chapter 3, Section 4.

Table 3 summarizes preliminary surcharge loading recommendations for each adjacent structure; please note that

**Table 3. Preliminary Surcharge Loading from Adjacent Building Foundations**

| Existing Structure | Preliminary Surcharge Pressure (psf) | Area of Application (feet BGS) |
|---|---|---|
| 37-Story Tower<br>(1120 South Grand Avenue)[1] | 1,500 | Lower 15 feet of east below-grade wall, along south 40 feet of wall |
| 7-Story Grand Lofts<br>(1100 South Grand Avenue)[2] | 1,000 | Lower 35 feet of east below-grade wall, along north 130 feet of wall |
| 6-Level Parking Garage<br>(1127 – 1143 Olive Street)[3] | 2,000 | Full height of south wall, along entire wall |

1. Based on average applied bearing pressure of 6,500 psf
2. Based on 8,000 psf applied bearing pressure
3. Based on estimated applied pressure of 3,500 psf resulting from an 8-foot-wide equivalent footing

Please note it will be necessary to update these surcharge recommendations once MKA reviews the structural conditions of these buildings.

### 6.2.3 Wall Back-Drainage

Permanent retaining walls should be constructed with adequate back-drainage to prevent the buildup of hydrostatic pressure behind the walls. Typically, a pre-fabricated geo-composite drainage board is fixed to the shoring wall and the below-grade building wall is constructed by the placement of shotcrete directly against the drainage board.

In addition to drainage boards, the Grading Division requires the installation of rock pockets consisting of 1 cubic foot of crushed rock spaced at 8-foot centers around the perimeter of the below-grade building walls to promote drainage.

The Grading Division requires each rock pocket to be drained at each 8-foot center back into the building and discharged to a suitable outlet. Drainage and discharge provisions should be included by others on the design drawings.

The impact of the Grading Division's back-drainage requirements on the design and construction of the proposed development is complicated by an LADBS requirement related to methane mitigation. The methane mitigation requirement is to include vent risers at each penetration through the below-grade building wall for the purpose of mitigating the potential for methane gas to enter the building through the penetrations. The combined result of the two requirements is that an alternative method to provide back-drainage in a manner that meets requirement may be desirable.

One alternative method includes perimeter drainage elements at the base of the below-grade building walls. Per LADBS requirements, the subject pipe only needs to be drained into the building at fewer locations and conveyed to the building sump system.

Therefore, the number of vent risers required as a function of the wall back-drainage system could be significantly reduced.

## 6.3 TEMPORARY EXCAVATIONS AND VERTICAL CUTS

If necessary, temporary, unsurcharged slopes should not exceed a 1H: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 dense native material but should not exceed 4 feet in height.

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.

## 6.4 TEMPORARY SHORING

### 6.4.1 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 where the maximum value is 26H psf as shown on Figure 12. Internally braced shoring should be designed to resist a trapezoidal earth pressure distribution where the maximum value is equal to 30H psf as shown on Figure 13.

In addition to the lateral earth pressures from the weight of the retained soil, surcharge loading from the adjacent building foundations will develop on the temporary shoring wall as well. The surcharge pressures presented in Table 3 should also be applied to temporary shoring walls at the designated locations.

For cantilevered and braced shoring design, where the surface at the top of the shoring is sloped, the recommended lateral earth pressures should be increased as indicated in Table 2.

The design of temporary shoring walls should consider the location of construction cranes and other potentially heavy equipment or loads that may act against the shoring system.

An alternative to designing the south below-grade building wall to support surcharge loading due to the existing foundation is to provide direct support to the existing foundation using solider piles (aka underpinning).

### 6.4.2 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 400 psf per foot of depth, up to a maximum of 4,000 psf of depth. 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 piles can be backfilled with controlled low strength material (CLSM) in conformance with the City of Los Angeles Department of Building and Safety information bulletin P/BC 2014-121, 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 required depth of embedment should be determined based on the provisions of and Section 1806.3.4 and Section 1807.3.2.1 of the 2020 Los Angeles Building Code.

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 400 psf for drilled solider piles. For soldier piles that are vibrated into the supporting soil, the frictional resistance between the soldier piles and the soil below the excavated level may be taken as 800 psf.

Installation of soldier piles by vibration techniques should not be installed where sensitive structures or utilities are present. We recommend the feasibility of vibration installation be evaluated by the shoring contractor.

### 6.4.3 Sheet Piles

The allowable lateral bearing value (passive value) of the native soil below the level of excavation may be assumed to be 400 psf per foot of depth, up to a maximum of 4,000 psf of depth. To resist vertical loading from tiebacks, a coefficient of friction between the sheets and the retained earth of

0.4 can be used. Due to the small surface area of the sheets, we do not recommend assuming end resistance.

It will be difficult/labor intensive to install back of wall drainage behind the sheet piles if they will be used as the permanent walls. If back of wall drainage is not provided, it will be necessary to design the basement for hydrostatic pressures.

### 6.4.4 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 6 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.

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 35 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.

### 6.4.5 Tiebacks

The use of tiebacks may be limited where existing structures are present and tieback agreements with adjacent property owners will be required.

The capacities of anchors should be determined by testing the initial anchors as outlined below. We anticipate that gravity-filled anchors will achieve 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 used, we estimate 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 use 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.

Total deflection during the 24-hour, 200 percent tests should not exceed 12 inches during loading. 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. Anchor 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. 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.

### 6.4.6 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 footings (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 6,000 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 movement of the shoring, the Rakers should be tightly wedged against the footings and/or shoring system.

### 6.4.7 Monitoring

Some means of monitoring the performance of the shoring system is recommended. Monitoring should consist of periodic surveying of the lateral and vertical locations of the tops of all the soldier piles. When 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.

Additionally, we recommend an existing condition survey be performed to document the condition of the adjacent buildings along the eastern and southern edge of the proposed building. They survey should include photographs and placement of monitoring devices (crack monitors, for instance) if appropriate and should be performed prior to the start of the shoring installation.

### 6.4.8 Shoring Construction Considerations

Due to the presence of localized fill material, cobbles, and boulders; granular soil that may be subject to caving; and potential groundwater seepage perched on fine-grained layers at depth, difficult drilling is expected for soldier pile and tieback installations.

## 6.5 FLOOR SLABS

The tower floor slab will be established above the top of the tower mat foundation and the floor slab can be supported on properly compacted fill placed on the mat foundation.

Outside the limits of the mat foundation, the parking garage floor slab may be established on the native dense and/or stiff soil.

A capillary break section should be installed beneath the tower and parking garage floor slabs where finish flooring is planned. The capillary break should consist of 6 inches of gravel underlying a 15-mil HDPE membrane.

## 6.6 SEISMIC DESIGN

### 6.6.1 LABC Seismic Design Parameters

In accordance ASCE-7-16, Chapter 20, and based on the results of the prior PS-logging performed immediately adjacent to the site, the seismic site classification is C.

For preliminary design purposes, Table 4 presents LABC-prescribed seismic design parameters for Site Class C noting that we will develop a site specific response spectrum (SSRS) in accordance with our performance-based design services described briefly in Section 6.6.2.

**Table 4 –Seismic Design Parameters**

| Criteria | Value |
|---|---|
| $MCE_R$ Ground Motion at Short Periods, $S_s$ | 1.939 |
| $MCE_R$ Ground Motion at 1Second Period, $S_1$ | 0.689 |
| Site Class | C |
| Site-Modified Spectral Acceleration Value at Short Periods, $S_{MS}$ | 2.327 |
| Site-Modified Spectral Acceleration Value at 1 Second Period, $S_{M1}$ | 0.965 |
| Design Spectral Response Acceleration at short periods, $S_{DS}$ | 1.541 |
| Design Spectral Response Acceleration at 1 second period, $S_{D1}$ | 0.643 |
| $MCE_G$ Peak Ground Acceleration, $PGA_M$ | 0.995 |

Our SSRS will be developed for two levels of ground shaking. These levels of ground shaking correspond to the Risk-Targeted Maximum Considered Earthquake ($MCE_R$) and Design Earthquake (DE) determined in accordance with ASCE 7-16.

### 6.6.2 Performance-Based Design Parameters

As part of our services during the design development phase of the project, we will develop SSRS and generate earthquake time history records in accordance with the provisions outlined in LADBS

Information Bulletin P/BC 2017-123 titled *Alternative Design Procedure for Seismic Analysis and Design of Tall Buildings and Buildings Utilizing Complex Structural Systems.*

Time-history record will be developed in general accordance with Chapter 16 of ASCE-7-16 in collaboration with the project structural engineer and the geotechnical and structural peer review consultants.

## 6.7 ON-SITE INFILTRATION

In accordance with the City of Los Angeles Bureau of Sanitation Low Impact Development guidelines, the invert of on-site infiltration must be at least 10 feet above the seasonal high groundwater level. Considering that the seasonal high level is at least 125 feet BGS based on the data from the prior borings, the invert of stormwater infiltration systems should be 115 feet BGS or above.

As recommended in Section 6.1.5, stormwater infiltration should be performed at least 40 feet below the bottom of the mat foundation and at least 30 feet below the bottom of the parking garage spread footings assuming a design infiltration rate of 19 inches per hour.

Drywells should be spaced at least 10 feet from proposed spread footings.

## 6.8 SITE PREPARATION

### 6.8.1 General

Site preparation for this project will primarily include exposing the bottom of foundations and floor slabs and preparing soil at the bottom of trenches, behind below-grade walls, and behind free-standing site retaining walls to receive backfill. For foundation and floor slab support, the exposed bottoms do not require special preparation, except when disturbed by construction activities.

In this case, loose or otherwise disturbed soil should be removed and either replaced with structural concrete for footing bottoms or re-compacted prior to the placement of concrete for floor slabs. 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 6.9.2.

### 6.8.2 Groundwater Seepage

If groundwater seepage is encountered within excavations, the seepage should be collected and disposed of in accordance with regulatory guidelines. If seepage results in disturbance and/or softening of the excavation bottom, the disturbed material should be removed and replaced with 1-inch-minus crushed rock to provide a firm and unyielding base.

## 6.9 GRADING CONSIDERATIONS

### 6.9.1 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.

### 6.9.2 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 D1557. Cohesive fill, though not anticipated for this project, should be compacted to at least 90 percent of the maximum dry density, as determined by ASTM D1557, 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 fill, compaction may be achieved using heavy equipment and vibration.

#### 6.9.3 Site Drainage

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

### 6.10 MATERIALS FOR FILL

#### 6.10.1 General

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. It should be noted that cobbles exceeding the size tolerances were observed in the upper approximately 8 to 28 feet BGS of the on-site material. 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.

#### 6.10.2 On-Site Native Soil

The on-site native granular soil is suitable for use in the required fills provided particles larger than 3 inches in largest dimension are removed. Where larger-sized material is used, the percentage of these materials in a representative section of the fill should be limited to 5 percent.

The on-site native silty soil is not considered suitable for use in structural fill or within 2 feet of floor slabs or other flatwork, but may be used as secondary fill in landscaping areas.

#### 6.10.3 Imported Granular Material

Imported fill material should be primarily granular in nature and reviewed by our field technician prior to import to the site.

## 7.0 SUPPLEMENTAL FIELD EXPLORATIONS AND ANALYSIS

As noted herein, the recommendations presented herein are considered preliminary pending the initiation of the design development phase of the project. At that time, our assumptions regarding foundation loading for the proposed tower and parking garage should be re-evaluated, along with foundation loading and corresponding surcharge loading from the adjacent structures.

Supplemental field explorations are also recommended to confirm the preliminary recommendations presented herein remain applicable.

## 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
- Installation of drywells

## 9.0 LIMITATIONS

We have prepared this preliminary report for use by MREG 1105 Olive 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.

## 10.0 CLOSING

We sincerely appreciate the opportunity to provide professional services for this project and look forward to working with you on this project. Please contact us at you convenience to discuss any questions you may have regarding this report.

Sincerely,
**Langan Engineering and Environmental Services, Inc.**

ANDREW J. ATRY
C 84728
signed 12/15/20

Andrew J. Atry, PE
Project Engineer

Shaun Wilkins

SHAUN H. WILKINS
No. 2665
signed 12/15/20

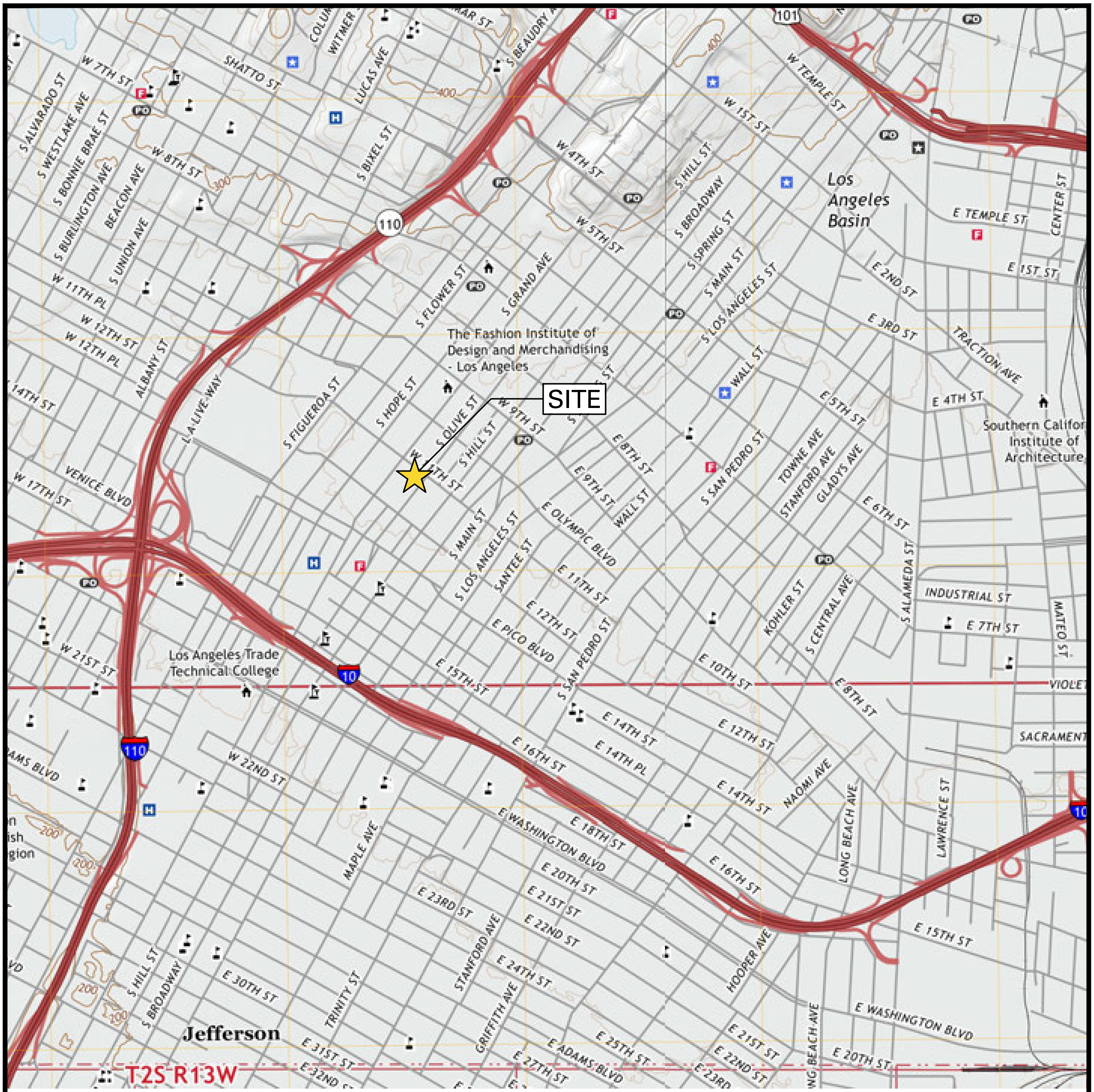
Shaun Wilkins
Senior Project Geologist

CHRISTOPHER JOHN ZADOORIAN
GE2493
signed 12/15/20

Christopher J. Zadoorian, GE
Associate

# FIGURES

**LANGAN**



**LEGEND:**

★ SITE LOCATION

2000 1000 0 2000

SCALE IN FEET

REFERENCE: USGS 7.5-MINUTE TOPOGRAPHIC MAP OF THE HOLLYWOOD AND LOS ANGELES, CA QUADRANGLES (2018).

| | | | | |
|---|---|---|---|---|
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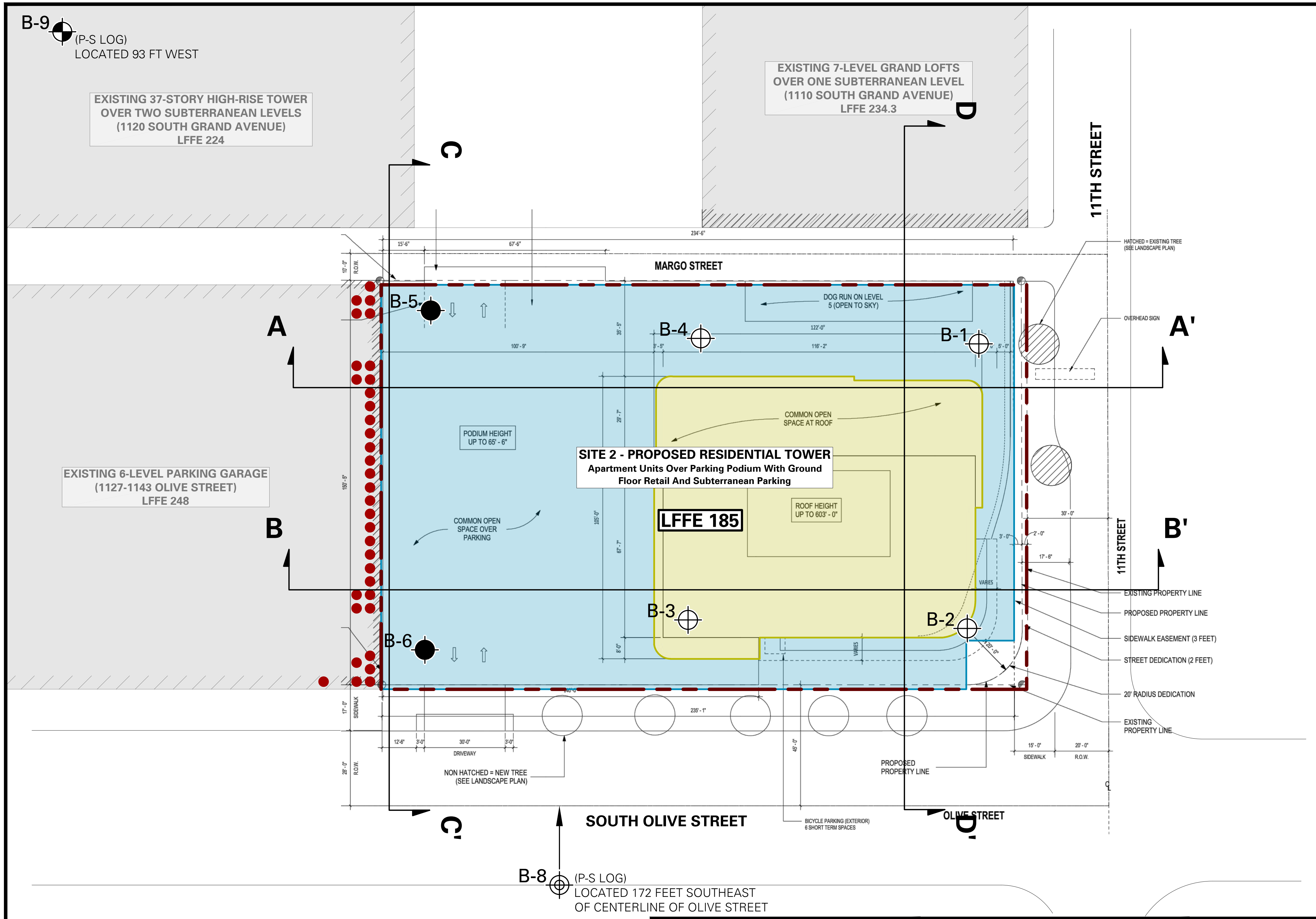

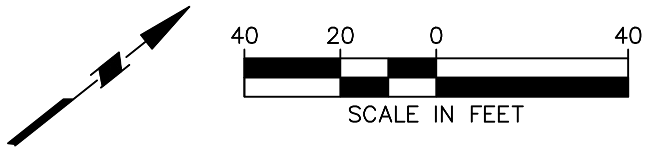

**LEGEND:**

| | |
|---|---|
| B-5 ● | PRIOR BORING<br>(GEODESIGN, 2017) |
| B-1 ⊕ | PRIOR BORING<br>(GEODESIGN, 2007) |
| B-9 ⊕ | PRIOR BORING<br>(GEODESIGN 2017, DTLASITE3-1-01) |
| B-8 ⊕ | PRIOR BORING<br>(GEODESIGN 2015, DTLASITE1-01-01-03) |
| ▬ ▬ | PROPERTY LINE |
| 🟦 | PROPOSED FOUNDATION FOOTPRINT |
| 🟨 | LIMITS OF HIGH-RISE RESIDENTIAL TOWER |
| LFFE 185 | LOWEST FINISHED FLOOR ELEVATION (FEET, MSL) |
| ● | EXISTING CIDH SHAFTS |

**NOTES:**

1. BACKGROUND SITE PLAN REFERENCED FROM ARCHITECTURAL PLANS PREPARED BY CALLISONRTKL, PROJECT NO.: 040-170198.00, DATED APRIL 10, 2020.
2. PRIOR BORING LOCATIONS REFERENCED FROM REPORT PREPARED BY GEODESIGN, PROJECT: DTLASITE2-1-01, DATED JUNE 1, 2018.

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Project

# PROPOSED HIGH-RISE RESIDENTIAL TOWER

1105 SOUTH OLIVE STREET
LOS ANGELES, CA, 90015
LOS ANGELES COUNTY CALIFORNIA

Figure Title

# SITE PLAN

Project No.
700070601

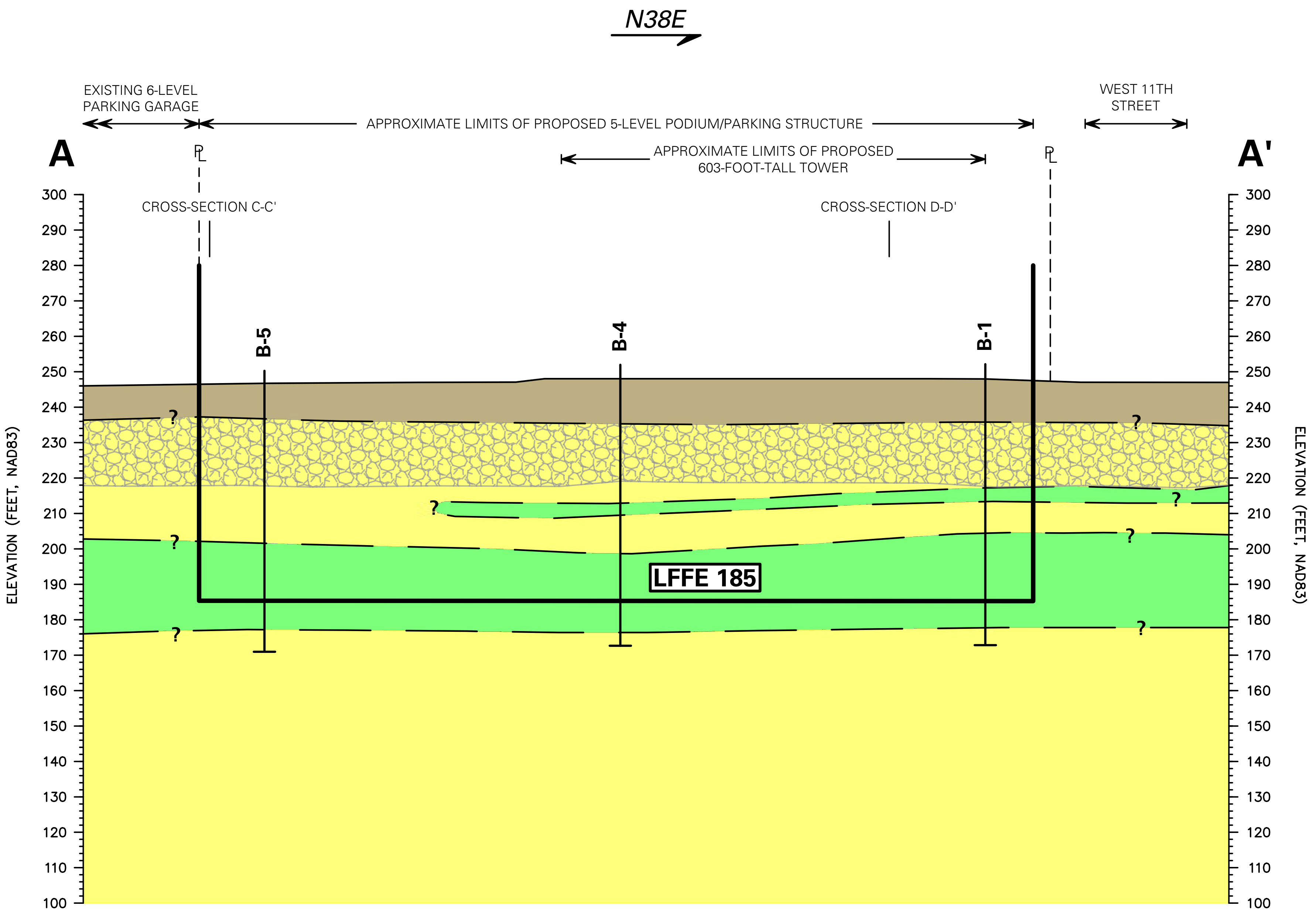
Date
DECEMBER 2020

Scale
AS SHOWN

Drawn By
MAG

Figure No.
2

Filename: \\langan.com\data\IRV\data6\700070601\Project Data\CAD\01\2D-DesignFiles\GEOTECHNICAL FIGURES.dwg Date: 12/15/2020 Time: 10:37 User: cconstantino Style Table: Langan.stb Layout: 2 - SP

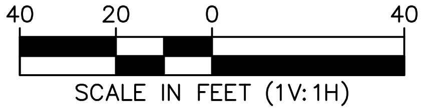


**LEGEND:**

| | |
|---|---|
| 🟫 | UPPER SILT OR FILL |
| 🟩 | STIFF TO HARD SILT AND CLAY |
| 🟨 | DENSE TO VERY DENSE SAND |
| ▨ | COBBLES |
| LFFE 185 | PROPOSED LOWEST FINISHED FLOOR ELEVATION (FEET, MSL) |
| — ? — | INFERRED GEOLOGIC CONTACT |

**NOTES:**

1. FIGURE DISPLAYS GENERALIZED SUBSURFACE CONDITIONS FOR A DETAILED DESCRIPTION OF CONDITIONS ENCOUNTERED, REFER TO BORING LOGS.
2. REFER TO SITE PLAN FOR LOCATION OF CROSS-SECTION.



SCALE IN FEET (1V: 1H)

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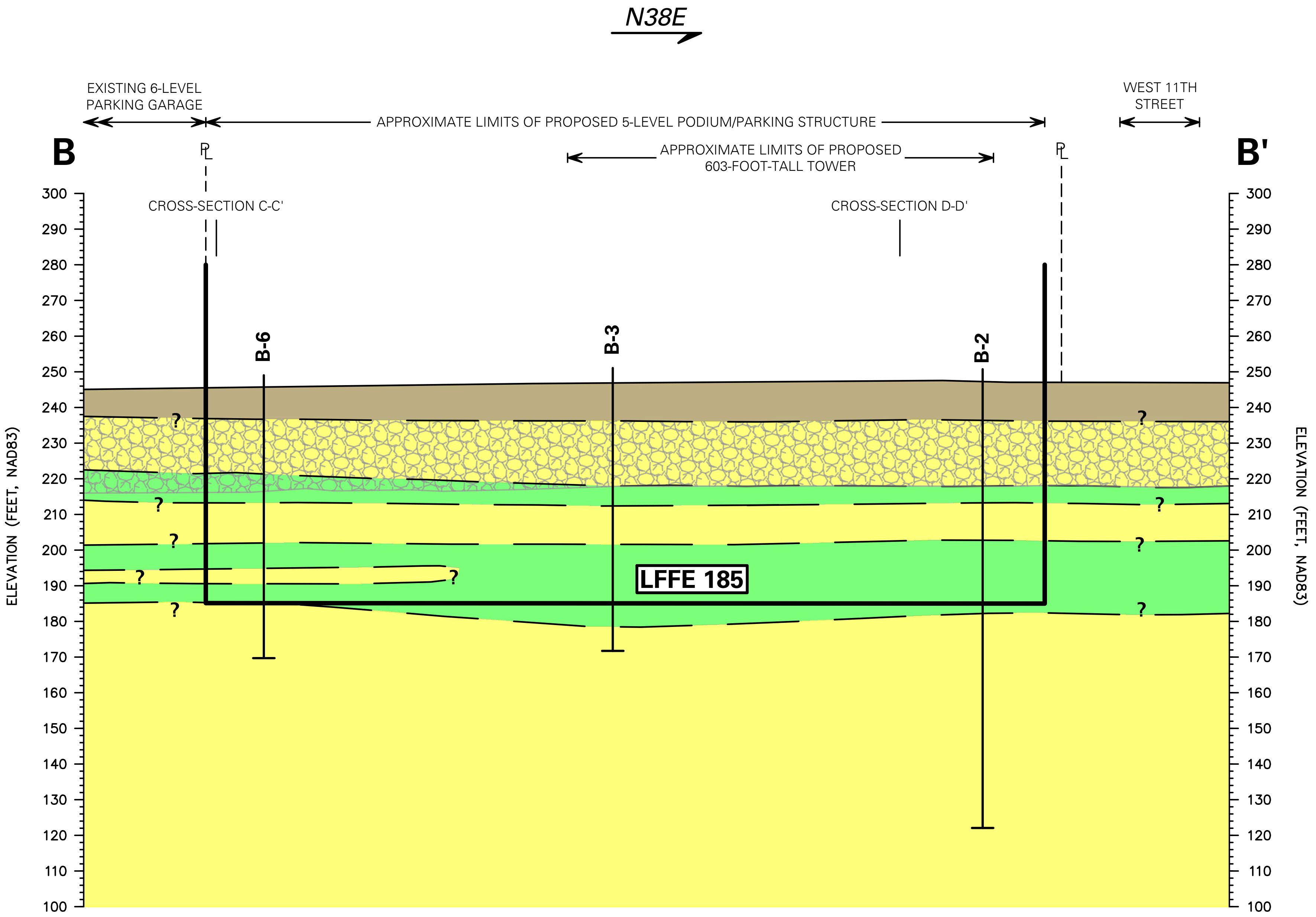
**PROPOSED HIGH-RISE RESIDENTIAL TOWER**
1105 SOUTH OLIVE STREET
LOS ANGELES, CA, 90015
LOS ANGELES COUNTY CALIFORNIA

Figure Title
**CROSS SECTION A-A'**

Project No. 700070601
Date DECEMBER 2020
Scale AS SHOWN
Drawn By CDC
Figure No. **3**

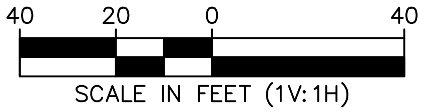


**LEGEND:**

| | |
|---|---|
| 🟫 | UPPER SILT OR FILL |
| 🟩 | STIFF TO HARD SILT AND CLAY |
| 🟨 | DENSE TO VERY DENSE SAND |
| ▨ | COBBLES |
| LFFE 185 | PROPOSED LOWEST FINISHED FLOOR ELEVATION (FEET, MSL) |
| — ? — | INFERRED GEOLOGIC CONTACT |

**NOTES:**

1. FIGURE DISPLAYS GENERALIZED SUBSURFACE CONDITIONS FOR A DETAILED DESCRIPTION OF CONDITIONS ENCOUNTERED, REFER TO BORING LOGS.
2. REFER TO SITE PLAN FOR LOCATION OF CROSS-SECTION.



SCALE IN FEET (1V:1H)

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**PROPOSED HIGH-RISE RESIDENTIAL TOWER**

1105 SOUTH OLIVE STREET
LOS ANGELES, CA, 90015
LOS ANGELES COUNTY CALIFORNIA

Figure Title

**CROSS SECTION B-B'**

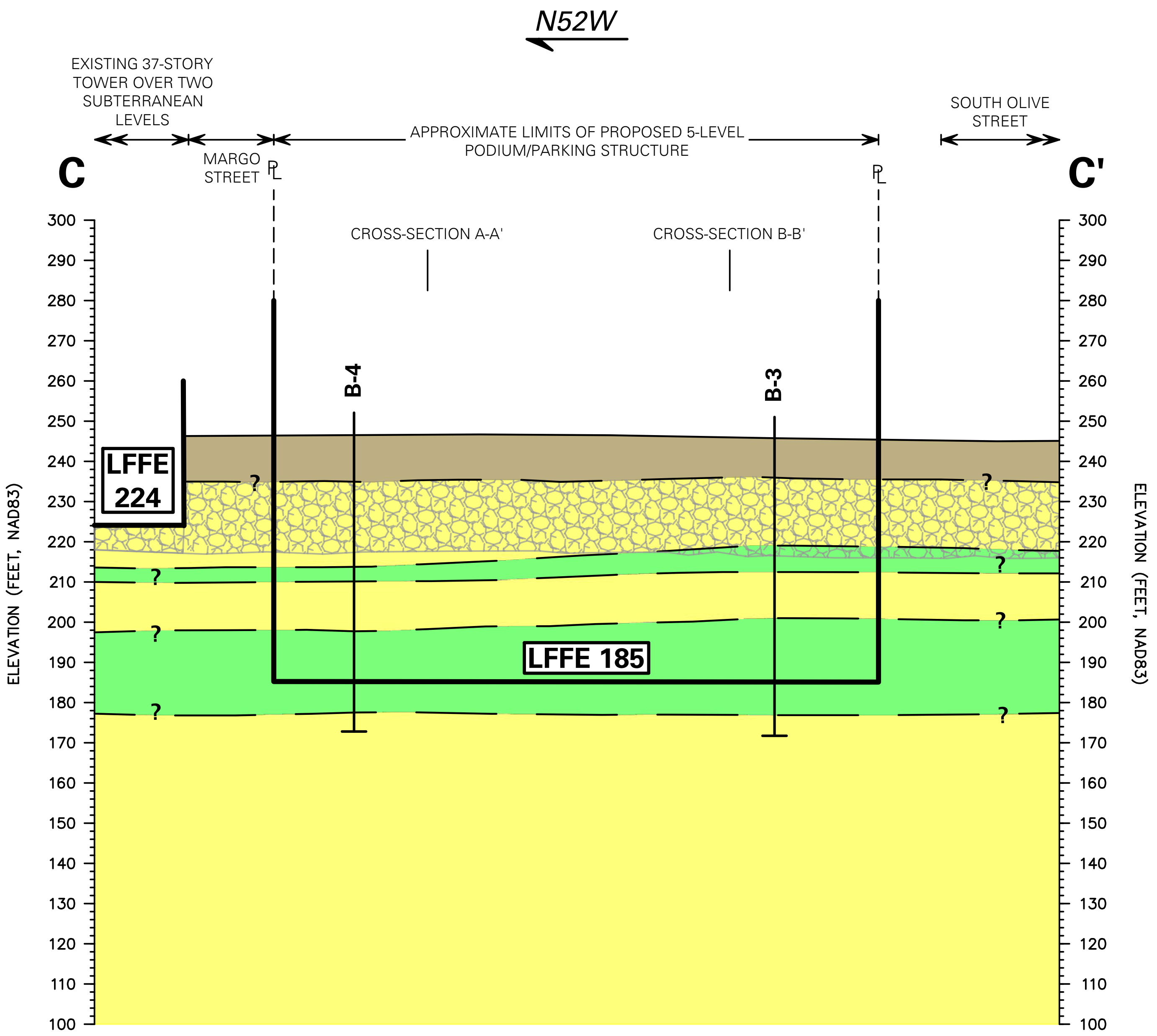
Project No. 700070601

Date DECEMBER 2020

Scale AS SHOWN

Drawn By CDC

Figure No. **4**

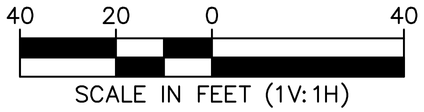


**LEGEND:**

| | |
|---|---|
| 🟫 | UPPER SILT OR FILL |
| 🟩 | STIFF TO HARD SILT AND CLAY |
| 🟨 | DENSE TO VERY DENSE SAND |
| ▨ | COBBLES |
| LFFE 185 | PROPOSED LOWEST FINISHED FLOOR ELEVATION (FEET, MSL) |
| — ? — | INFERRED GEOLOGIC CONTACT |

**NOTES:**

1. FIGURE DISPLAYS GENERALIZED SUBSURFACE CONDITIONS FOR A DETAILED DESCRIPTION OF CONDITIONS ENCOUNTERED, REFER TO BORING LOGS.
2. REFER TO SITE PLAN FOR LOCATION OF CROSS-SECTION.



SCALE IN FEET (1V:1H)

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**PROPOSED HIGH-RISE RESIDENTIAL TOWER**

1105 SOUTH OLIVE STREET
LOS ANGELES, CA, 90015
LOS ANGELES COUNTY CALIFORNIA

Figure Title

**CROSS SECTION C-C'**

Project No.
700070601

Date
DECEMBER 2020

Scale
AS SHOWN

Drawn By
CDC

Figure No.
**5**

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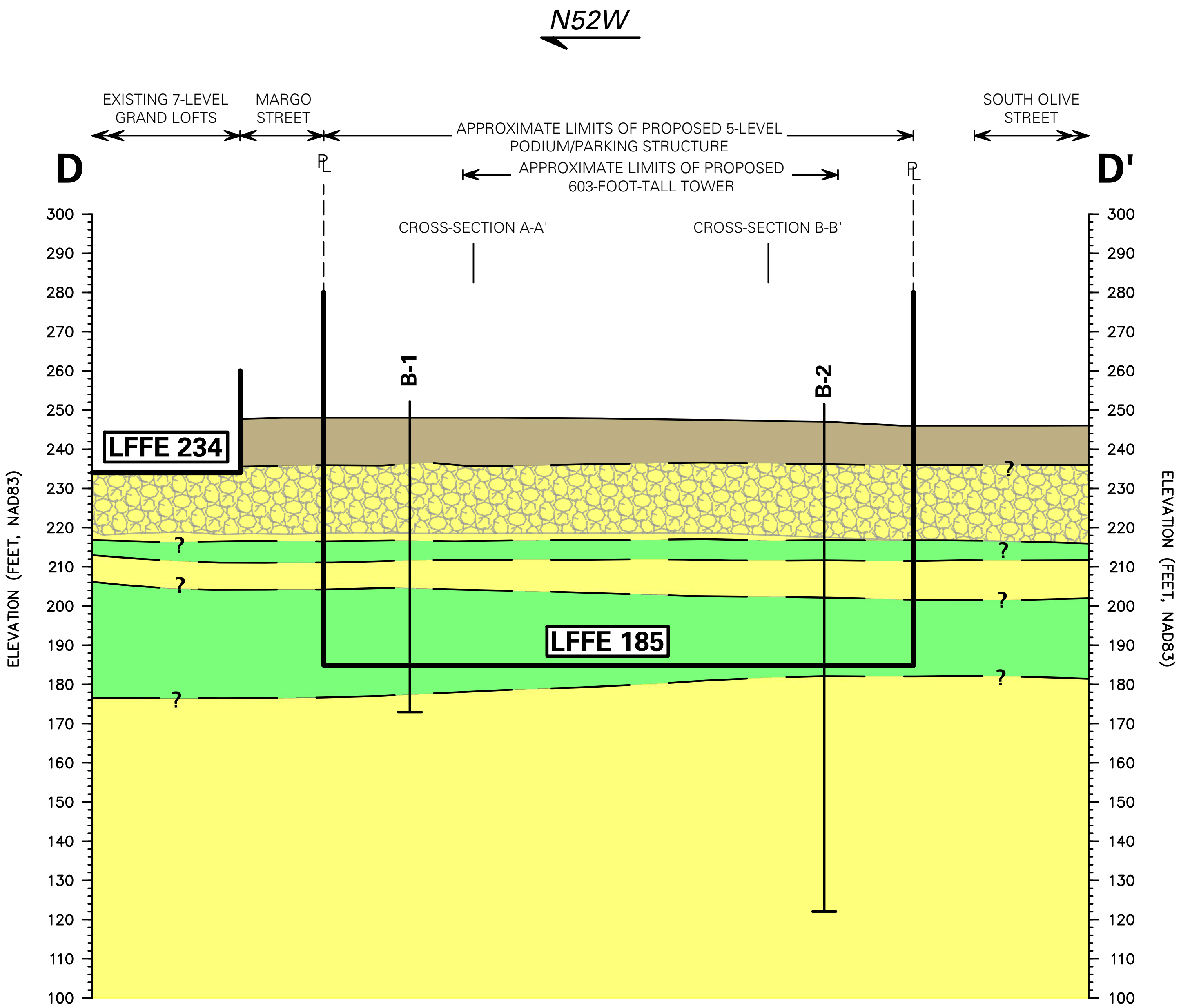
Filename: \\langan.com\data\IRV\data6\700070601\Project Data\CAD\01\2D-DesignFiles\GEOTECHNICAL FIGURES.dwg Date: 12/15/2020 Time: 10:37 User: cconstantino Style Table: Langan.stb Layout: 5- XSC



**LEGEND:**

| | |
|---|---|
| 🟫 | UPPER SILT OR FILL |
| 🟩 | STIFF TO HARD SILT AND CLAY |
| 🟨 | DENSE TO VERY DENSE SAND |
| ▨ | COBBLES |
| LFFE 185 | PROPOSED LOWEST FINISHED FLOOR ELEVATION (FEET, MSL) |
| — ? — | INFERRED GEOLOGIC CONTACT |

**NOTES:**

1. FIGURE DISPLAYS GENERALIZED SUBSURFACE CONDITIONS FOR A DETAILED DESCRIPTION OF CONDITIONS ENCOUNTERED, REFER TO BORING LOGS.
2. REFER TO SITE PLAN FOR LOCATION OF CROSS-SECTION.

40 20 0 40

SCALE IN FEET (1V:1H)

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**PROPOSED HIGH-RISE RESIDENTIAL TOWER**

1105 SOUTH OLIVE STREET
LOS ANGELES, CA, 90015
LOS ANGELES COUNTY CALIFORNIA

Figure Title

**CROSS SECTION D-D'**

Project No.
700070601

Date
DECEMBER 2020

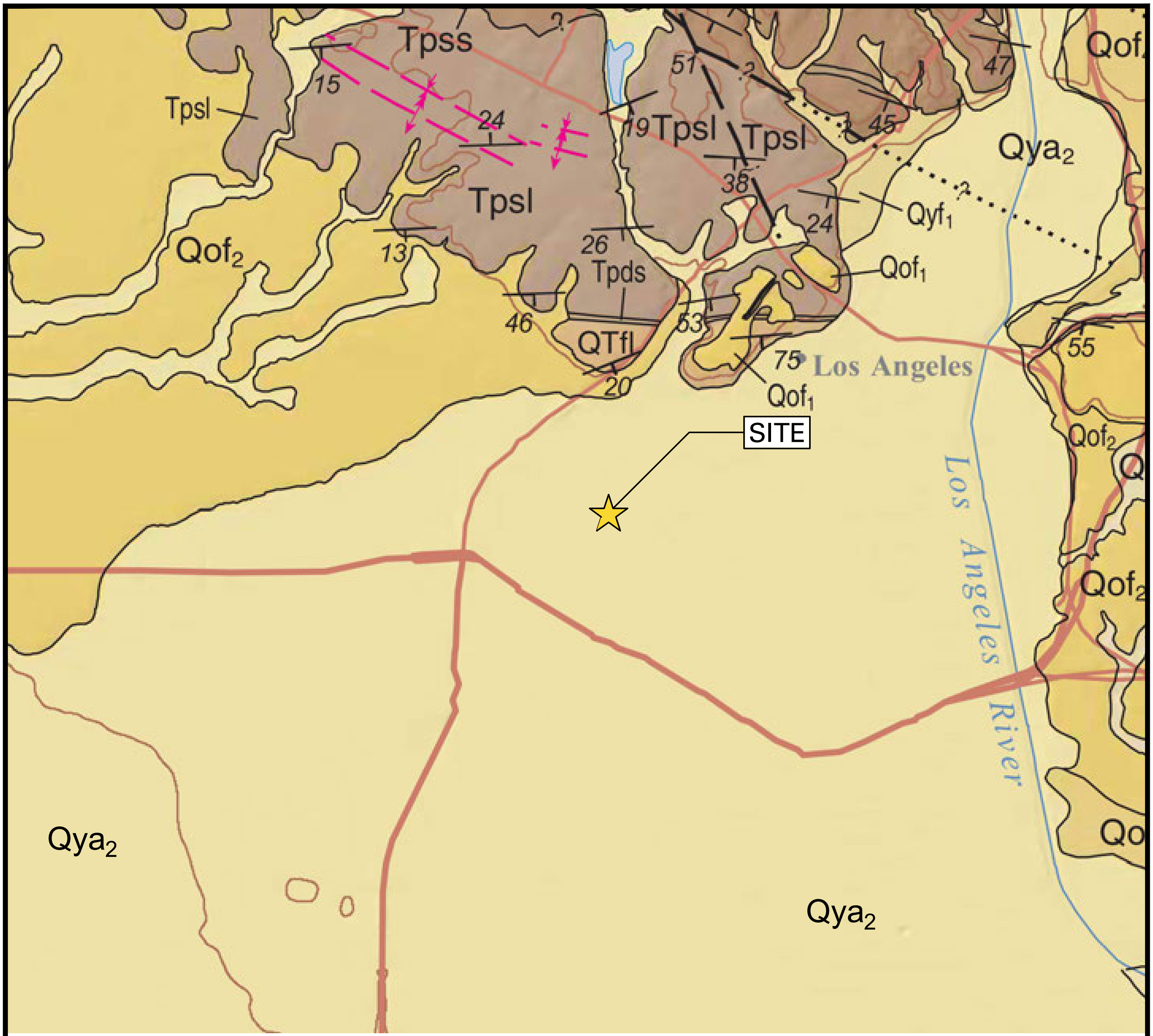
Scale
AS SHOWN

Drawn By
CDC

Figure No.
**6**

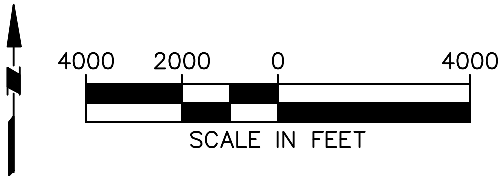
LANGAN

Filename: \\langan.com\data\IRV\data6\700070601\Project Data\CAD\01\2D-DesignFiles\GEOTECHNICAL FIGURES.dwg Date: 12/15/2020 Time: 10:37 User: cconstantino Style Table: Langan.stb Layout: 6 - XSD



**LEGEND:**

| | |
|---|---|
| Qya | YOUNG ALLUVIUM |
| Qyf | YOUNG ALLUVIAL FAN DEPOSITS |
| Qof | OLD ALLUVIAL FAN DEPOSITS |
| QTfl | LOS ANGELES BASIN FERNANDO FORMATION |
| Tpsl | PUENTE FORMATION - SILTSTONE |
| Tpds | PUENTE FORMATION - DIATOMACEOUS SHALE |
| Tpss | PUENTE FORMATION - SANDSTONE |



REFERENCE: CGS PRELIMINARY GEOLOGIC MAP OF THE LOS ANGELES 30' x 60' QUADRANGLE, CALIFORNIA (CAMPBELL ET AL., 2014).

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**PROPOSED HIGH-RISE RESIDENTIAL TOWER**

1105 SOUTH OLIVE STREET
LOS ANGELES, CA, 90015
LOS ANGLES COUNTY CALIFORNIA

Figure Title

**REGIONAL GEOLOGIC MAP**

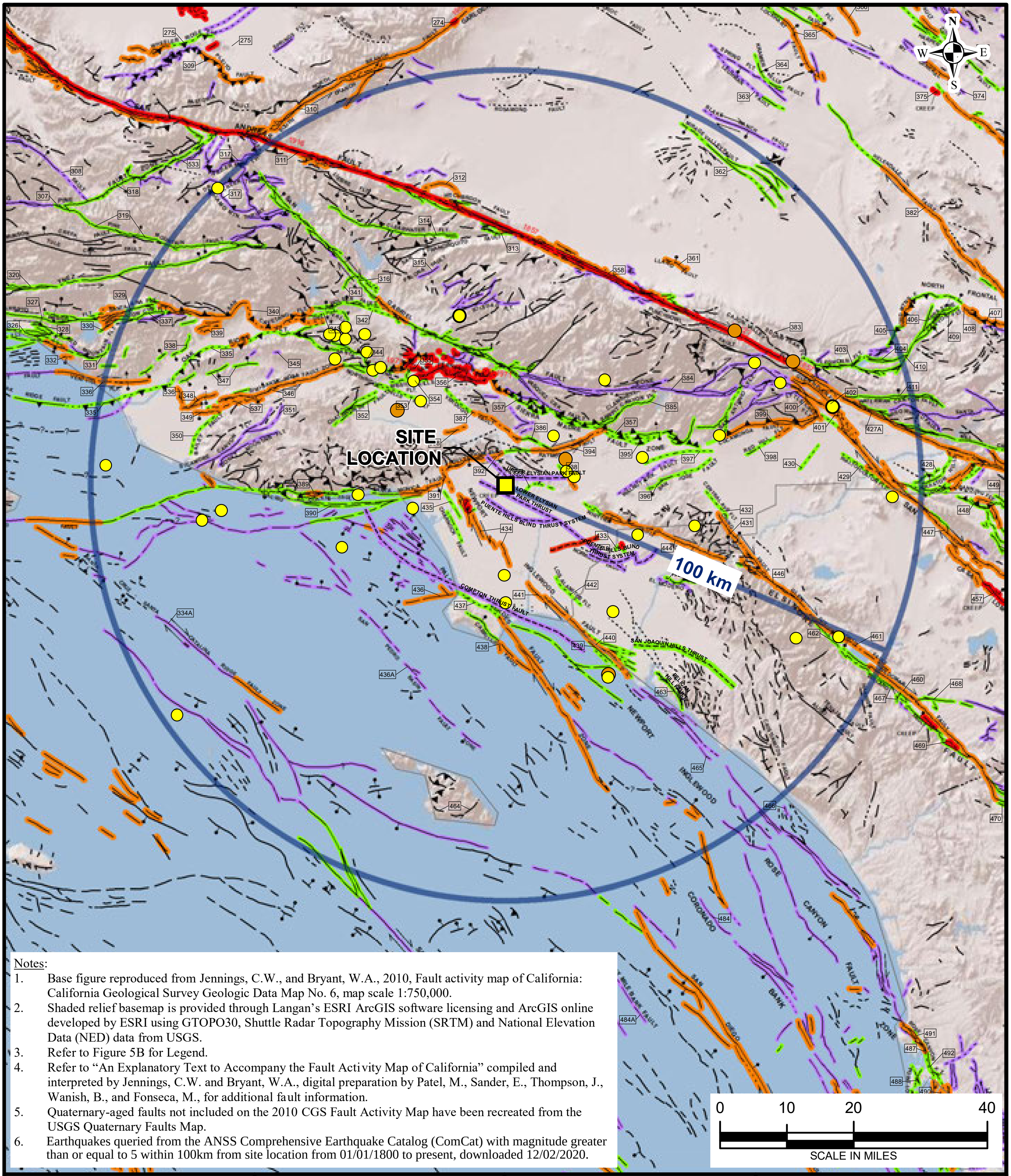
Project No.
700070601

Date
DECEMBER 2020

Scale
AS SHOWN

Drawn By
MAG

Figure No.
**7**



Notes:

1. Base figure reproduced from Jennings, C.W., and Bryant, W.A., 2010, Fault activity map of California: California Geological Survey Geologic Data Map No. 6, map scale 1:750,000.
2. Shaded relief basemap is provided through Langan's ESRI ArcGIS software licensing and ArcGIS online developed by ESRI using GTOPO30, Shuttle Radar Topography Mission (SRTM) and National Elevation Data (NED) data from USGS.
3. Refer to Figure 5B for Legend.
4. Refer to "An Explanatory Text to Accompany the Fault Activity Map of California" compiled and interpreted by Jennings, C.W. and Bryant, W.A., digital preparation by Patel, M., Sander, E., Thompson, J., Wanish, B., and Fonseca, M., for additional fault information.
5. Quaternary-aged faults not included on the 2010 CGS Fault Activity Map have been recreated from the USGS Quaternary Faults Map.
6. Earthquakes queried from the ANSS Comprehensive Earthquake Catalog (ComCat) with magnitude greater than or equal to 5 within 100km from site location from 01/01/1800 to present, downloaded 12/02/2020.

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**PROPOSED HIGH-RISE RESIDENTIAL TOWER**

1105 SOUTH OLIVE STREET
LOS ANGELES, CA, 90015

LOS ANGELES COUNTY CALIFORNIA

Figure Title

# QUATERNARY FAULT AND EARTHQUAKE EPICENTER MAP

Project No.
700070601

Date
DECEMBER 2020

Scale
1 inch = 20 miles

Drawn By
NB

Figure
8A

Path: \\Langan.com\data\IRV\data7\700070701\Project Data\ArcGIS\MXD\Geotech_Figures\2010_Fault Map IRV.mxd Date: 12/2/2020 User: NBurke Time: 7:00:03 PM

**LEGEND:**

🟨 Site Location

## Fault Age

**The age classifications are based on geologic evidence to determine the youngest faulted unit and the oldest unfaulted unit along each fault of fault seciton**

- 🟥 Historic
- 🟧 Holocene
- 🟩 Late Quaternary
- 🟪 Quaternary
- ▭ 100 km

## Earthquake Epicenter

- ● Magnitude 5 to 5.9
- ● Magnitude 6 to 6.9
- ● Magnitude 7 to 7.4
- ● Magnitude 7.5 to 8

## Pre Quaternary Faults

- —— fault, certain
- – — – fault, approx. located
- ········ fault, concealed
- —▲— thrust fault, certain
- - ▲ - thrust fault, approx. located
- ···▲··· thrust fault, approx. located, queried
- —┴— fault, certain, barball
- ···┴·· fault, concealed, barball
- - ┴ - fault, approx. located, barball

## Quaternary Faults

- —— fault, certain
- — — fault, approx. located
- —?— fault, approx. located, queried
- – ? – fault, inferred, queried
- ······ fault, concealed
- ··?·· fault, concealed, queried
- —▼— thrust fault, certain
- - ▼ - thrust fault, approx. located
- ···▼··· thrust fault, concealed
- —— dextral fault, certain
- – — – dextral fault, approx. located
- ······ dextral fault, concealed
- —— sinistral fault, certain
- – — – sinistral fault, approx. located
- ······ sinistral fault, concealed
- —— thrust fault, certain (2)
- — — thrust fault, approx. located (2)
- ······ thrust fault, concealed (2)
- —┴— fault, solid, barball
- - ┴ - fault, dashed, barball
- ···┴··· fault, dotted, barball
- —┴— dextral fault, solid, barball
- ··?·· fault, dotted, queried, ballbar
- ··?·· fault, dotted, queried, ballbar (2)
- —— fault, solid, dip
- — — fault, dashed, dip
- ······ fault, dotted, dip
- —┴— reverse fault, solid
- - ┴ - reverse fault, dashed
- ···┴··· reverse fault, dotted

| LANGAN | Project | Figure Title | Project No. | Figure |
|---|---|---|---|---|
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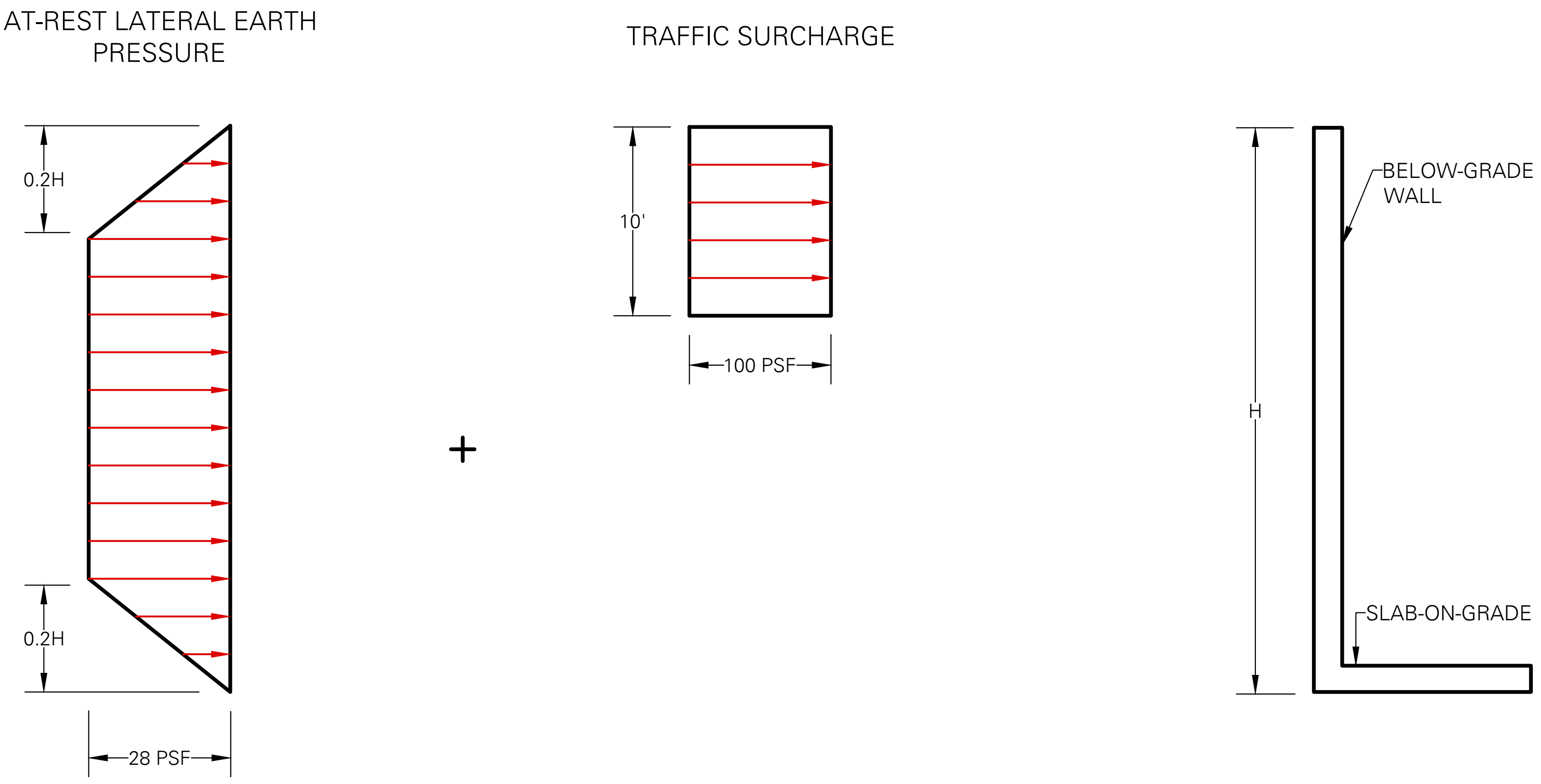
Path: \\Langan.com\data\IRV\data7\700070701\Project Data\ArcGIS\MXD\Geotech_Figures\2010_CGS Fault Map Legend.mxd Date: 12/2/2020 User: NBurke Time: 6:59:22 PM



**NOT TO SCALE**

NOTE: IN ADDITION TO THESE LATERAL EARTH PRESSURES, SURCHARGE PRESSURES PRESENTED IN SECTION 6.2 SHOULD BE INCLUDED WHERE APPLICABLE.

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ARIZONA CALIFORNIA
ABU DHABI ATHENS DOHA
DUBAI ISTANBUL PANAMA
Langan Engineering & Environmental Services, Inc.

Project

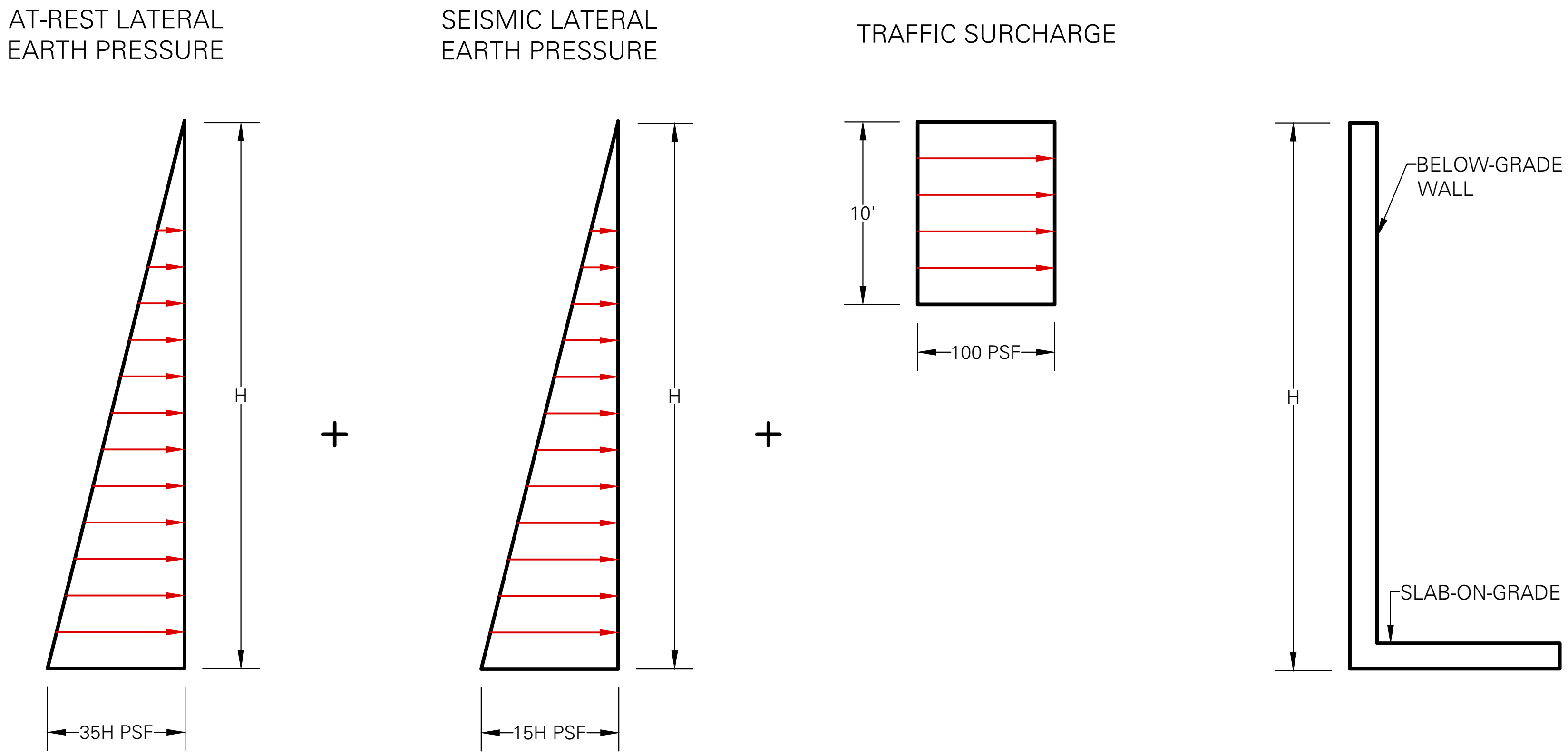
**PROPOSED HIGH-RISE RESIDENTIAL TOWER**

1105 SOUTH OLIVE STREET
LOS ANGELES, CA, 90015
LOS ANGELES COUNTY CALIFORNIA

Figure Title

**AT-REST LATERAL EARTH PRESSURE**

Project No.: 700070601
Date: DECEMBER 2020
Scale: NOT TO SCALE
Drawn By: CDC
Figure No.: 9



**NOT TO SCALE**

NOTE: IN ADDITION TO THESE LATERAL EARTH PRESSURES, SURCHARGE PRESSURES PRESENTED IN SECTION 6.2 SHOULD BE INCLUDED WHERE APPLICABLE.

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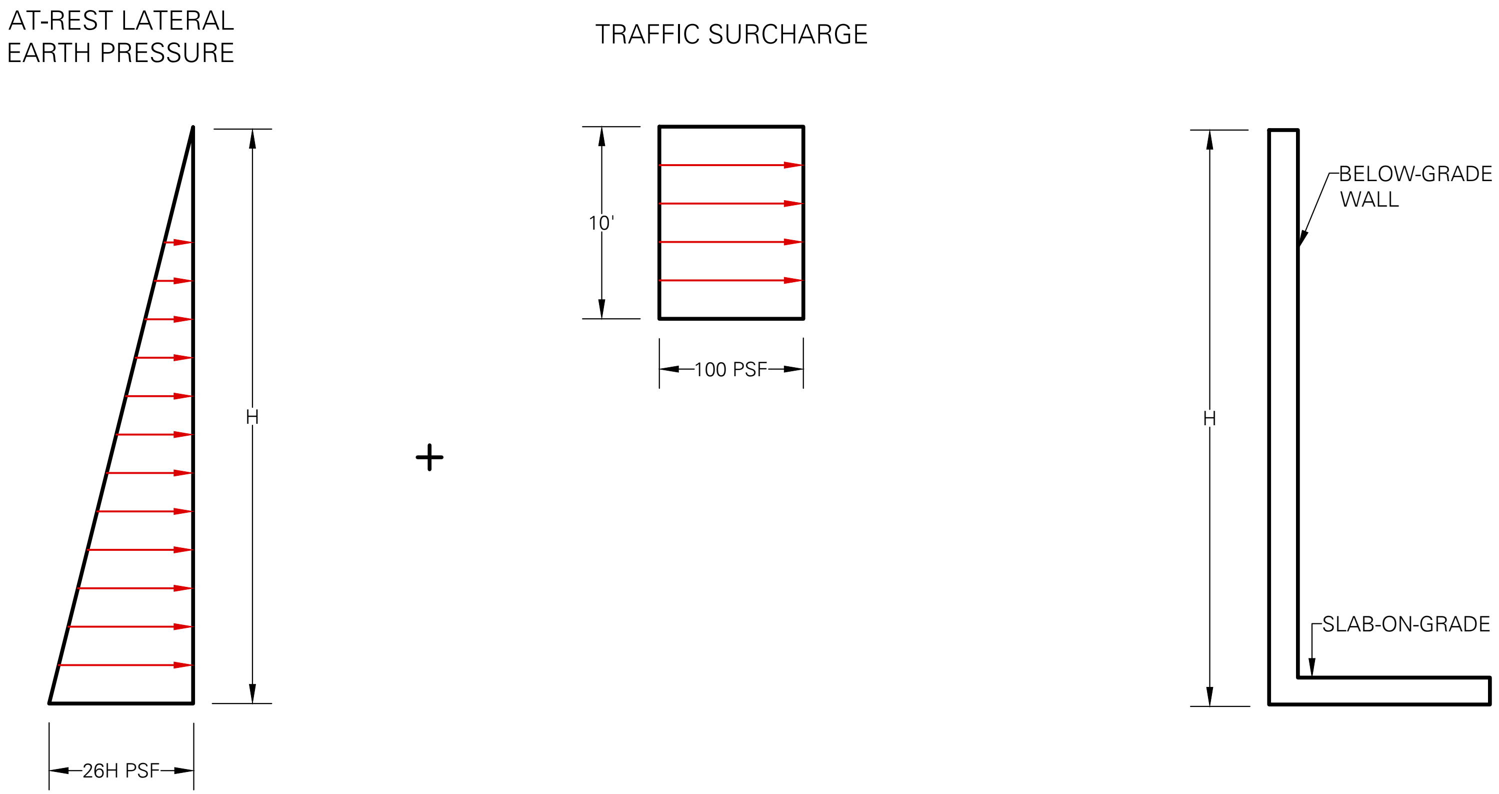
**PROPOSED HIGH-RISE RESIDENTIAL TOWER**
1105 SOUTH OLIVE STREET
LOS ANGELES, CA, 90015
LOS ANGELES COUNTY CALIFORNIA

Figure Title
**SEISMIC LATERAL EARTH PRESSURE**

Project No.
700070601
Date
DECEMBER 2020
Scale
NOT TO SCALE
Drawn By
CDC

Figure No.
**10**

Filename: \\langan.com\data\IRV\data6\700070601\Project Data\CAD\01\2D-DesignFiles\LATERAL EARTH PRESSURES\LATERAL EARTH PRESSURE.dwg Date: 12/15/2020 Time: 10:37 User: cconstantino Style Table: Langan.stb Layout: 02



**NOT TO SCALE**

NOTE: IN ADDITION TO THESE LATERAL EARTH PRESSURES, SURCHARGE PRESSURES PRESENTED IN SECTION 6.2 SHOULD BE INCLUDED WHERE APPLICABLE.

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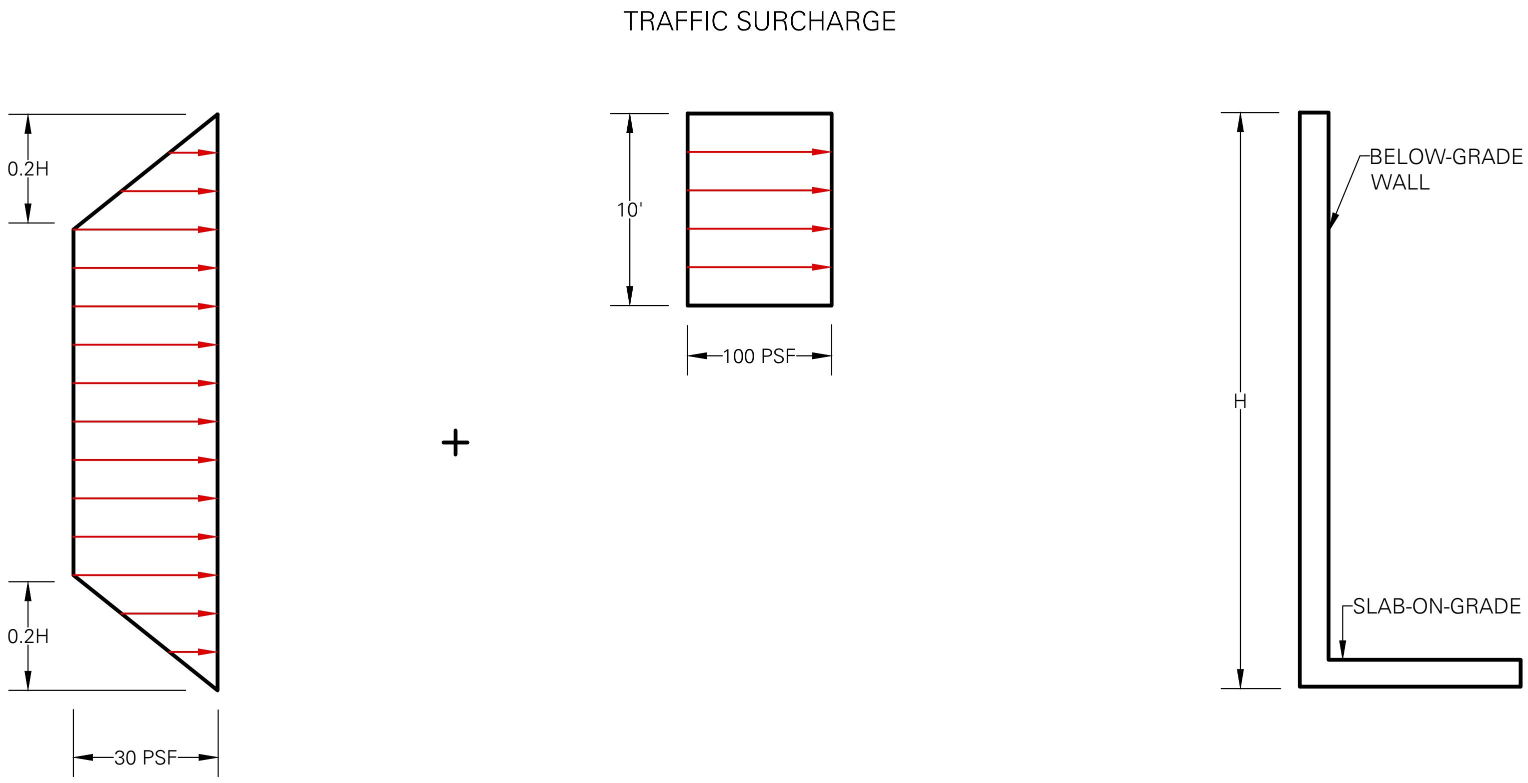
**PROPOSED HIGH-RISE RESIDENTIAL TOWER**
1105 SOUTH OLIVE STREET
LOS ANGELES, CA, 90015
LOS ANGELES COUNTY CALIFORNIA

Figure Title
**LATERAL EARTH PRESSURE - CANTILEVER SHORING**

Project No.
700070601
Date
DECEMBER 2020
Scale
NOT TO SCALE
Drawn By
CDC

Figure No.
**11**

Filename: \\langan.com\data\IRV\data6\700070601\Project Data\CAD\01\2D-DesignFiles\LATERAL EARTH PRESSURES\LATERAL EARTH PRESSURE.dwg Date: 12/15/2020 Time: 10:38 User: cconstantino Style Table: Langan.stb Layout: 03



**NOT TO SCALE**

NOTE: IN ADDITION TO THESE LATERAL EARTH PRESSURES, SURCHARGE PRESSURES PRESENTED IN SECTION 6.2 SHOULD BE INCLUDED WHERE APPLICABLE.

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Langan Engineering & Environmental Services, Inc.

Project
**PROPOSED HIGH-RISE RESIDENTIAL TOWER**

1105 SOUTH OLIVE STREET
LOS ANGELES, CA, 90015

LOS ANGELES COUNTY CALIFORNIA

Figure Title
**LATERAL EARTH PRESSURE - BRACED SHORING**

Project No.
700070601

Date
DECEMBER 2020

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CDC

Figure No.
**12**

# APPENDIX A

## Prior Field Explorations and Laboratory Testing

## APPENDIX A

## SUPLEMENTAL SUBSURFACE EXPLORATIONS

### *GENERAL*

To supplement our prior explorations and perform percolation testing, two additional borings (B-5 and B-6) were completed to depths to depths of 75.5 and 76.0 feet BGS. The borings were completed by Martini Drilling in December 2017 using hollow-stem auger drilling equipment.

The locations of the explorations were determined in the field by rolling-wheel measurements from surveyed existing site features. This information should be considered accurate only to the degree implied by the methods used.

A member of our geotechnical staff observed and logged the explorations. We collected representative samples of the various soil encountered in the explorations.

### *SOIL SAMPLING*

Samples were collected from the borings using modified California split-spoon samplers in general accordance with ASTM D3550. 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, unless otherwise noted.

In addition, SPTs were performed in the borings in general accordance with ASTM D1586. 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, unless otherwise noted.

Sampling methods and intervals are shown on the exploration logs.

### *SOIL CLASSIFICATION*

The soil samples were classified in accordance with the “Explorations Key” (Table A-1) and “Soil Classification System” (Table A-2), which are presented in this appendix. 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 are shown on the exploration logs.

## LABORATORY TESTING

### *MOISTURE CONTENT*

The natural moisture content of select soil samples was determined in general accordance with ASTM D2216. 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 D2937. 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*

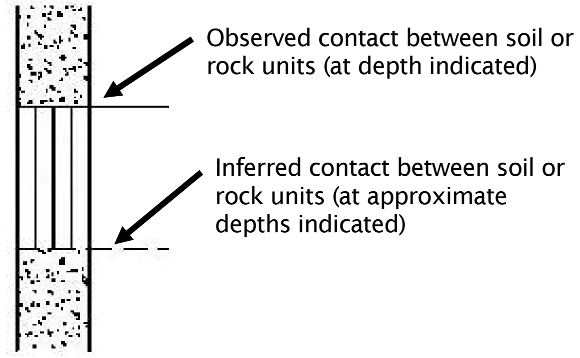
One-dimensional consolidation testing was performed in general accordance with ASTM D2435. The tests measure the volume change of a soil sample under predetermined loads. The test results are presented in this appendix.

### *STRENGTH TESTING*

Direct shear testing was performed on select soil samples general accordance with ASTM D3080. The test results are presented in this 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<br>(more than 50% retained on No. 200 sieve) | GRAVEL<br>(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<br>(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<br>(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:** | | **Percent** | **Sand and Gravel In:** | |
| | **Fine-Grained Soil** | **Coarse-Grained Soil** | | **Fine-Grained Soil** | **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**

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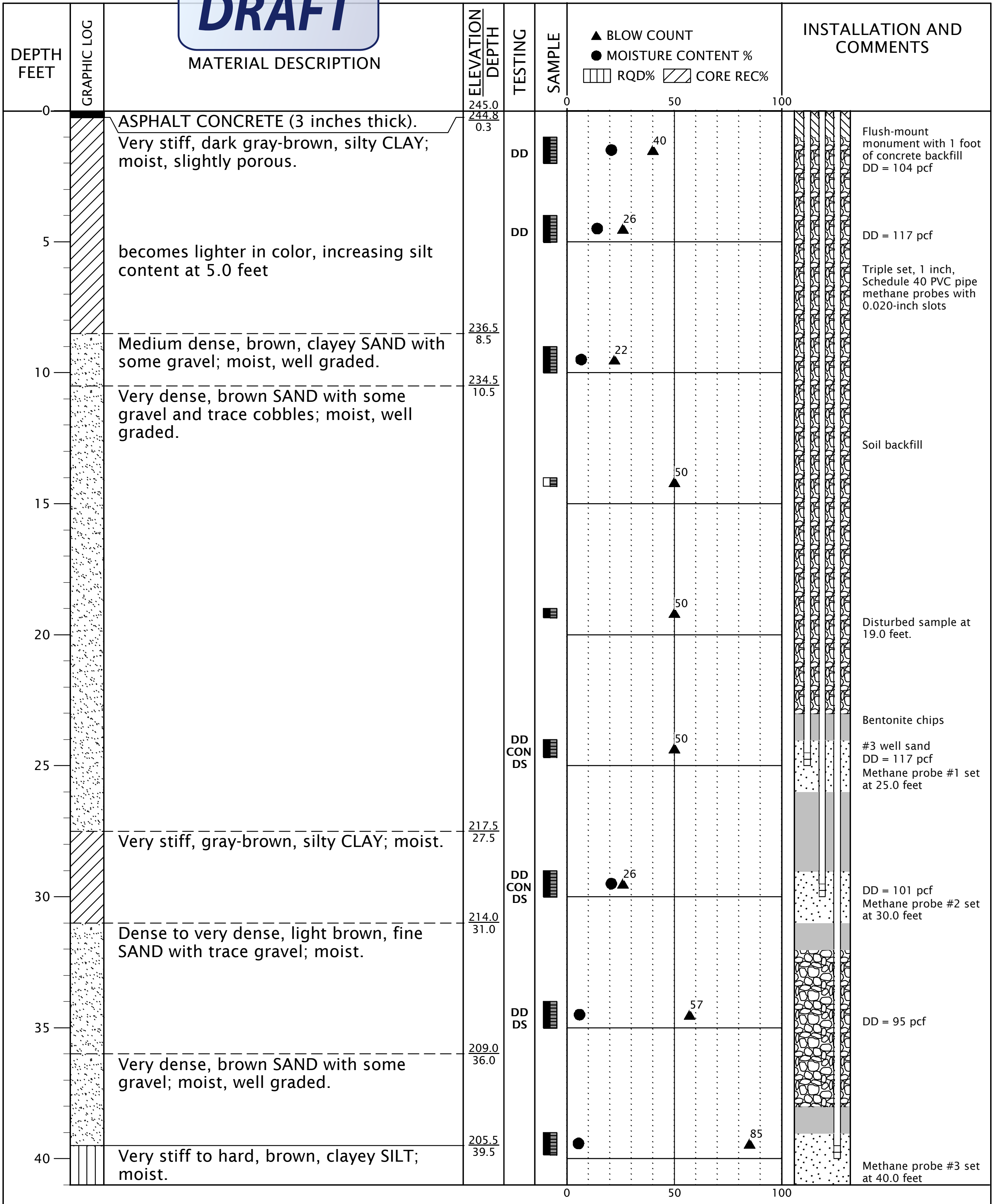

| MATERIAL DESCRIPTION | ELEVATION / DEPTH |
|---|---|
| ASPHALT CONCRETE (3 inches thick). | 244.8 / 0.3 |
| Very stiff, dark gray-brown, silty CLAY; moist, slightly porous.<br>becomes lighter in color, increasing silt content at 5.0 feet | 236.5 / 8.5 |
| Medium dense, brown, clayey SAND with some gravel; moist, well graded. | 234.5 / 10.5 |
| Very dense, brown SAND with some gravel and trace cobbles; moist, well graded. | 217.5 / 27.5 |
| Very stiff, gray-brown, silty CLAY; moist. | 214.0 / 31.0 |
| Dense to very dense, light brown, fine SAND with trace gravel; moist. | 209.0 / 36.0 |
| Very dense, brown SAND with some gravel; moist, well graded. | 205.5 / 39.5 |
| Very stiff to hard, brown, clayey SILT; moist. | |

Flush-mount monument with 1 foot of concrete backfill
DD = 104 pcf

DD = 117 pcf

Triple set, 1 inch, Schedule 40 PVC pipe methane probes with 0.020-inch slots

Soil backfill

Disturbed sample at 19.0 feet.

Bentonite chips

#3 well sand
DD = 117 pcf
Methane probe #1 set at 25.0 feet

DD = 101 pcf
Methane probe #2 set at 30.0 feet

DD = 95 pcf

Methane probe #3 set at 40.0 feet

DRILLED BY: JDK Drilling, Inc.
LOGGED BY: WKS
COMPLETED: 04/09/07

BORING METHOD: hollow-stem auger (see report text)
BORING BIT DIAMETER: 8.0 in

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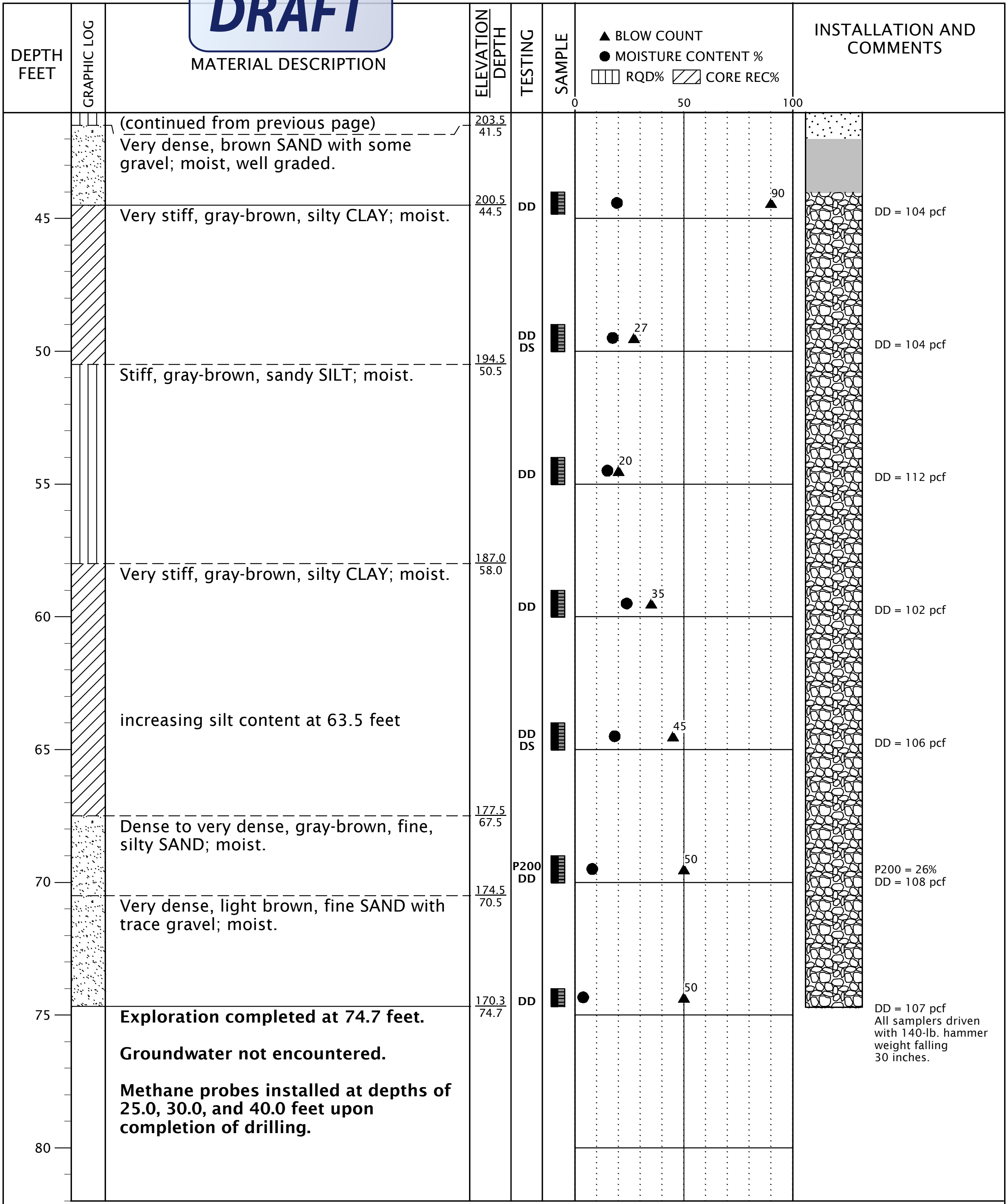
**BORING B-1**

PROPOSED RESIDENTIAL APARTMENT TOWER
LOS ANGELES, CA

**FIGURE A-1**

BORING LOG MERUELO-11-01-BORING_LOGS.GPJ GEODESIGN.GDT PRINT DATE: 5/16/07:KYK



203.5
41.5
(continued from previous page)

Very dense, brown SAND with some gravel; moist, well graded.

200.5
44.5
Very stiff, gray-brown, silty CLAY; moist.

194.5
50.5
Stiff, gray-brown, sandy SILT; moist.

187.0
58.0
Very stiff, gray-brown, silty CLAY; moist.

increasing silt content at 63.5 feet

177.5
67.5
Dense to very dense, gray-brown, fine, silty SAND; moist.

174.5
70.5
Very dense, light brown, fine SAND with trace gravel; moist.

170.3
74.7
**Exploration completed at 74.7 feet.**

**Groundwater not encountered.**

**Methane probes installed at depths of 25.0, 30.0, and 40.0 feet upon completion of drilling.**

All samplers driven with 140-lb. hammer weight falling 30 inches.

DRILLED BY: JDK Drilling, Inc.    LOGGED BY: WKS    COMPLETED: 04/09/07

BORING METHOD: hollow-stem auger (see report text)    BORING BIT DIAMETER: 8.0 in

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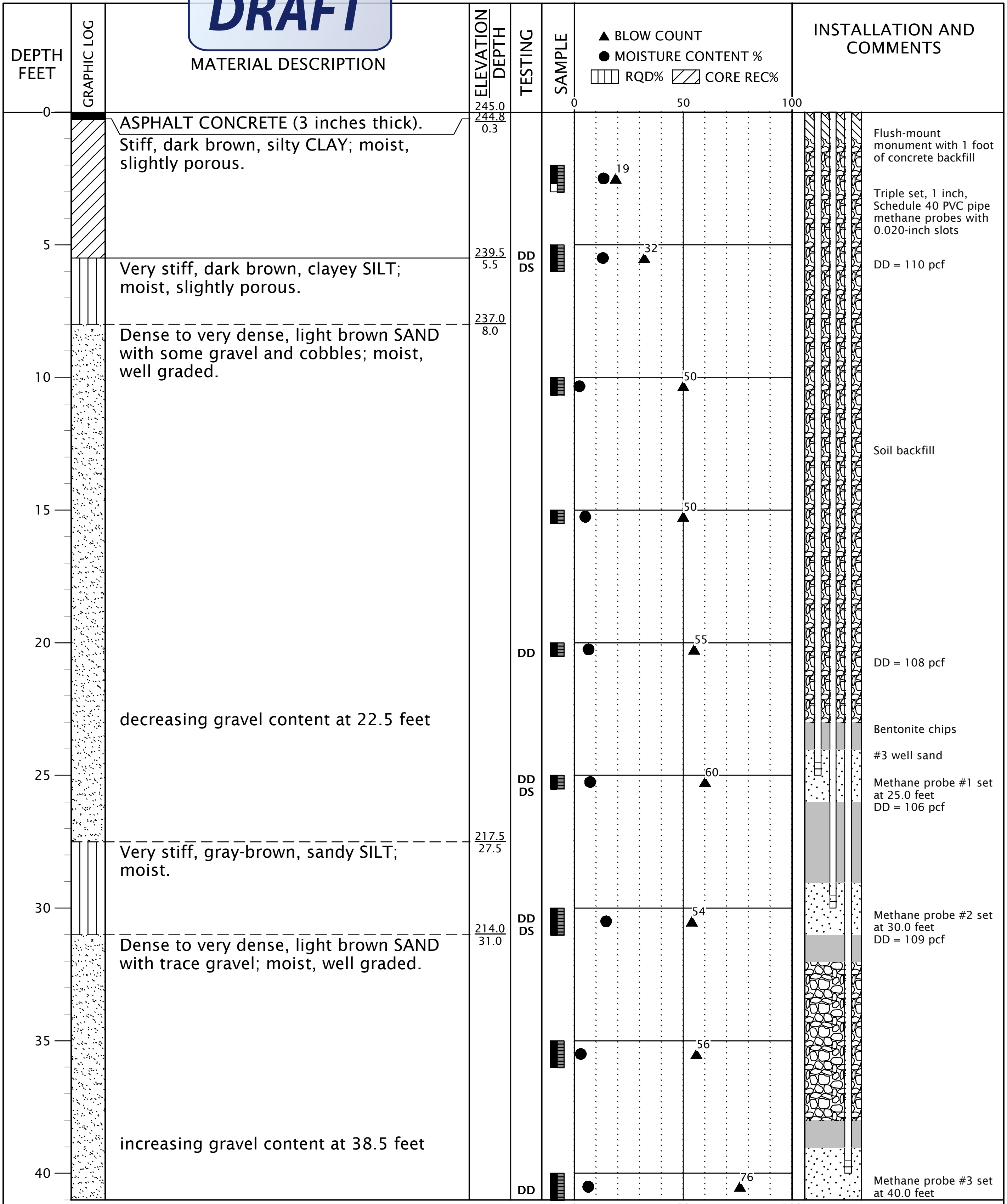
**BORING B-1**
(continued)

PROPOSED RESIDENTIAL APARTMENT TOWER
LOS ANGELES, CA

**FIGURE A-1**

BORING LOG MERUELO-11-01-BORING_LOGS.GPJ GEODESIGN.GDT PRINT DATE: 5/16/07:KYK



| Depth (feet) | Material Description | Elevation / Depth | Testing | Installation and Comments |
|---|---|---|---|---|
| 0 | ASPHALT CONCRETE (3 inches thick). | 245.0<br>244.8 / 0.3 | | Flush-mount monument with 1 foot of concrete backfill |
| 0.3 | Stiff, dark brown, silty CLAY; moist, slightly porous. | | | Triple set, 1 inch, Schedule 40 PVC pipe methane probes with 0.020-inch slots |
| 5.5 | Very stiff, dark brown, clayey SILT; moist, slightly porous. | 239.5 / 5.5 | DD<br>DS | DD = 110 pcf |
| 8.0 | Dense to very dense, light brown SAND with some gravel and cobbles; moist, well graded. | 237.0 / 8.0 | | Soil backfill |
| 20 | | | DD | DD = 108 pcf |
| 22.5 | decreasing gravel content at 22.5 feet | | | Bentonite chips<br>#3 well sand |
| 25 | | | DD<br>DS | Methane probe #1 set at 25.0 feet<br>DD = 106 pcf |
| 27.5 | Very stiff, gray-brown, sandy SILT; moist. | 217.5 / 27.5 | | |
| 30 | | | DD<br>DS | Methane probe #2 set at 30.0 feet<br>DD = 109 pcf |
| 31.0 | Dense to very dense, light brown SAND with trace gravel; moist, well graded. | 214.0 / 31.0 | | |
| 38.5 | increasing gravel content at 38.5 feet | | | |
| 40 | | | DD | Methane probe #3 set at 40.0 feet |

DRILLED BY: JDK Drilling, Inc. LOGGED BY: WKS COMPLETED: 04/09/07

BORING METHOD: hollow-stem auger (see report text) BORING BIT DIAMETER: 8.0 in

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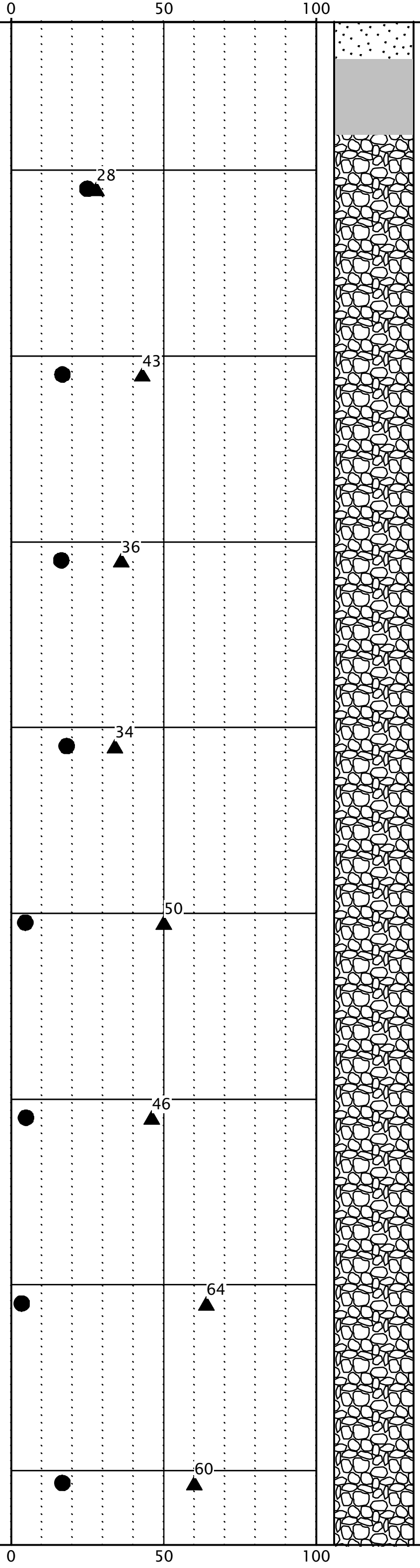
**BORING B-2**

PROPOSED RESIDENTIAL APARTMENT TOWER
LOS ANGELES, CA

**FIGURE A-2**

BORING LOG MERUELO-11-01-BORING_LOGS.GPJ GEODESIGN.GDT PRINT DATE: 5/16/07:KYK

| DEPTH FEET | MATERIAL DESCRIPTION | ELEVATION / DEPTH | TESTING | BLOW COUNT | INSTALLATION AND COMMENTS |
|---|---|---|---|---|---|
| | (continued from previous page) | | | | DD = 114 pcf |
| | Very stiff, gray-brown, silty CLAY; moist. | 202.5 / 42.5 | | | |
| 45 | | | DD<br>CON | 28 | DD = 95 pcf |
| 50 | | | DD | 43 | DD = 110 pcf |
| | Very stiff, gray-brown, fine, sandy SILT; moist. | 192.0 / 53.0 | | | |
| 55 | | | P200<br>DD | 36 | P200 = 51%<br>DD = 111 pcf |
| | Very stiff, gray-brown, silty CLAY; moist. | 186.0 / 59.0 | | | |
| 60 | | | DD<br>DS | 34 | DD = 104 pcf |
| | Very dense, light brown SAND with trace gravel; moist, well graded. | 182.5 / 62.5 | | | |
| 65 | | | DD | 50 | DD = 103 pcf |
| 70 | with 6-inch layer of very stiff, gray-brown, silty clay at 70.0 feet<br>grades to fine sand without gravel at 71.0 feet | | DD<br>DS | 46 | DD = 106 pcf |
| 75 | | | | 64 | |
| | | | | | Water added to facilitate drilling at 77.0 feet. |
| | Very stiff, gray-brown, sandy SILT; moist. | 166.0 / 79.0 | | | |
| 80 | | | DD | 60 | DD = 141 pcf |
| | Very dense, light brown SAND with some gravel; moist, well graded. | 164.5 / 80.5 | | | |



DRILLED BY: JDK Drilling, Inc.
LOGGED BY: WKS
COMPLETED: 04/09/07

BORING METHOD: hollow-stem auger (see report text)
BORING BIT DIAMETER: 8.0 in

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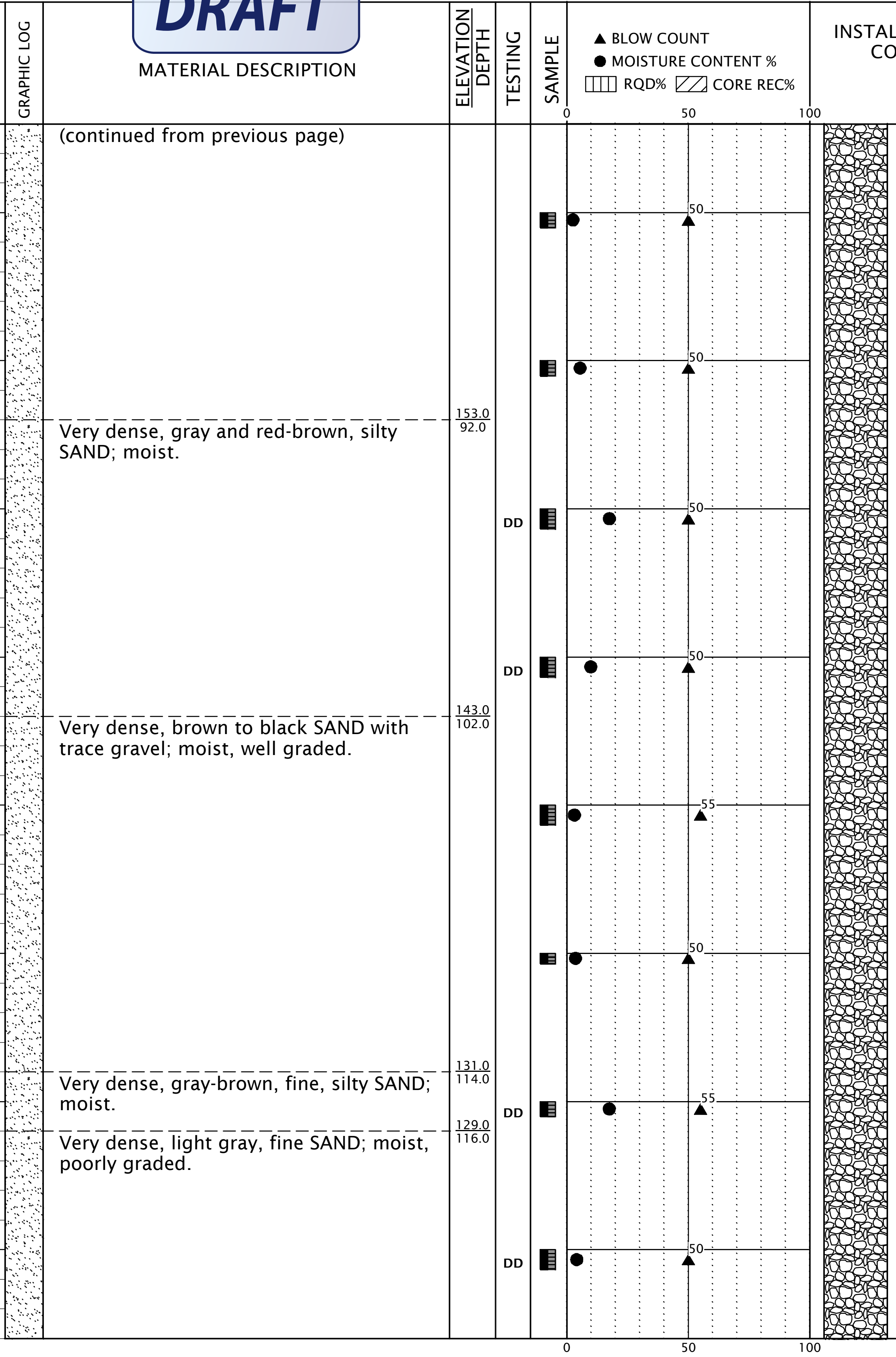
**BORING B-2**
(continued)

PROPOSED RESIDENTIAL APARTMENT TOWER
LOS ANGELES, CA

**FIGURE A-2**

DRAFT

| DEPTH FEET | MATERIAL DESCRIPTION | ELEVATION DEPTH | TESTING | BLOW COUNT | INSTALLATION AND COMMENTS |
|---|---|---|---|---|---|
| | (continued from previous page) | | | | |
| 85 | | | | 50 | |
| 90 | | | | 50 | |
| | Very dense, gray and red-brown, silty SAND; moist. | 153.0<br>92.0 | | | |
| 95 | | | DD | 50 | Water added to facilitate drilling at 95.0 feet.<br>DD = 125 pcf |
| 100 | | | DD | 50 | DD = 127 pcf |
| | Very dense, brown to black SAND with trace gravel; moist, well graded. | 143.0<br>102.0 | | | |
| 105 | | | | 55 | |
| 110 | | | | 50 | Disturbed sample at 110.0 feet. |
| | Very dense, gray-brown, fine, silty SAND; moist. | 131.0<br>114.0 | | | |
| 115 | | | DD | 55 | DD = 131 pcf |
| | Very dense, light gray, fine SAND; moist, poorly graded. | 129.0<br>116.0 | | | |
| 120 | | | DD | 50 | DD = 131 pcf |



DRILLED BY: JDK Drilling, Inc.
LOGGED BY: WKS
COMPLETED: 04/09/07

BORING METHOD: hollow-stem auger (see report text)
BORING BIT DIAMETER: 8.0 in

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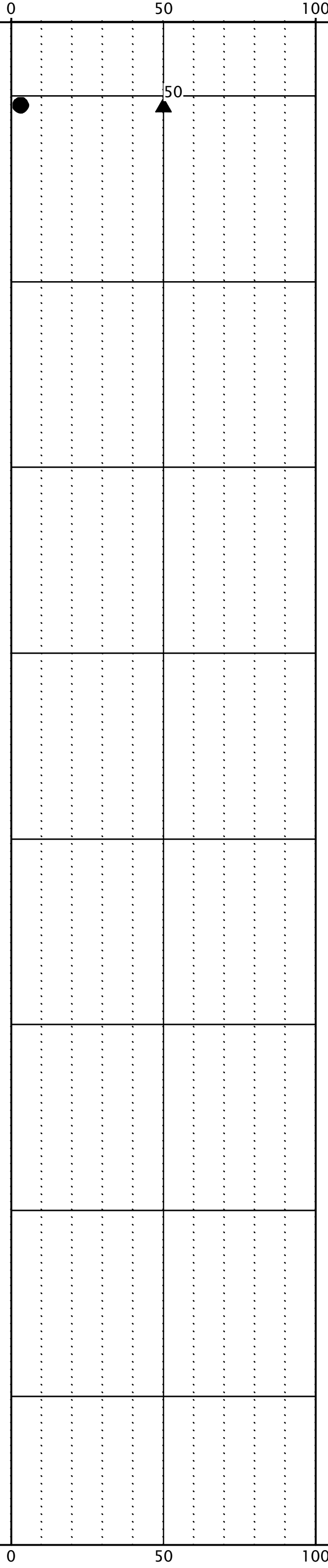
**BORING B-2**
(continued)

PROPOSED RESIDENTIAL APARTMENT TOWER
LOS ANGELES, CA

**FIGURE A-2**

BORING LOG MERUELO-11-01-BORING_LOGS.GPJ GEODESIGN.GDT PRINT DATE: 5/16/07:KYK

| DEPTH FEET | GRAPHIC LOG | MATERIAL DESCRIPTION | ELEVATION DEPTH | TESTING | SAMPLE | ▲ BLOW COUNT<br>● MOISTURE CONTENT %<br>▥ RQD% ▨ CORE REC% | INSTALLATION AND COMMENTS |
|---|---|---|---|---|---|---|---|
| | | (continued from previous page) | | | | | |
| 125 | | **Exploration completed at 125.5 feet.**<br>**Groundwater not encountered.**<br>**Methane probes installed at depths of 25.0, 30.0, and 40.0 feet upon completion of drilling.** | 119.5<br>125.5 | DD | ▤ | 50 | DD = 134 pcf<br>All samplers driven with 140-lb. hammer weight falling 30 inches. |
| 130 | | | | | | | |
| 135 | | | | | | | |
| 140 | | | | | | | |
| 145 | | | | | | | |
| 150 | | | | | | | |
| 155 | | | | | | | |
| 160 | | | | | | | |



DRILLED BY: JDK Drilling, Inc.

LOGGED BY: WKS

COMPLETED: 04/09/07

BORING METHOD: hollow-stem auger (see report text)

BORING BIT DIAMETER: 8.0 in

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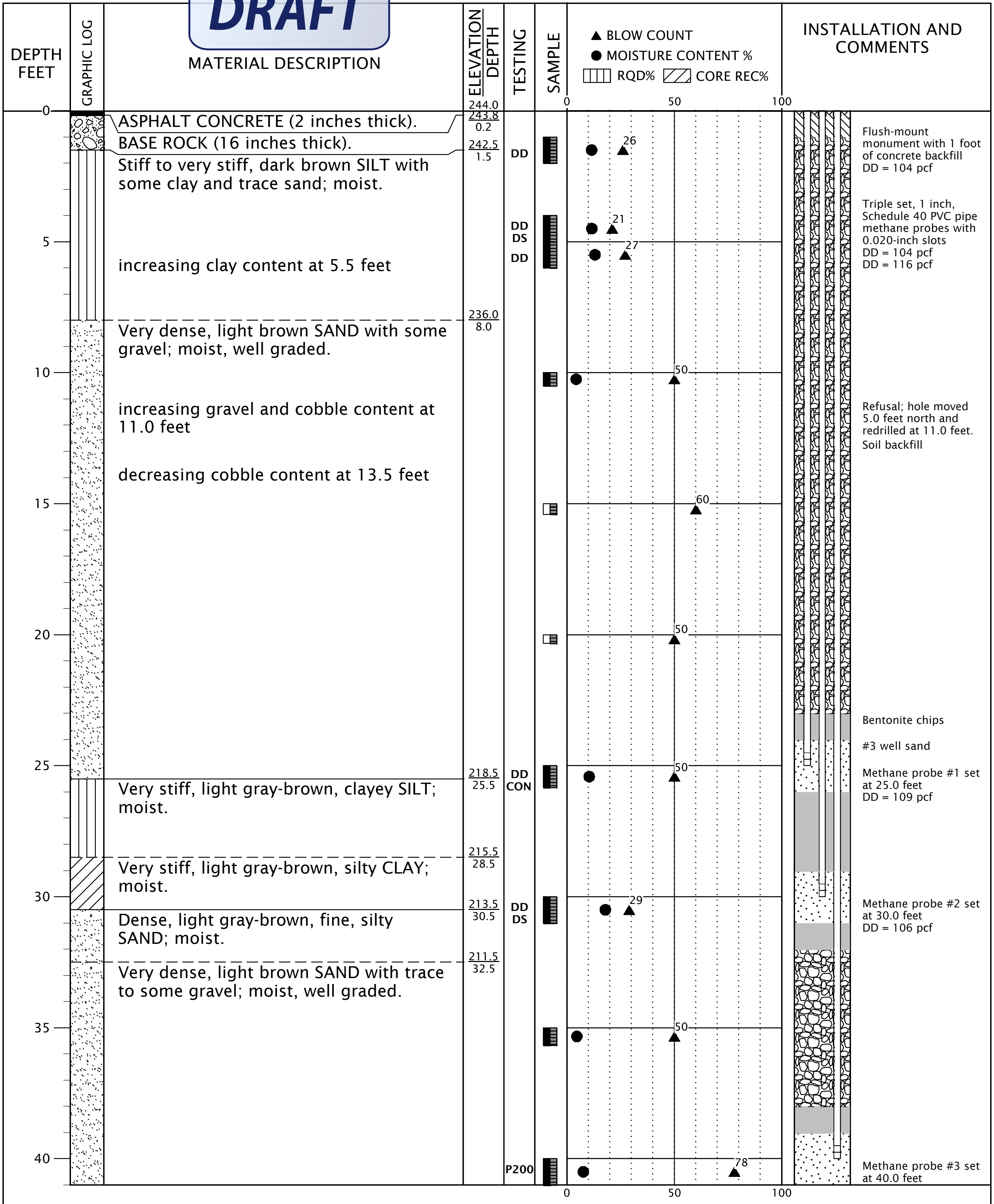
**BORING B-2**
(continued)

PROPOSED RESIDENTIAL APARTMENT TOWER
LOS ANGELES, CA

**FIGURE A-2**



| ELEVATION / DEPTH | MATERIAL DESCRIPTION | TESTING |
|---|---|---|
| 244.0 | | |
| 243.8<br>0.2 | ASPHALT CONCRETE (2 inches thick). | |
| 242.5<br>1.5 | BASE ROCK (16 inches thick). | |
| | Stiff to very stiff, dark brown SILT with some clay and trace sand; moist. | DD<br>DD<br>DS<br>DD |
| | increasing clay content at 5.5 feet | |
| 236.0<br>8.0 | Very dense, light brown SAND with some gravel; moist, well graded. | |
| | increasing gravel and cobble content at 11.0 feet | |
| | decreasing cobble content at 13.5 feet | |
| 218.5<br>25.5 | Very stiff, light gray-brown, clayey SILT; moist. | DD<br>CON |
| 215.5<br>28.5 | Very stiff, light gray-brown, silty CLAY; moist. | |
| 213.5<br>30.5 | Dense, light gray-brown, fine, silty SAND; moist. | DD<br>DS |
| 211.5<br>32.5 | Very dense, light brown SAND with trace to some gravel; moist, well graded. | P200 |

INSTALLATION AND COMMENTS

Flush-mount monument with 1 foot of concrete backfill
DD = 104 pcf

Triple set, 1 inch, Schedule 40 PVC pipe methane probes with 0.020-inch slots
DD = 104 pcf
DD = 116 pcf

Refusal; hole moved 5.0 feet north and redrilled at 11.0 feet.
Soil backfill

Bentonite chips

#3 well sand

Methane probe #1 set at 25.0 feet
DD = 109 pcf

Methane probe #2 set at 30.0 feet
DD = 106 pcf

Methane probe #3 set at 40.0 feet

DRILLED BY: JDK Drilling, Inc.    LOGGED BY: WKS/DTW    COMPLETED: 04/10/07

BORING METHOD: hollow-stem auger (see report text)    BORING BIT DIAMETER: 8.0 in

BORING LOG MERUELO-11-01-BORING_LOGS.GPJ GEODESIGN.GDT PRINT DATE: 5/16/07:KYK

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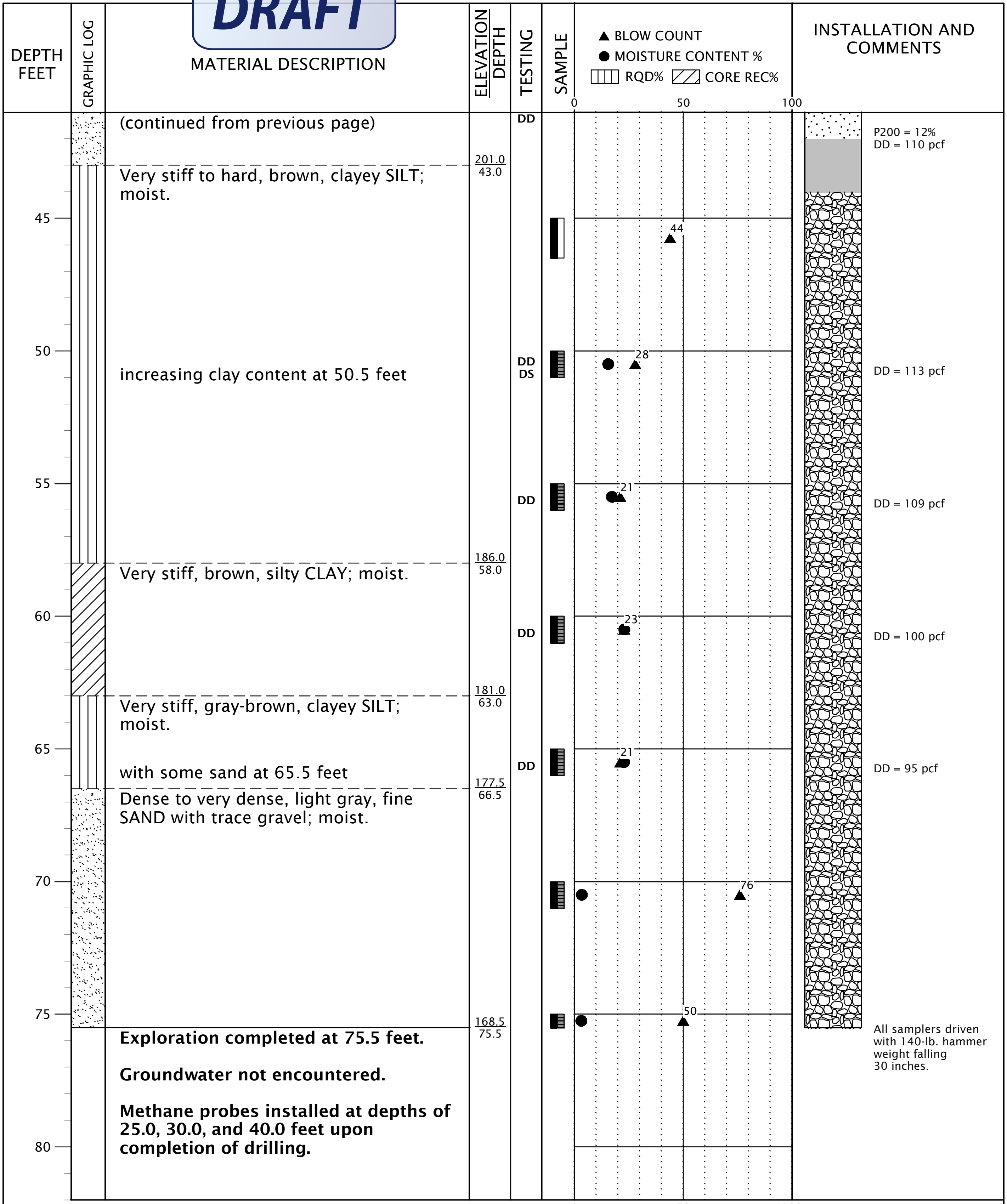
**BORING B-3**

PROPOSED RESIDENTIAL APARTMENT TOWER
LOS ANGELES, CA

**FIGURE A-3**



| DEPTH FEET | MATERIAL DESCRIPTION | ELEVATION DEPTH | TESTING | ▲ BLOW COUNT | INSTALLATION AND COMMENTS |
|---|---|---|---|---|---|
| | (continued from previous page) | | DD | | P200 = 12%<br>DD = 110 pcf |
| | Very stiff to hard, brown, clayey SILT; moist. | 201.0<br>43.0 | | | |
| 45 | | | | 44 | |
| 50 | increasing clay content at 50.5 feet | | DD<br>DS | 28 | DD = 113 pcf |
| 55 | | | DD | 21 | DD = 109 pcf |
| | Very stiff, brown, silty CLAY; moist. | 186.0<br>58.0 | | | |
| 60 | | | DD | 23 | DD = 100 pcf |
| | Very stiff, gray-brown, clayey SILT; moist. | 181.0<br>63.0 | | | |
| 65 | with some sand at 65.5 feet | | DD | 21 | DD = 95 pcf |
| | Dense to very dense, light gray, fine SAND with trace gravel; moist. | 177.5<br>66.5 | | | |
| 70 | | | | 76 | |
| 75 | | | | 50 | All samplers driven with 140-lb. hammer weight falling 30 inches. |
| | **Exploration completed at 75.5 feet.**<br>**Groundwater not encountered.**<br>**Methane probes installed at depths of 25.0, 30.0, and 40.0 feet upon completion of drilling.** | 168.5<br>75.5 | | | |
| 80 | | | | | |

DRILLED BY: JDK Drilling, Inc.

LOGGED BY: WKS/DTW

COMPLETED: 04/10/07

BORING METHOD: hollow-stem auger (see report text)

BORING BIT DIAMETER: 8.0 in

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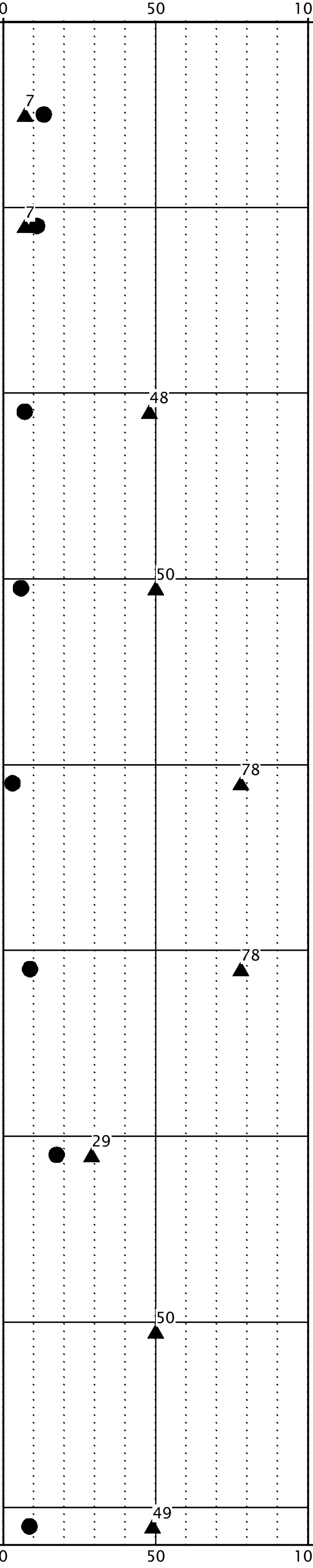
**BORING B-3**
(continued)

PROPOSED RESIDENTIAL APARTMENT TOWER
LOS ANGELES, CA

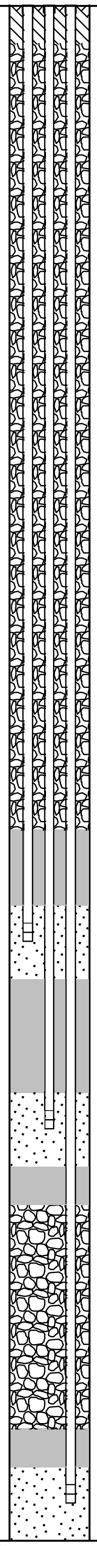
**FIGURE A-3**

| DEPTH FEET | MATERIAL DESCRIPTION | ELEVATION DEPTH | TESTING | BLOW COUNT |
|---|---|---|---|---|
| 0 | | 244.0 | | |
| | ASPHALT CONCRETE (3 inches thick). | 243.8<br>0.3 | | |
| | CONCRETE (4 inches thick). | 243.4<br>0.6 | | |
| | Medium stiff, dark brown, sandy SILT; moist - **FILL**. | | DD | 7 |
| 5 | with lenses and layers of silty sand with some decayed vegetation fragments at 5.0 feet | | DD | 7 |
| | Dense, brown, silty SAND with trace to some gravel; moist. | 235.0<br>9.0 | | |
| 10 | | | DS | 48 |
| | Very dense, brown SAND with some gravel; moist, well graded. | 232.0<br>12.0 | | |
| 15 | | | | 50 |
| 20 | | | | 78 |
| 25 | | | | 78 |
| | increasing moisture at 26.0 feet | | | |
| | Dense, gray-brown, fine, silty SAND; moist. | 216.0<br>28.0 | | |
| 30 | | | | |
| | Very stiff, gray-brown CLAY; moist. | 213.5<br>30.5 | DD | 29 |
| | Dense, light brown SAND with trace to some gravel; moist, well graded. | 210.0<br>34.0 | | |
| 35 | | | | 50 |
| 40 | | | DD | 49 |

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



INSTALLATION AND COMMENTS



Flush-mount monument with 1 foot of concrete backfill

DD = 104 pcf
Triple set, 1 inch, Schedule 40 PVC pipe methane probes with 0.020-inch slots

DD = 99 pcf

Soil backfill

Bentonite chips

#3 well sand

Methane probe #1 set at 25.0 feet

Methane probe #2 set at 30.0 feet
DD = 107 pcf

Disturbed sample at 35.0 feet.

Methane probe #3 set at 40.0 feet

DRILLED BY: JDK Drilling, Inc.

LOGGED BY: WKS

COMPLETED: 04/10/07

BORING METHOD: hollow-stem auger (see report text)

BORING BIT DIAMETER: 8.0 in

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MERUELO-11-01

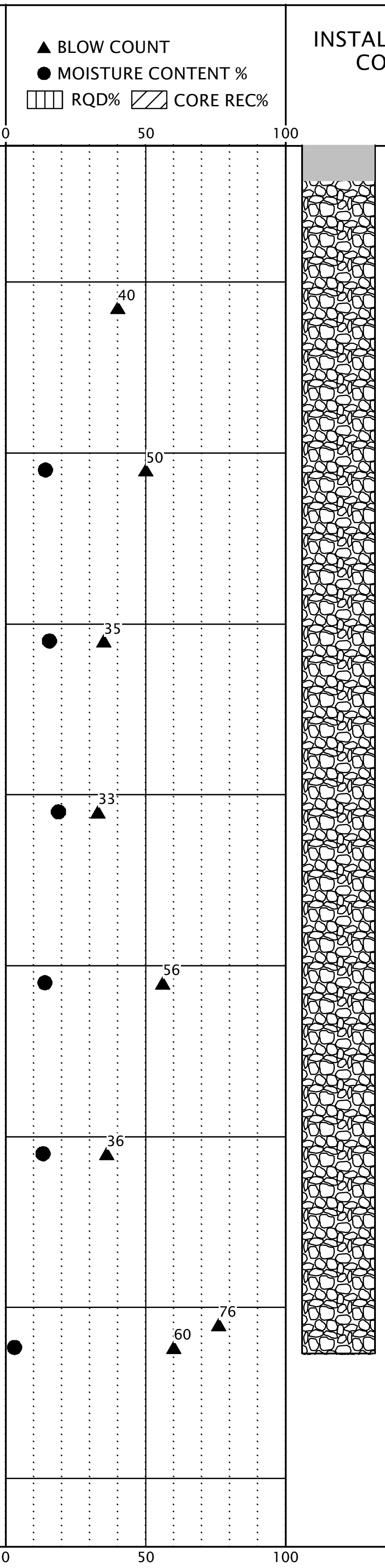
MAY 2007

**BORING B-4**

PROPOSED RESIDENTIAL APARTMENT TOWER
LOS ANGELES, CA

**FIGURE A-4**

| DEPTH FEET | MATERIAL DESCRIPTION | ELEVATION DEPTH | TESTING | SAMPLE | INSTALLATION AND COMMENTS |
|---|---|---|---|---|---|
| | (continued from previous page) | | CON | | DD = 109 pcf |
| 45 | Hard, gray-brown, silty CLAY; moist. | 198.5<br>45.5 | | ▮ | |
| 50 | Very stiff, gray-brown, sandy SILT; moist. | 195.0<br>49.0 | DD | ▮ | DD = 116 pcf |
| 55 | Medium dense to dense, gray-brown, fine, clayey SAND; moist. | 191.0<br>53.0 | DD | ▮ | DD = 111 pcf |
| 60 | Very stiff, gray-brown, silty CLAY; moist.<br>with some sand at 61.0 feet | 187.5<br>56.5 | DD | ▮ | DD = 134 pcf |
| 65 | Very stiff, gray-brown, sandy SILT; moist. | 182.0<br>62.0 | DD | ▮ | DD = 136 pcf |
| 70 | Dense, light brown and gray, fine, silty SAND; moist. | 176.0<br>68.0 | DD | ▮ | DD = 143 pcf |
| 75 | Very dense to dense, light brown SAND with trace gravel; moist, well graded. | 173.0<br>71.0 | | ▮ | All samplers driven with 140-lb. hammer weight falling 30 inches. |
| 80 | **Exploration completed at 76.3 feet.**<br>**Groundwater not encountered.**<br>**Methane probes installed at depths of 25.0, 30.0, and 40.0 feet upon completion of drilling.** | 167.7<br>76.3 | | | |



DRILLED BY: JDK Drilling, Inc.

LOGGED BY: WKS

COMPLETED: 04/10/07

BORING METHOD: hollow-stem auger (see report text)

BORING BIT DIAMETER: 8.0 in

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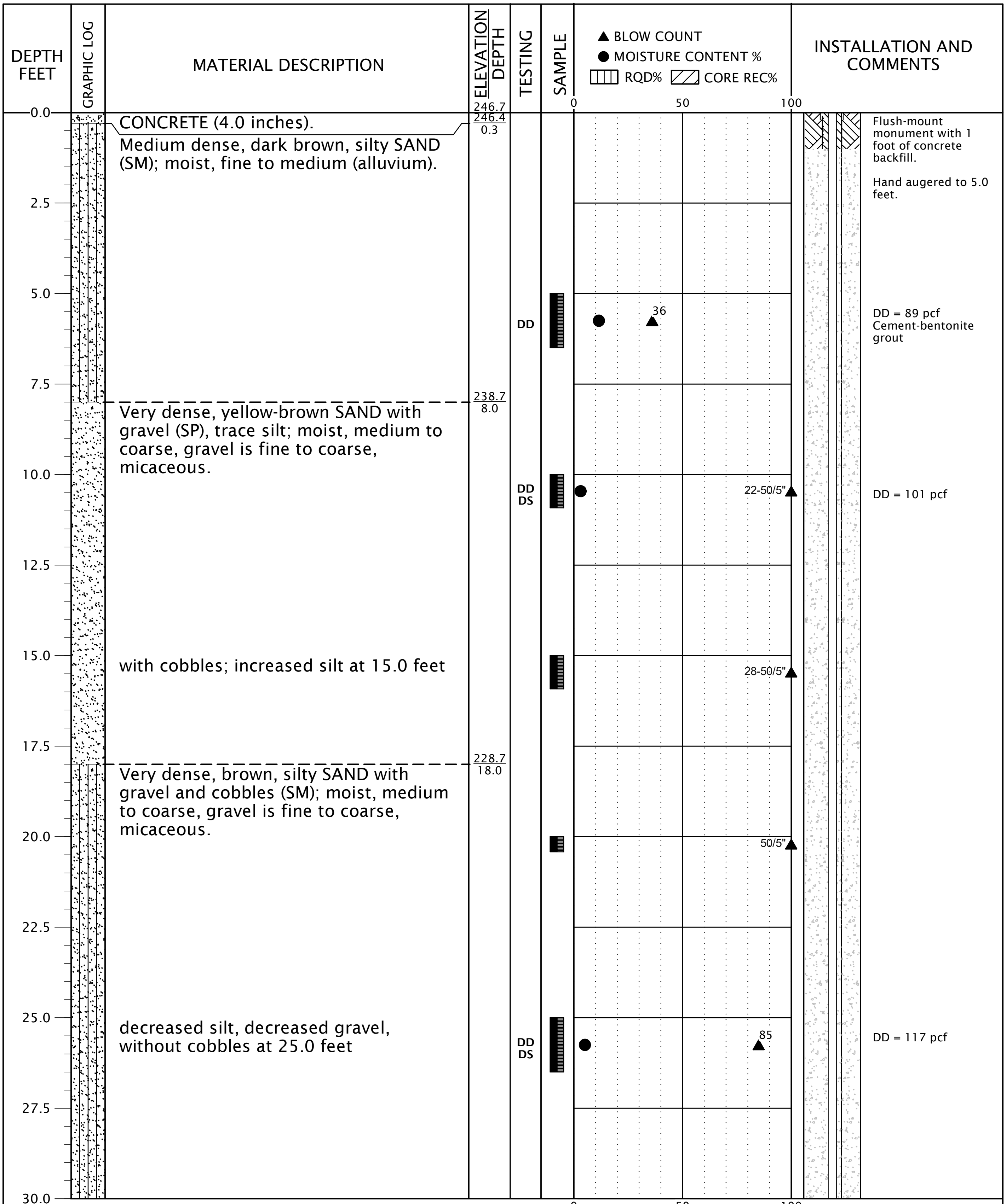
MAY 2007

**BORING B-4**
(continued)

PROPOSED RESIDENTIAL APARTMENT TOWER
LOS ANGELES, CA

**FIGURE A-4**



| DEPTH FEET | MATERIAL DESCRIPTION | ELEVATION DEPTH | TESTING | BLOW COUNT | INSTALLATION AND COMMENTS |
|---|---|---|---|---|---|
| 0.0 | CONCRETE (4.0 inches). | 246.7<br>246.4<br>0.3 | | | Flush-mount monument with 1 foot of concrete backfill.<br>Hand augered to 5.0 feet. |
| | Medium dense, dark brown, silty SAND (SM); moist, fine to medium (alluvium). | | | | |
| 5.0 | | | DD | 36 | DD = 89 pcf<br>Cement-bentonite grout |
| 8.0 | Very dense, yellow-brown SAND with gravel (SP), trace silt; moist, medium to coarse, gravel is fine to coarse, micaceous. | 238.7<br>8.0 | | | |
| 10.0 | | | DD<br>DS | 22-50/5" | DD = 101 pcf |
| 15.0 | with cobbles; increased silt at 15.0 feet | | | 28-50/5" | |
| 18.0 | Very dense, brown, silty SAND with gravel and cobbles (SM); moist, medium to coarse, gravel is fine to coarse, micaceous. | 228.7<br>18.0 | | | |
| 20.0 | | | | 50/5" | |
| 25.0 | decreased silt, decreased gravel, without cobbles at 25.0 feet | | DD<br>DS | 85 | DD = 117 pcf |

DRILLED BY: Martini Drilling LOGGED BY: AJA COMPLETED: 12/04/17

BORING METHOD: hollow-stem auger (see document text) BORING BIT DIAMETER: 8 inches

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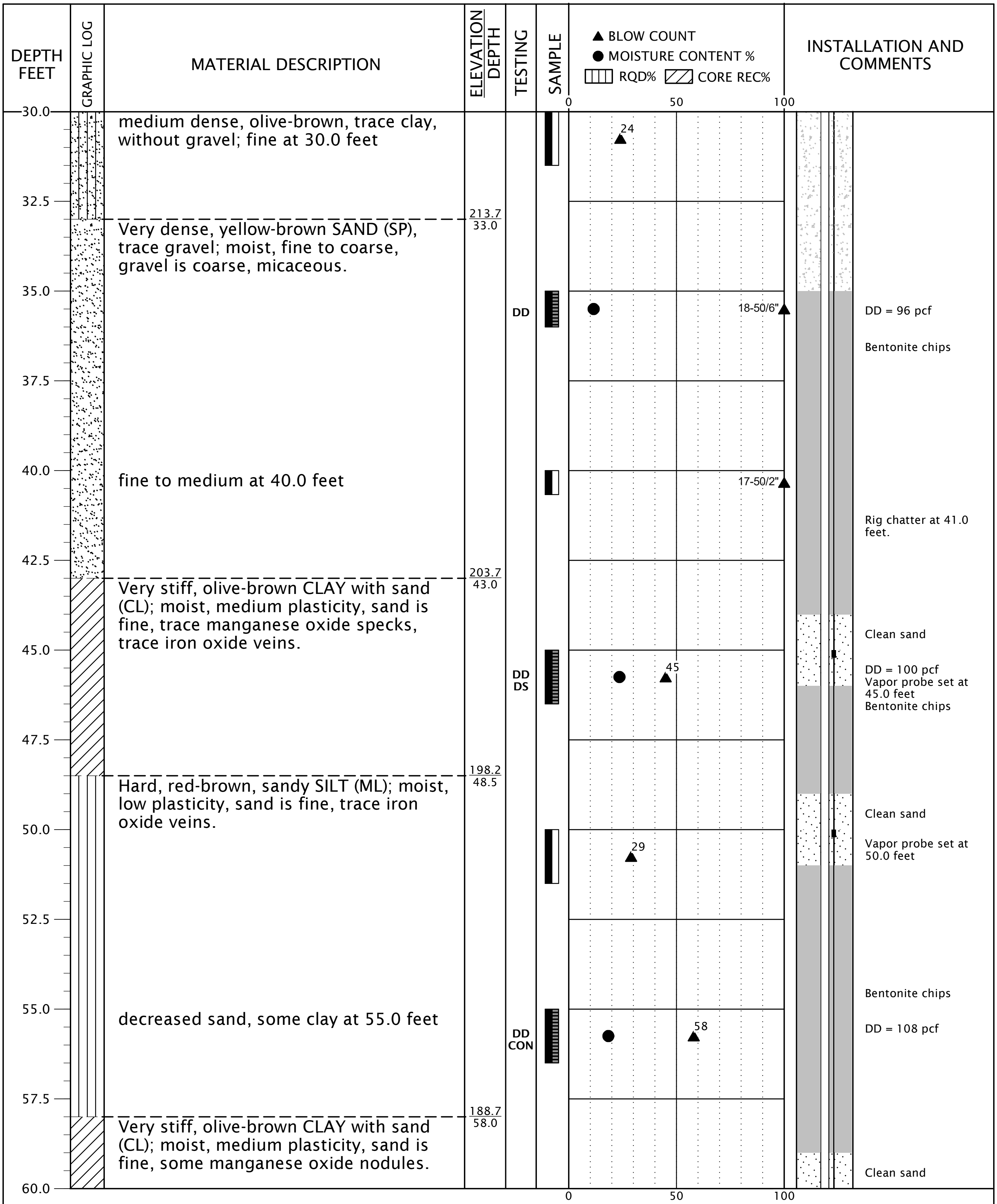
DTLASITE2-1-01

JUNE 2018

**BORING B-5**

PROPOSED HIGH-RISE TOWER - SITE 2
LOS ANGELES, CA

**FIGURE A-1**



| DEPTH FEET | MATERIAL DESCRIPTION | ELEVATION DEPTH | TESTING | BLOW COUNT | INSTALLATION AND COMMENTS |
|---|---|---|---|---|---|
| 30.0 | medium dense, olive-brown, trace clay, without gravel; fine at 30.0 feet | | | 24 | |
| 33.0 | Very dense, yellow-brown SAND (SP), trace gravel; moist, fine to coarse, gravel is coarse, micaceous. | 213.7<br>33.0 | | | |
| 35.0 | | | DD | 18-50/6" | DD = 96 pcf<br>Bentonite chips |
| 40.0 | fine to medium at 40.0 feet | | | 17-50/2" | Rig chatter at 41.0 feet. |
| 43.0 | Very stiff, olive-brown CLAY with sand (CL); moist, medium plasticity, sand is fine, trace manganese oxide specks, trace iron oxide veins. | 203.7<br>43.0 | | | Clean sand |
| 45.0 | | | DD<br>DS | 45 | DD = 100 pcf<br>Vapor probe set at 45.0 feet<br>Bentonite chips |
| 48.5 | Hard, red-brown, sandy SILT (ML); moist, low plasticity, sand is fine, trace iron oxide veins. | 198.2<br>48.5 | | | Clean sand |
| 50.0 | | | | 29 | Vapor probe set at 50.0 feet |
| 55.0 | decreased sand, some clay at 55.0 feet | | DD<br>CON | 58 | Bentonite chips<br>DD = 108 pcf |
| 58.0 | Very stiff, olive-brown CLAY with sand (CL); moist, medium plasticity, sand is fine, some manganese oxide nodules. | 188.7<br>58.0 | | | Clean sand |

DRILLED BY: Martini Drilling LOGGED BY: AJA COMPLETED: 12/04/17

BORING METHOD: hollow-stem auger (see document text) BORING BIT DIAMETER: 8 inches

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JUNE 2018

**BORING B-5**
(continued)

PROPOSED HIGH-RISE TOWER - SITE 2
LOS ANGELES, CA

**FIGURE A-1**

| DEPTH FEET | MATERIAL DESCRIPTION | ELEVATION DEPTH | TESTING | ▲ BLOW COUNT | INSTALLATION AND COMMENTS |
|---|---|---|---|---|---|
| 60.0 | (continued from previous page) | | | 23 | Vapor probe set at 60.0 feet |
| 62.5 | Hard, pale yellow-brown, sandy SILT (ML); moist, nonplastic, sand is fine to medium, micaceous. | 184.2<br>62.5 | | | |
| 65.0 | | | DD | 55 | DD = 86 pcf<br>Bentonite chips |
| 68.5 | Very dense, yellow-brown, silty SAND (SM); moist, fine to medium. | 178.2<br>68.5 | | | |
| 70.0 | | | | 11-50/6" | |
| 71.0 | Very dense, light brown SAND (SP), trace gravel; moist, medium to coarse, gravel is fine, micaceous. | 175.7<br>71.0 | | | Grades to sand in tip of sampler at 71.0 feet.<br>Some rig chatter at 72.0 feet. |
| 75.0 | | | | 50/6" | |
| 75.5 | **Exploration completed at a depth of 75.5 feet.**<br>**No groundwater encountered. Boring converted into methane monitoring well.** | 171.2<br>75.5 | | | Percolation test at 75.5 feet. |

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

DRILLED BY: Martini Drilling LOGGED BY: AJA COMPLETED: 12/04/17

BORING METHOD: hollow-stem auger (see document text) BORING BIT DIAMETER: 8 inches

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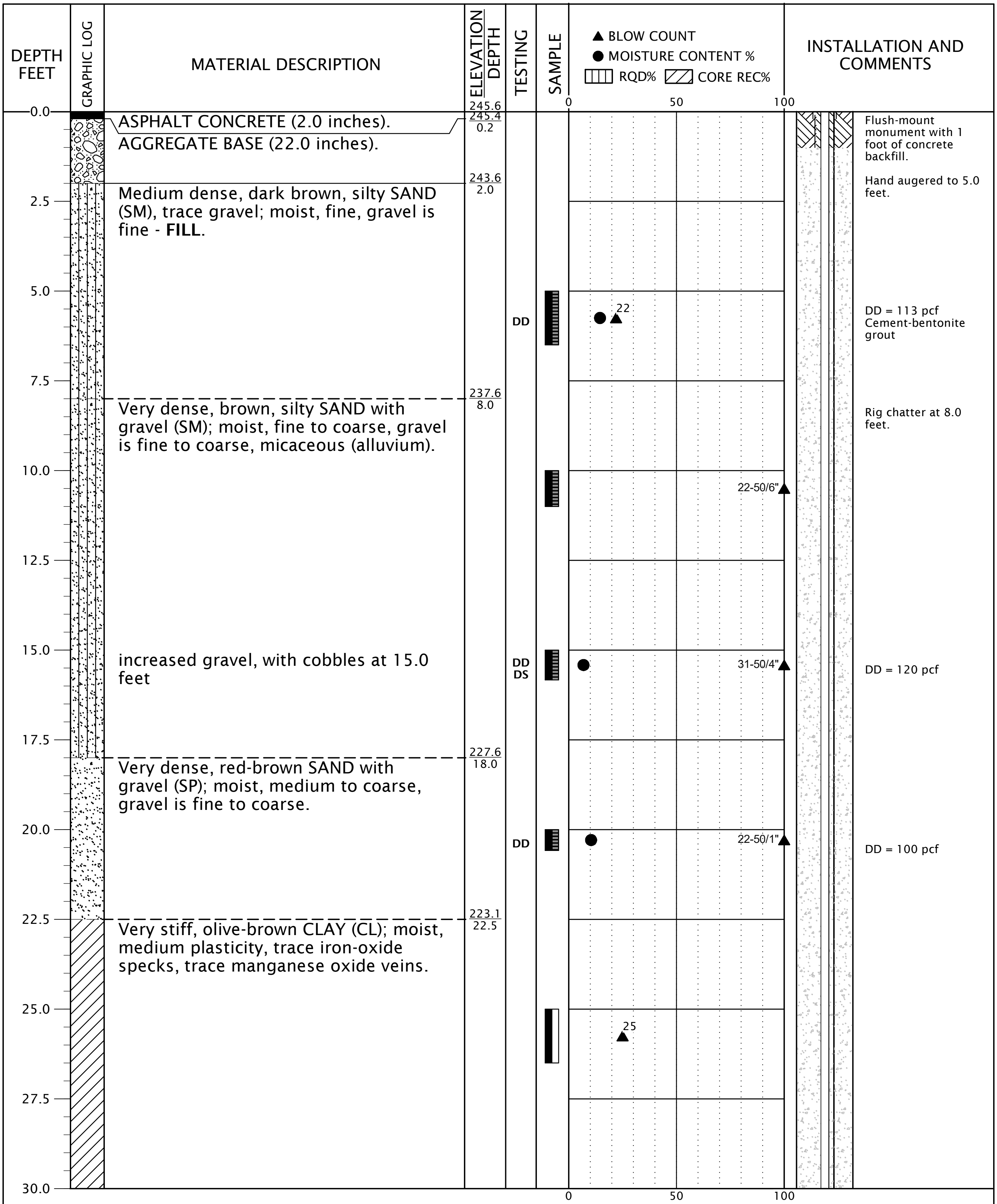
**BORING B-5**
(continued)

PROPOSED HIGH-RISE TOWER - SITE 2
LOS ANGELES, CA

**FIGURE A-1**



| DEPTH FEET | MATERIAL DESCRIPTION | ELEVATION DEPTH | TESTING | BLOW COUNT | INSTALLATION AND COMMENTS |
|---|---|---|---|---|---|
| 0.0 | ASPHALT CONCRETE (2.0 inches). | 245.6 |  |  |  |
|  | AGGREGATE BASE (22.0 inches). | 245.4<br>0.2 |  |  | Flush-mount monument with 1 foot of concrete backfill.<br>Hand augered to 5.0 feet. |
| 2.0 | Medium dense, dark brown, silty SAND (SM), trace gravel; moist, fine, gravel is fine - **FILL**. | 243.6<br>2.0 |  |  |  |
| 5.0 |  |  | DD | 22 | DD = 113 pcf<br>Cement-bentonite grout |
| 8.0 | Very dense, brown, silty SAND with gravel (SM); moist, fine to coarse, gravel is fine to coarse, micaceous (alluvium). | 237.6<br>8.0 |  |  | Rig chatter at 8.0 feet. |
| 10.0 |  |  |  | 22-50/6" |  |
| 15.0 | increased gravel, with cobbles at 15.0 feet |  | DD<br>DS | 31-50/4" | DD = 120 pcf |
| 18.0 | Very dense, red-brown SAND with gravel (SP); moist, medium to coarse, gravel is fine to coarse. | 227.6<br>18.0 |  |  |  |
| 20.0 |  |  | DD | 22-50/1" | DD = 100 pcf |
| 22.5 | Very stiff, olive-brown CLAY (CL); moist, medium plasticity, trace iron-oxide specks, trace manganese oxide veins. | 223.1<br>22.5 |  |  |  |
| 25.0 |  |  |  | 25 |  |
| 30.0 |  |  |  |  |  |

DRILLED BY: Martini Drilling

LOGGED BY: AJA

COMPLETED: 12/01/17

BORING METHOD: hollow-stem auger (see document text)

BORING BIT DIAMETER: 8 inches

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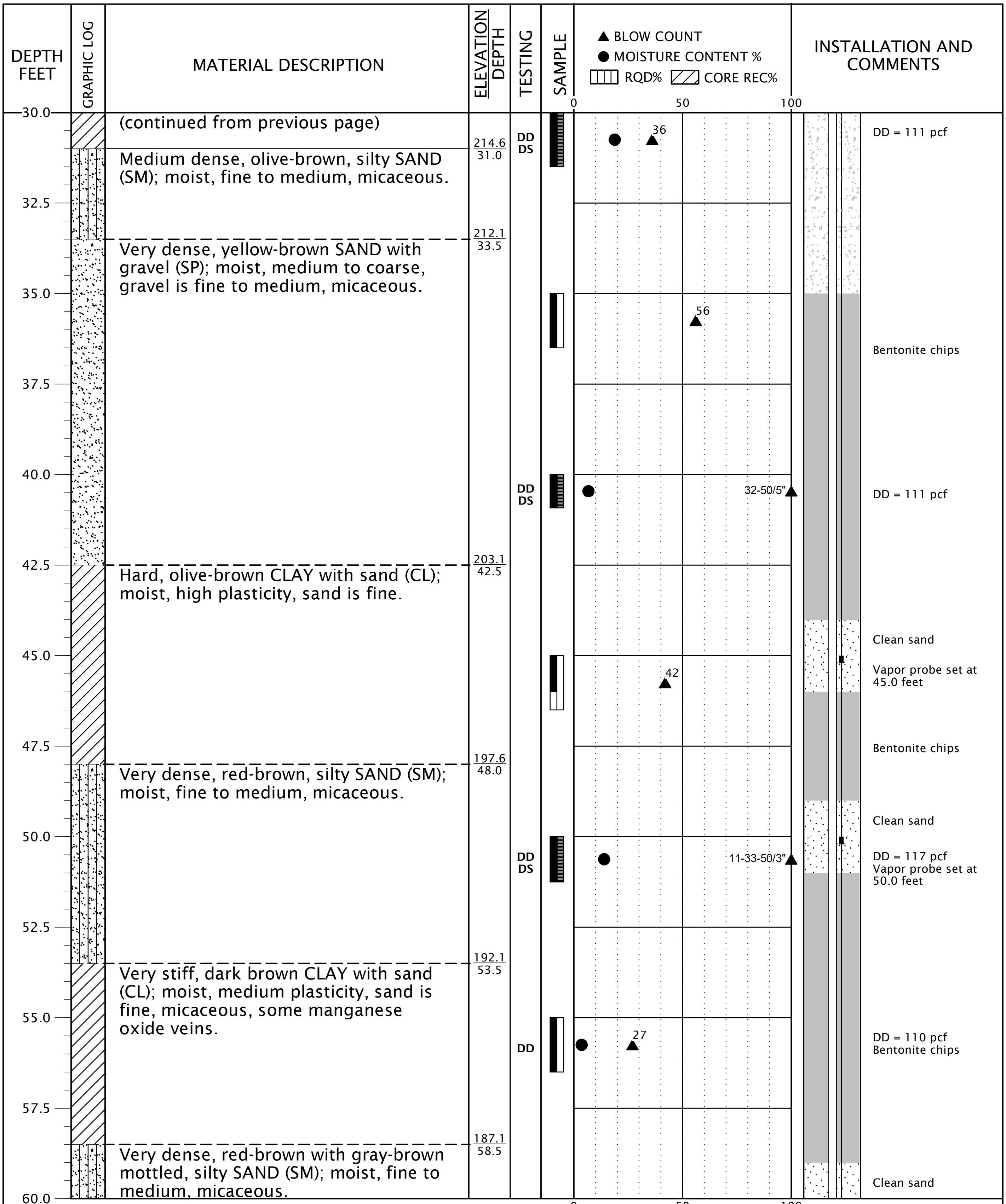
**BORING B-6**

PROPOSED HIGH-RISE TOWER - SITE 2
LOS ANGELES, CA

**FIGURE A-2**



| MATERIAL DESCRIPTION | ELEVATION<br>DEPTH | TESTING | INSTALLATION AND COMMENTS |
|---|---|---|---|
| (continued from previous page) | | DD<br>DS | DD = 111 pcf |
| Medium dense, olive-brown, silty SAND (SM); moist, fine to medium, micaceous. | 214.6<br>31.0 | | |
| Very dense, yellow-brown SAND with gravel (SP); moist, medium to coarse, gravel is fine to medium, micaceous. | 212.1<br>33.5 | DD<br>DS | Bentonite chips<br>DD = 111 pcf |
| Hard, olive-brown CLAY with sand (CL); moist, high plasticity, sand is fine. | 203.1<br>42.5 | | Clean sand<br>Vapor probe set at 45.0 feet<br>Bentonite chips |
| Very dense, red-brown, silty SAND (SM); moist, fine to medium, micaceous. | 197.6<br>48.0 | DD<br>DS | Clean sand<br>DD = 117 pcf<br>Vapor probe set at 50.0 feet |
| Very stiff, dark brown CLAY with sand (CL); moist, medium plasticity, sand is fine, micaceous, some manganese oxide veins. | 192.1<br>53.5 | DD | DD = 110 pcf<br>Bentonite chips |
| Very dense, red-brown with gray-brown mottled, silty SAND (SM); moist, fine to medium, micaceous. | 187.1<br>58.5 | | Clean sand |

DRILLED BY: Martini Drilling LOGGED BY: AJA COMPLETED: 12/01/17

BORING METHOD: hollow-stem auger (see document text) BORING BIT DIAMETER: 8 inches

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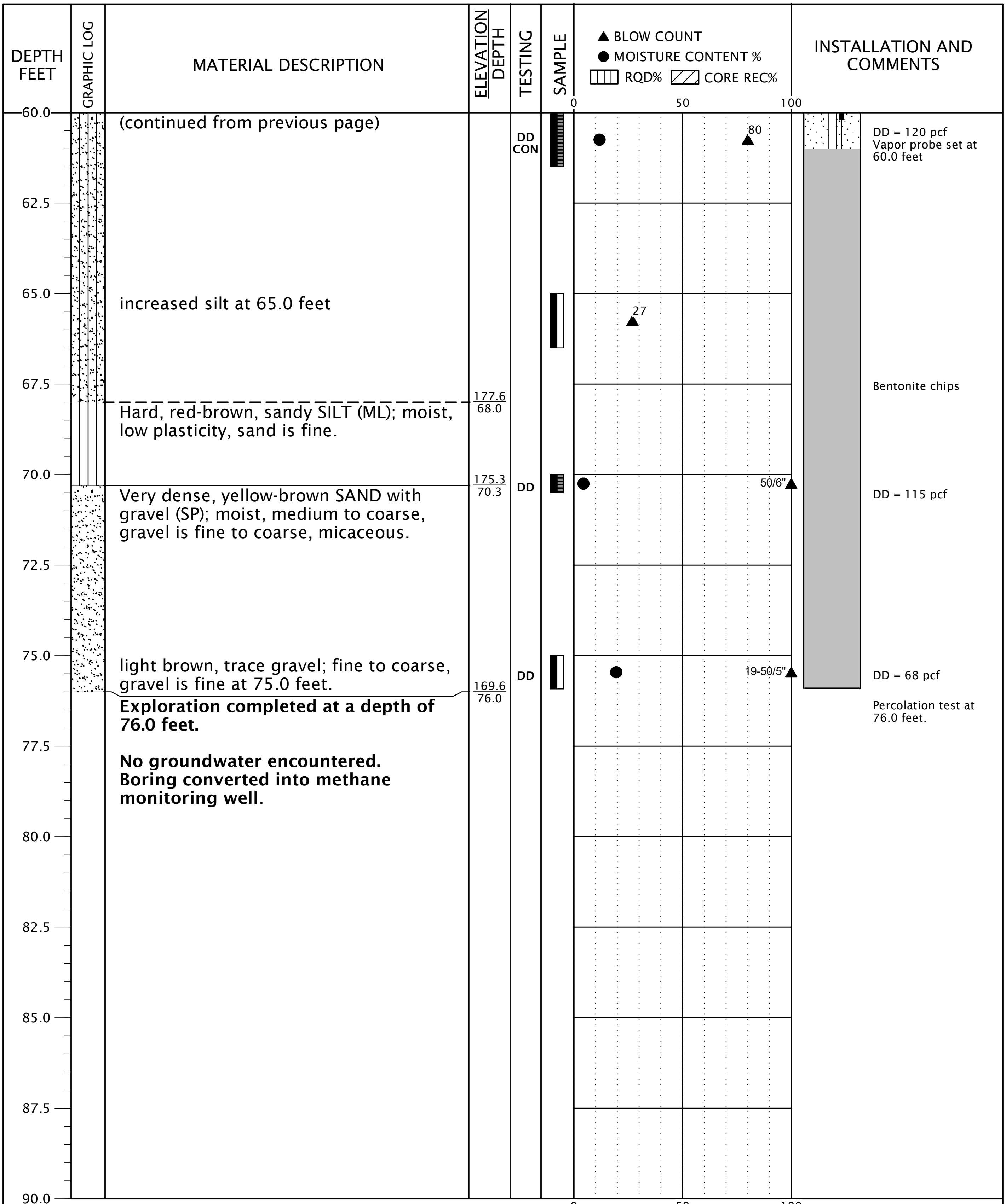
**BORING B-6**
(continued)

PROPOSED HIGH-RISE TOWER - SITE 2
LOS ANGELES, CA

**FIGURE A-2**



| DEPTH FEET | MATERIAL DESCRIPTION | ELEVATION DEPTH | TESTING | BLOW COUNT | INSTALLATION AND COMMENTS |
|---|---|---|---|---|---|
| 60.0 | (continued from previous page) | | DD<br>CON | 80 | DD = 120 pcf<br>Vapor probe set at 60.0 feet |
| 65.0 | increased silt at 65.0 feet | | | 27 | |
| 67.5 | | | | | Bentonite chips |
| | Hard, red-brown, sandy SILT (ML); moist, low plasticity, sand is fine. | 177.6<br>68.0 | | | |
| 70.0 | Very dense, yellow-brown SAND with gravel (SP); moist, medium to coarse, gravel is fine to coarse, micaceous. | 175.3<br>70.3 | DD | 50/6" | DD = 115 pcf |
| 75.0 | light brown, trace gravel; fine to coarse, gravel is fine at 75.0 feet. | | DD | 19-50/5" | DD = 68 pcf |
| | **Exploration completed at a depth of 76.0 feet.**<br>**No groundwater encountered. Boring converted into methane monitoring well.** | 169.6<br>76.0 | | | Percolation test at 76.0 feet. |

DRILLED BY: Martini Drilling

LOGGED BY: AJA

COMPLETED: 12/01/17

BORING METHOD: hollow-stem auger (see document text)

BORING BIT DIAMETER: 8 inches

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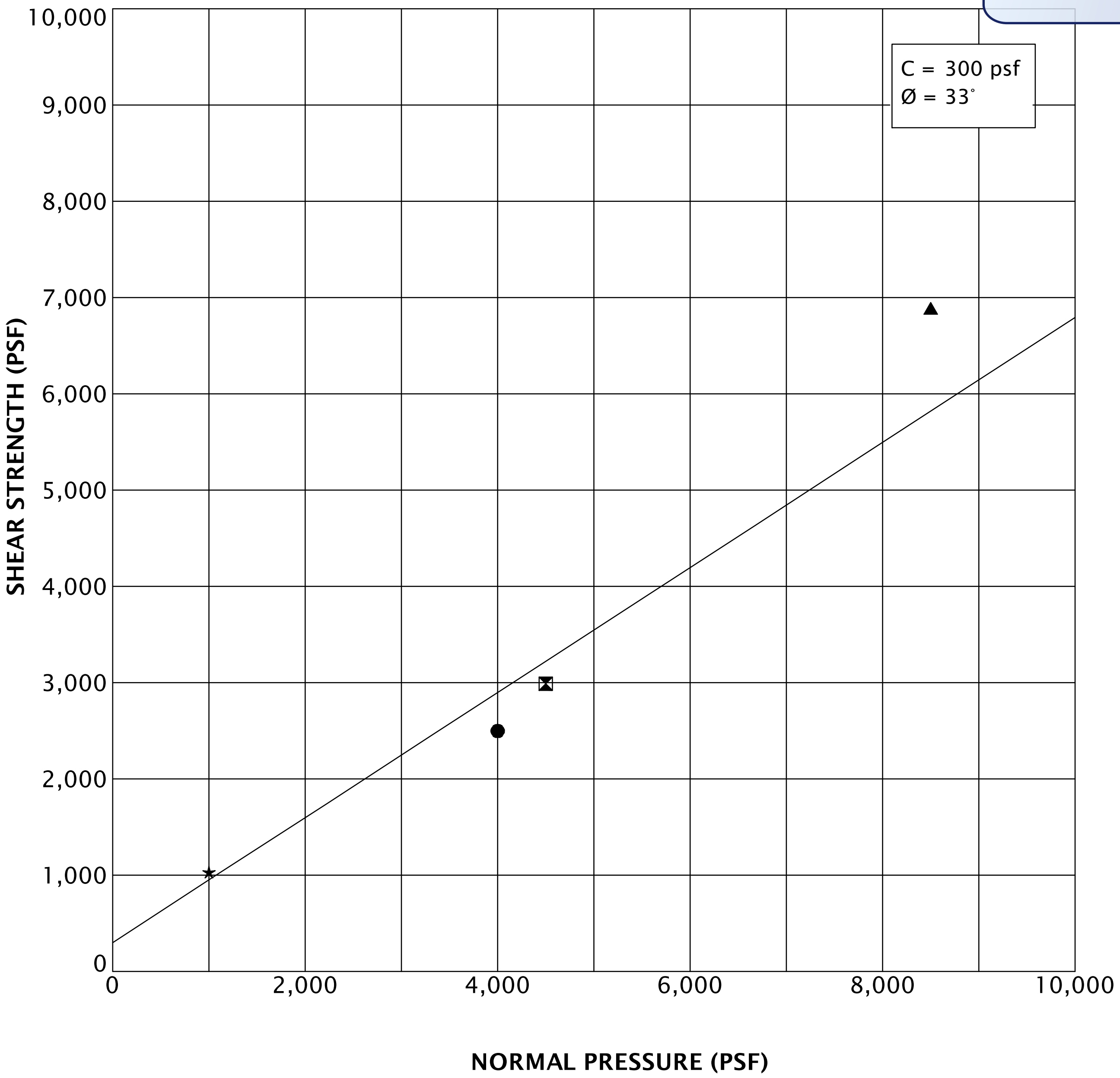
## BORING B-6
(continued)

PROPOSED HIGH-RISE TOWER - SITE 2
LOS ANGELES, CA

**FIGURE A-2**

DRAFT



| KEY | EXPLORATION NUMBER | SAMPLE DEPTH (FEET) | USCS | MOISTURE CONTENT (PERCENT) | DRY DENSITY (PCF) | SOAKED |
|---|---|---|---|---|---|---|
| ● | B-1 | 34.0 | SP | 5.8 | 95 | YES |
| ☒ | B-2 | 25.0 | SW | 7.2 | 104 | YES |
| ▲ | B-2 | 70.0 | SW | 4.8 | 106 | YES |
| ★ | B-4 | 10.0 | SW | 4.6 | 109 | YES |

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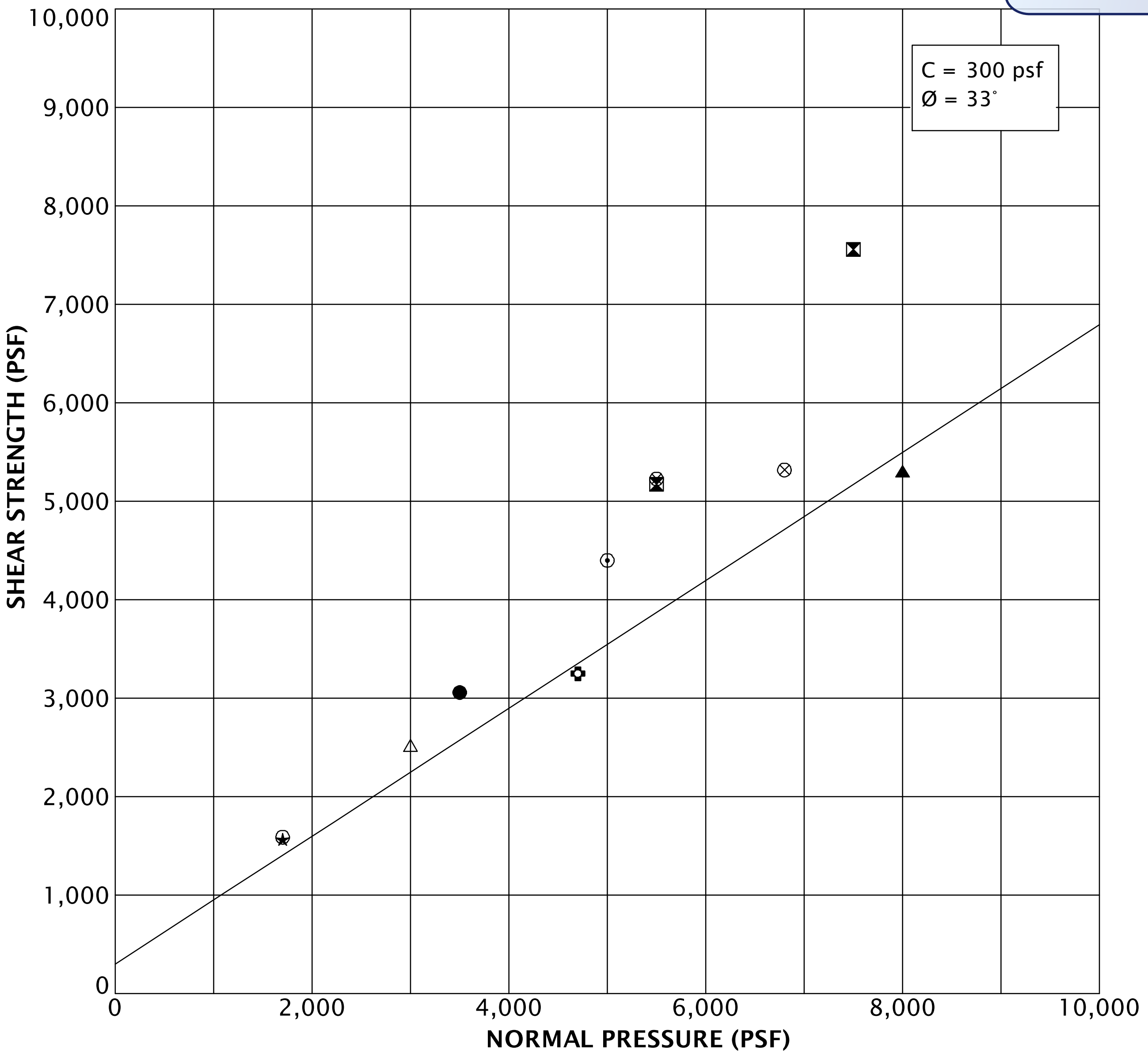
**DIRECT SHEAR TEST RESULTS (COARSE-GRAINED SOILS)**

PROPOSED RESIDENTIAL APARTMENT TOWER
LOS ANGELES, CA

**FIGURE A-6**



| KEY | EXPLORATION NUMBER | SAMPLE DEPTH (FEET) | USCS | MOISTURE CONTENT (PERCENT) | DRY DENSITY (PCF) | SOAKED |
|---|---|---|---|---|---|---|
| ● | B-1 | 29.0 | CL | 17.7 | 104 | YES |
| ☒ | B-1 | 49.0 | CL | 17.3 | 104 | YES |
| ▲ | B-1 | 64.0 | CL | 18.2 | 106 | YES |
| ★ | B-2 | 5.0 | CL | 13.1 | 111 | YES |
| ⊙ | B-2 | 30.0 | ML | 15.2 | 111 | YES |
| ✿ | B-2 | 60.0 | CL | 18.2 | 104 | YES |
| ○ | B-3 | 4.0 | ML | 11.5 | 107 | YES |
| △ | B-3 | 30.0 | CL | 20.9 | 104 | YES |
| ⊗ | B-3 | 50.0 | ML | 15.6 | 111 | YES |

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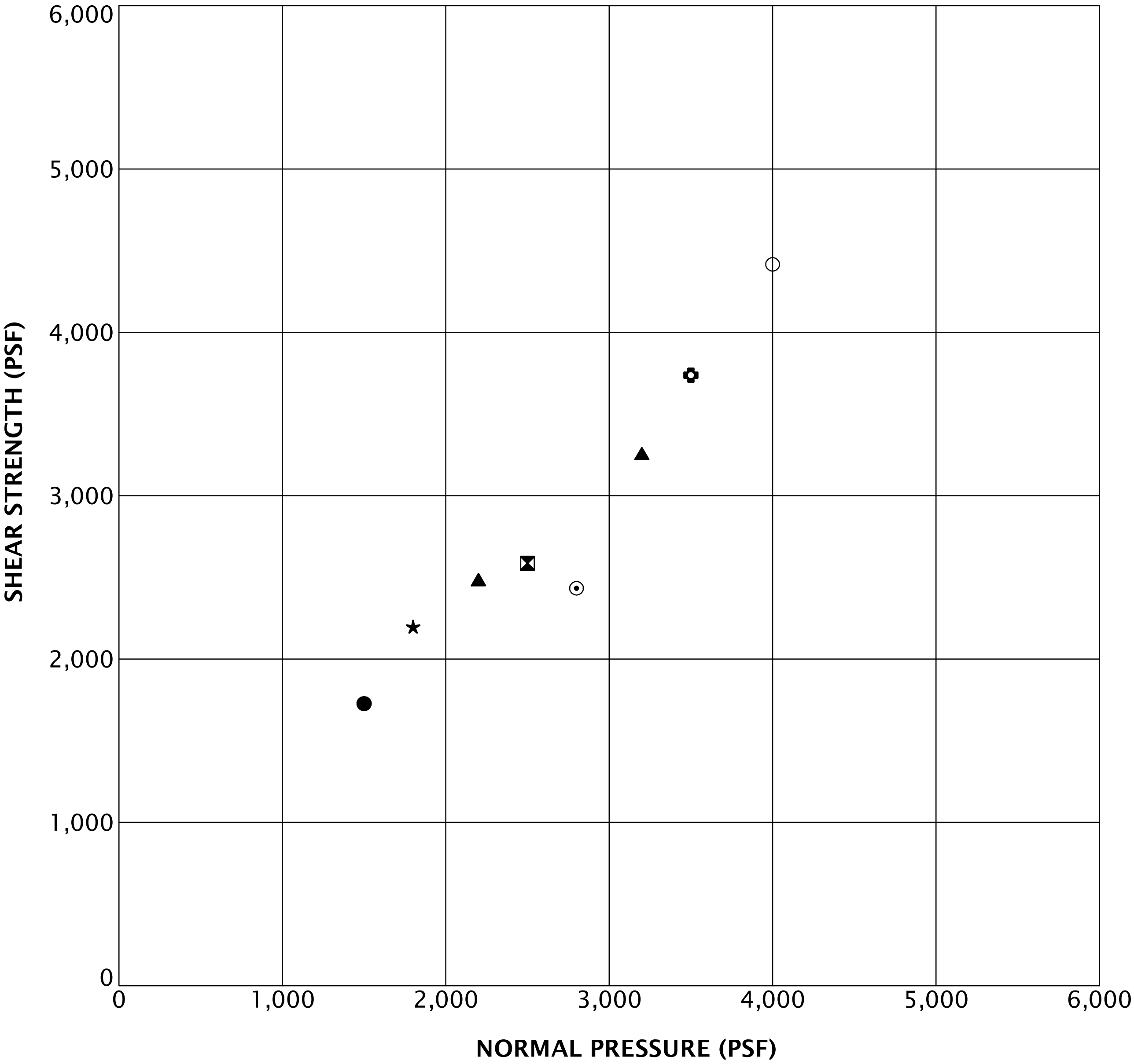
MAY 2007

**DIRECT SHEAR TEST RESULTS (FINE-GRAINED SOILS)**

PROPOSED RESIDENTIAL APARTMENT TOWER
LOS ANGELES, CA

**FIGURE A-7**

DIRECT_SHEAR_FAIL_ENV_USCS_TYPE_TITLE_9 MERUELO-11-01-BORING_LOGS.GPJ GEODESIGN.GDT PRINT DATE: 5/16/07:KYK



| KEY | EXPLORATION NUMBER | SAMPLE DEPTH (FEET) | MOISTURE CONTENT (PERCENT) | DRY DENSITY (PCF) | SOAKED |
|---|---|---|---|---|---|
| ● | B-5 | 10.0 | 3 | 101 | YES |
| ☒ | B-5 | 25.0 | 5 | 117 | YES |
| ▲ | B-5 | 45.0 | 24 | 100 | YES |
| ★ | B-6 | 15.0 | 7 | 120 | YES |
| ⊙ | B-6 | 30.0 | 19 | 111 | YES |
| ✿ | B-6 | 40.0 | 7 | 111 | YES |
| ○ | B-6 | 50.0 | 14 | 117 | YES |
| | | | | | |
| | | | | | |
| | | | | | |

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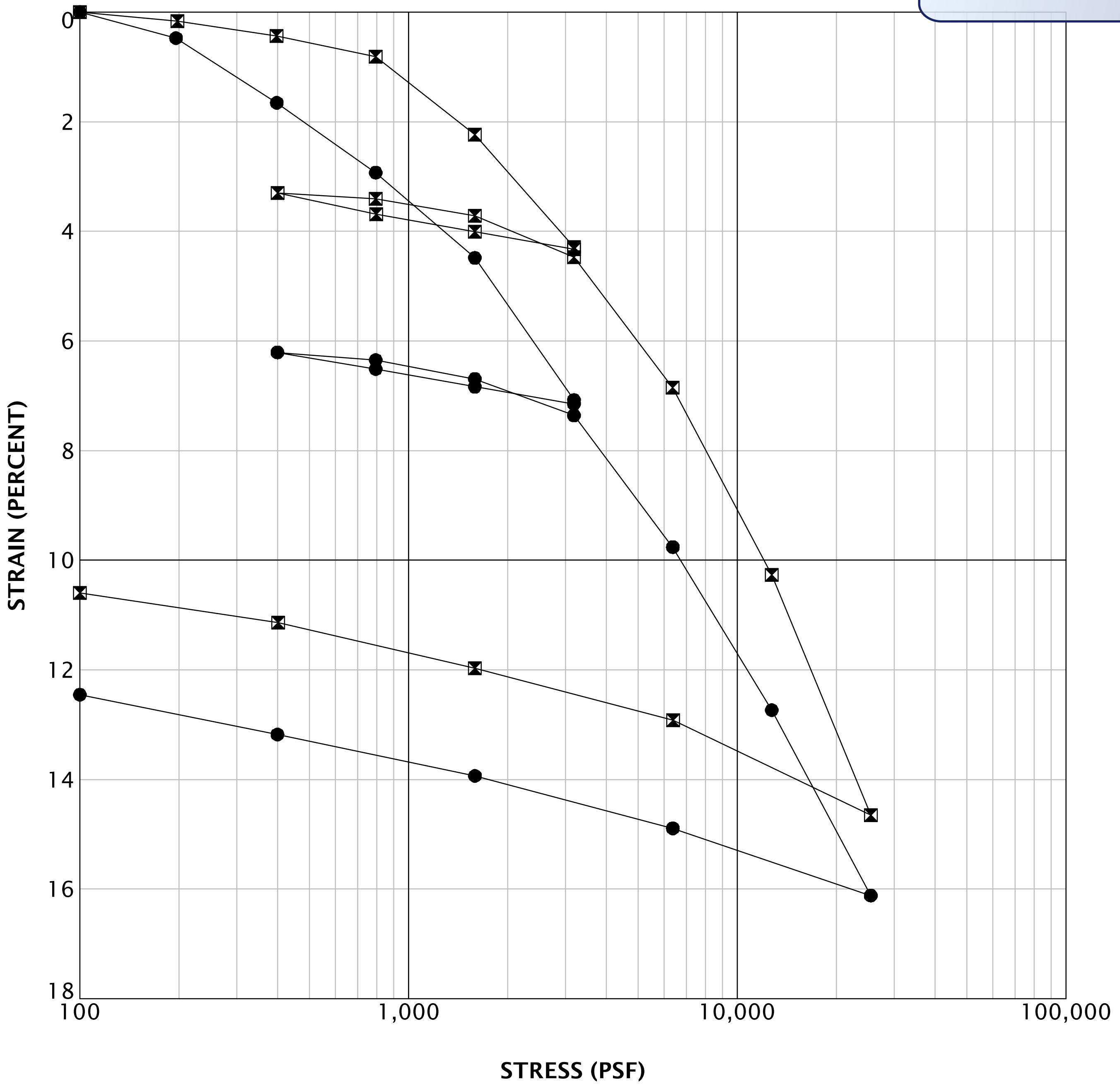
## DIRECT SHEAR TEST RESULTS

PROPOSED HIGH-RISE TOWER - SITE 2
LOS ANGELES, CA

**FIGURE A-4**



| KEY | EXPLORATION NUMBER | SAMPLE DEPTH (FEET) | USCS | MOISTURE CONTENT (PERCENT) | DRY DENSITY (PCF) | WATER ADDED AT (PSF) |
|---|---|---|---|---|---|---|
| ● | B-1 | 29.0 | CL | 20.6 | 101 | 3200 |
| ☒ | B-2 | 45.0 | CL | 24.9 | 95 | 3200 |

CONSOL_STRAIN_100K_H20_ADDED_USCS - ANAH MERUELO-11-01-BORING_LOGS.GPJ GEODESIGN.GDT PRINT DATE: 5/16/07:KYK

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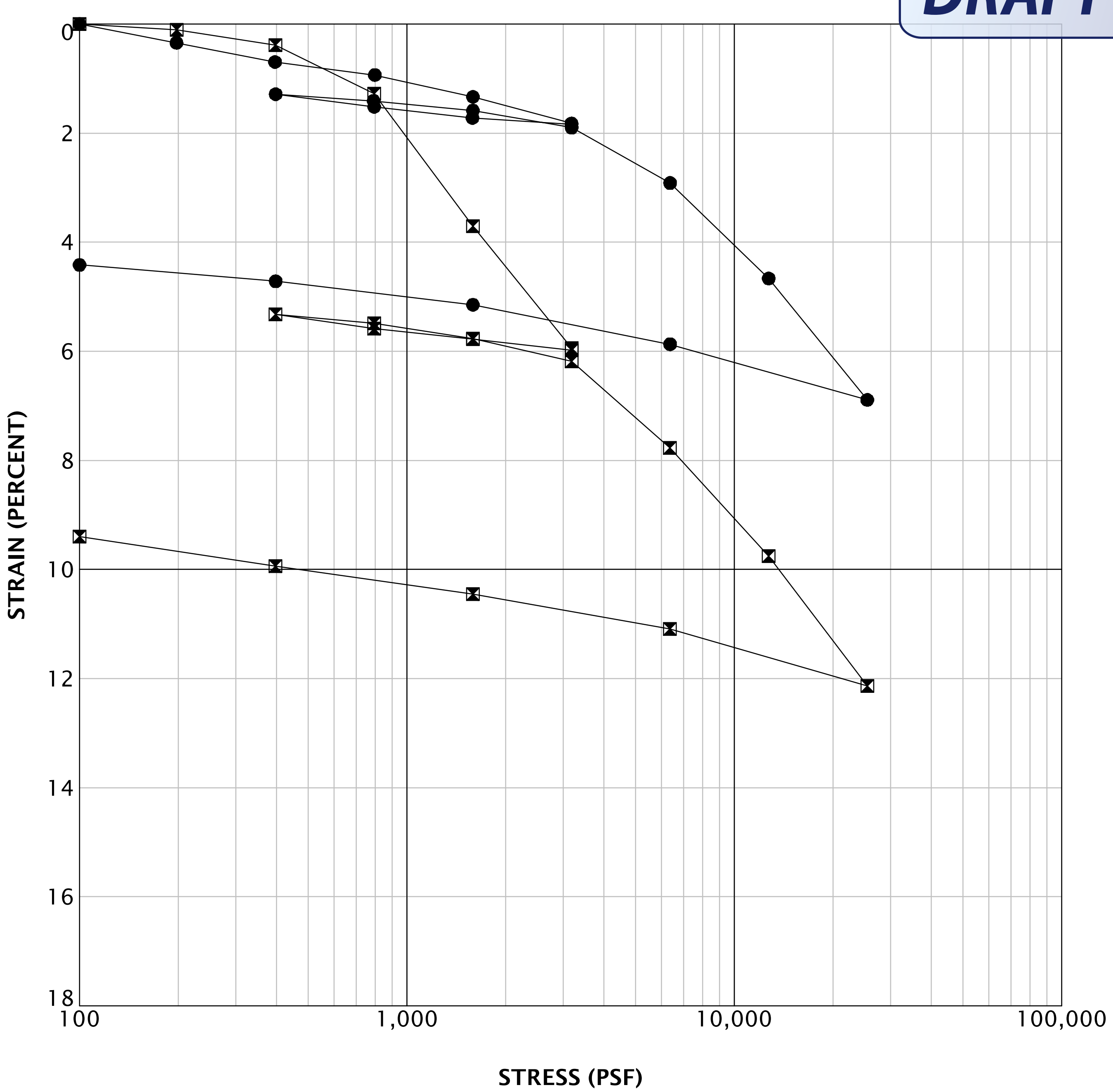
**CONSOLIDATION TEST RESULTS**

PROPOSED RESIDENTIAL APARTMENT TOWER
LOS ANGELES, CA

**FIGURE A-5**



| KEY | EXPLORATION NUMBER | SAMPLE DEPTH (FEET) | USCS | MOISTURE CONTENT (PERCENT) | DRY DENSITY (PCF) | WATER ADDED AT (PSF) |
|---|---|---|---|---|---|---|
| ● | B-3 | 25.0 | SW | 10.3 | 109 | 3200 |
| ☒ | B-4 | 40.0 | SW | 8.6 | 109 | 3200 |

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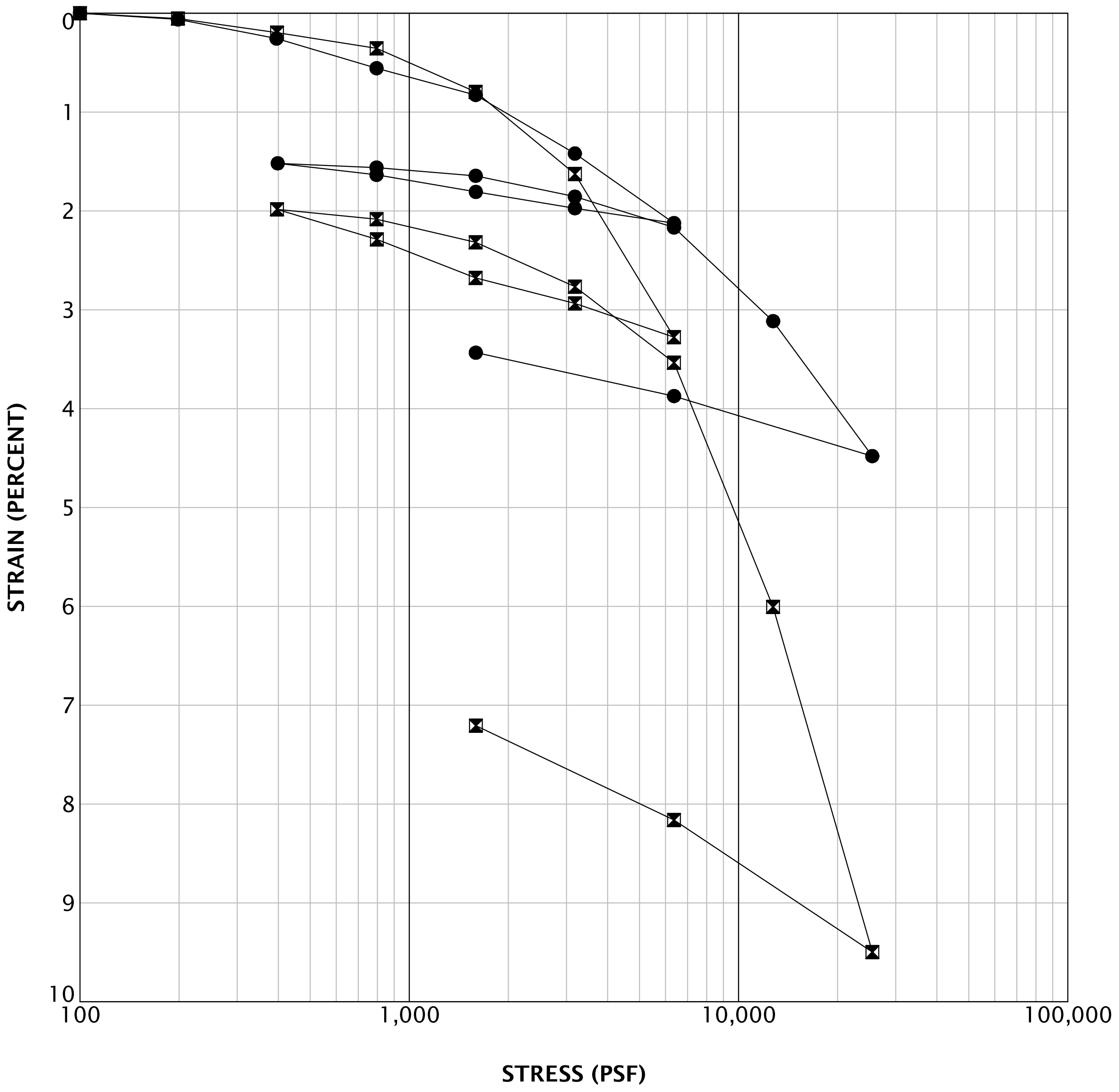
## CONSOLIDATION TEST RESULTS

(continued)

PROPOSED RESIDENTIAL APARTMENT TOWER
LOS ANGELES, CA

**FIGURE A-5**

CONSOL_STRAIN_100K_H20_ADDED_USCS - ANAH MERUELO-11-01-BORING_LOGS.GPJ GEODESIGN.GDT PRINT DATE: 5/16/07:KYK



| KEY | EXPLORATION NUMBER | SAMPLE DEPTH (FEET) | MOISTURE CONTENT (PERCENT) | DRY DENSITY (PCF) |
|---|---|---|---|---|
| ● | B-5 | 55.0 | 18 | 108 |
| ☒ | B-6 | 60.0 | 12 | 120 |
| | | | | |

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DTLASITE2-1-01

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## CONSOLIDATION TEST RESULTS

PROPOSED HIGH-RISE TOWER - SITE 2
LOS ANGELES, CA

**FIGURE A-3**

| 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 (PERCENT) | PLASTIC LIMIT (PERCENT) | PLASTICITY INDEX (PERCENT) |
| B-1 | 1.0 | 244.0 | 20.7 | 104 | | | | | | |
| B-1 | 4.0 | 241.0 | 14.1 | 117 | | | | | | |
| B-1 | 9.0 | 236.0 | 6.6 | | | | | | | |
| B-1 | 29.0 | 216.0 | 20.6 | 101 | | | | | | |
| B-1 | 34.0 | 211.0 | 5.8 | 95 | | | | | | |
| B-1 | 39.0 | 206.0 | 6.1 | | | | | | | |
| B-1 | 44.0 | 201.0 | 19.2 | 104 | | | | | | |
| B-1 | 49.0 | 196.0 | 17.3 | 104 | | | | | | |
| B-1 | 54.0 | 191.0 | 14.9 | 112 | | | | | | |
| B-1 | 59.0 | 186.0 | 23.7 | 102 | | | | | | |
| B-1 | 64.0 | 181.0 | 18.2 | 106 | | | | | | |
| B-1 | 69.0 | 176.0 | 7.9 | 108 | | | | | | |
| B-1 | 69.5 | 175.5 | | | | | 26 | | | |
| B-1 | 74.0 | 171.0 | 3.7 | 107 | | | | | | |
| B-2 | 2.0 | 243.0 | 13.4 | | | | | | | |
| B-2 | 5.0 | 240.0 | 13.1 | 110 | | | | | | |
| B-2 | 10.0 | 235.0 | 2.3 | | | | | | | |
| B-2 | 15.0 | 230.0 | 5.0 | | | | | | | |
| B-2 | 20.0 | 225.0 | 6.5 | 108 | | | | | | |
| B-2 | 25.0 | 220.0 | 7.2 | 106 | | | | | | |
| B-2 | 30.0 | 215.0 | 14.6 | 109 | | | | | | |
| B-2 | 35.0 | 210.0 | 2.9 | | | | | | | |
| B-2 | 40.0 | 205.0 | 6.3 | 114 | | | | | | |
| B-2 | 45.0 | 200.0 | 24.9 | 95 | | | | | | |
| B-2 | 50.0 | 195.0 | 16.8 | 110 | | | | | | |
| B-2 | 55.0 | 190.0 | 16.4 | 111 | | | | | | |
| B-2 | 55.5 | 189.5 | | | | | 51 | | | |

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## SUMMARY OF LABORATORY DATA

PROPOSED RESIDENTIAL APARTMENT TOWER
LOS ANGELES, CA

**FIGURE A-8**

| 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 (PERCENT) | PLASTIC LIMIT (PERCENT) | PLASTICITY INDEX (PERCENT) |
| B-2 | 60.0 | 185.0 | 18.2 | 104 | | | | | | |
| B-2 | 65.0 | 180.0 | 4.6 | 103 | | | | | | |
| B-2 | 70.0 | 175.0 | 4.8 | 106 | | | | | | |
| B-2 | 75.0 | 170.0 | 3.4 | | | | | | | |
| B-2 | 80.0 | 165.0 | 16.7 | 141 | | | | | | |
| B-2 | 85.0 | 160.0 | 2.5 | | | | | | | |
| B-2 | 90.0 | 155.0 | 5.4 | | | | | | | |
| B-2 | 95.0 | 150.0 | 17.5 | 125 | | | | | | |
| B-2 | 100.0 | 145.0 | 9.8 | 127 | | | | | | |
| B-2 | 105.0 | 140.0 | 3.1 | | | | | | | |
| B-2 | 110.0 | 135.0 | 3.5 | | | | | | | |
| B-2 | 115.0 | 130.0 | 17.4 | 131 | | | | | | |
| B-2 | 120.0 | 125.0 | 4.0 | 131 | | | | | | |
| B-2 | 125.0 | 120.0 | 3.1 | 134 | | | | | | |
| B-3 | 1.0 | 243.0 | 11.4 | 104 | | | | | | |
| B-3 | 4.0 | 240.0 | 13.0 | 103 | | | | | | |
| B-3 | 5.0 | 239.0 | 13.0 | 116 | | | | | | |
| B-3 | 10.0 | 234.0 | 4.3 | | | | | | | |
| B-3 | 25.0 | 219.0 | 10.3 | 109 | | | | | | |
| B-3 | 30.0 | 214.0 | 17.8 | 106 | | | | | | |
| B-3 | 35.0 | 209.0 | 4.6 | | | | | | | |
| B-3 | 40.0 | 204.0 | 7.6 | 110 | | | | | | |
| B-3 | 40.5 | 203.5 | | | | | 12 | | | |
| B-3 | 50.0 | 194.0 | 15.5 | 113 | | | | | | |
| B-3 | 55.0 | 189.0 | 17.2 | 109 | | | | | | |
| B-3 | 60.0 | 184.0 | 22.9 | 100 | | | | | | |
| B-3 | 65.0 | 179.0 | 22.8 | 95 | | | | | | |

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## SUMMARY OF LABORATORY DATA
(continued)

PROPOSED RESIDENTIAL APARTMENT TOWER
LOS ANGELES, CA

**FIGURE A-8**

| 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 (PERCENT) | PLASTIC LIMIT (PERCENT) | PLASTICITY INDEX (PERCENT) |
| B-3 | 70.0 | 174.0 | 3.4 | | | | | | | |
| B-3 | 75.0 | 169.0 | 3.2 | | | | | | | |
| B-4 | 2.0 | 242.0 | 13.3 | 104 | | | | | | |
| B-4 | 5.0 | 239.0 | 11.1 | 99 | | | | | | |
| B-4 | 10.0 | 234.0 | 7.0 | | | | | | | |
| B-4 | 15.0 | 229.0 | 5.8 | | | | | | | |
| B-4 | 20.0 | 224.0 | 3.0 | | | | | | | |
| B-4 | 25.0 | 219.0 | 8.7 | | | | | | | |
| B-4 | 30.0 | 214.0 | 17.5 | 107 | | | | | | |
| B-4 | 40.0 | 204.0 | 8.6 | 109 | | | | | | |
| B-4 | 50.0 | 194.0 | 14.1 | 116 | | | | | | |
| B-4 | 55.0 | 189.0 | 15.6 | 111 | | | | | | |
| B-4 | 60.0 | 184.0 | 18.8 | 134 | | | | | | |
| B-4 | 65.0 | 179.0 | 14.0 | 136 | | | | | | |
| B-4 | 70.0 | 174.0 | 13.3 | 143 | | | | | | |
| B-4 | 76.0 | 168.0 | 3.1 | | | | | | | |

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**SUMMARY OF LABORATORY DATA**
(continued)

PROPOSED RESIDENTIAL APARTMENT TOWER
LOS ANGELES, CA

**FIGURE A-8**

| 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-5 | 5.0 | 241.7 | 12 | 89 | | | | | | |
| B-5 | 10.0 | 236.7 | 3 | 101 | | | | | | |
| B-5 | 25.0 | 221.7 | 5 | 117 | | | | | | |
| B-5 | 35.0 | 211.7 | 12 | 96 | | | | | | |
| B-5 | 45.0 | 201.7 | 24 | 100 | | | | | | |
| B-5 | 55.0 | 191.7 | 18 | 108 | | | | | | |
| B-5 | 65.0 | 181.7 | 32 | 86 | | | | | | |
| B-6 | 5.0 | 240.6 | 14 | 113 | | | | | | |
| B-6 | 15.0 | 230.6 | 7 | 120 | | | | | | |
| B-6 | 20.0 | 225.6 | 10 | 100 | | | | | | |
| B-6 | 30.0 | 215.6 | 19 | 111 | | | | | | |
| B-6 | 40.0 | 205.6 | 7 | 111 | | | | | | |
| B-6 | 50.0 | 195.6 | 14 | 117 | | | | | | |
| B-6 | 55.0 | 190.6 | 4 | 110 | | | | | | |
| B-6 | 60.0 | 185.6 | 12 | 120 | | | | | | |
| B-6 | 70.0 | 175.6 | 4 | 115 | | | | | | |
| B-6 | 75.0 | 170.6 | 19 | 68 | | | | | | |

LAB SUMMARY DTLASITE2-1-01-B5_6.GPJ GEODESIGN.GDT PRINT DATE: 6/1/18:KT

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JUNE 2018

**SUMMARY OF LABORATORY DATA**

PROPOSED HIGH-RISE TOWER - SITE 2
LOS ANGELES, CA

**FIGURE A-5**

# APPENDIX B

## Prior Field Percolation Test Results

**Project Number:** DTLASite2-1-01
**Boring Number:** B-5
**Boring Diameter** 8 in = 0.66667 ft
**Hours Pre-Soak:** <1
**Date Pre-Soak Initiated:** 12/4/2017
**Depth of Bottom (Below Grade):** 75.50 ft
**Casing Stick-up:** 0 ft
**Total Length of Well Casing** 75.50 ft
**Name of Tester:** AJA
**Date Tested:** 12/4/2017
**Method to Prevent Caving:** Well Casing
**Checked by:** **Date:**

| Status | t-intial | t-final | Δt (min) | Δt (hours) | Initial Depth to Water (ft) | Final Depth to Water (ft) | Initial Head (ft) | Final Head (ft) | Water Level Drop (ft) | Water Level Drop (in) | Measured Infiltration Rate (in/hr) | $CF_t^2$ | $CF_V^3$ | $CF_S^4$ | $CF^5$ | Design Infiltration Rate (in/hr) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Presoak | 9:40 | 9:46 | 5.50 | 0.09 | 74.50 | 75.50 | 1.00 | 0.00 | 1.00 | 12.0 | 130.9 | - | - | - | - | |
| | 9:48 | 10:03 | 14.75 | 0.25 | 74.50 | 75.50 | 1.00 | 0.00 | 1.00 | 12.0 | 48.8 | - | - | - | - | |
| Time Trial Tests | 10:05 | 10:13 | 7.68 | 0.13 | 74.50 | 75.50 | 1.00 | 0.00 | 1.00 | 12.0 | 93.8 | - | - | - | - | |
| | 10:15 | 10:21 | 6.50 | 0.11 | 74.50 | 75.50 | 1.00 | 0.00 | 1.00 | 12.0 | 110.8 | - | - | - | - | |
| | 10:23 | 10:33 | 10.00 | 0.17 | 74.50 | 75.50 | 1.00 | 0.00 | 1.00 | 12.0 | 72.0 | - | - | - | - | |
| | 10:34 | 10:44 | 9.50 | 0.16 | 74.50 | 75.50 | 1.00 | 0.00 | 1.00 | 12.0 | 75.8 | - | - | - | - | |
| | 10:45 | 10:55 | 10.00 | 0.17 | 74.50 | 75.40 | 1.00 | 0.10 | 0.90 | 10.8 | 64.8 | - | - | - | - | |
| | 10:57 | 11:07 | 10.00 | 0.17 | 74.50 | 75.35 | 1.00 | 0.15 | 0.85 | 10.2 | 61.2 | - | - | - | - | |
| | 11:13 | 11:23 | 10.00 | 0.17 | 74.50 | 75.30 | 1.00 | 0.20 | 0.80 | 9.6 | 57.6 | - | - | - | - | |
| | 11:27 | 11:37 | 10.00 | 0.17 | 74.50 | 75.30 | 1.00 | 0.20 | 0.80 | 9.6 | 57.6 | - | - | - | - | |
| | 11:39 | 11:49 | 10.00 | 0.17 | 74.50 | 75.32 | 1.00 | 0.18 | 0.82 | 9.8 | 59.0 | 1.0 | 1.0 | 3.0 | 3.0 | 19.4 |

notes:

1. Average of the stabilized rate over the last three consecutive readings
2. Correction factor to account for non-vertical flow
3. Correction factor for site variability, number of tests, and thoroughness of subsurface investigation
4. Correction factor for long-term siltation, plugging and maintenance
5. Total correction factor = $CF_t$ x $CF_v$ x $CF_s$

**Project Number:** DTLASite2-1-01
**Boring Number:** B-6
**Boring Diameter** 8 in = 0.66667 ft
**Hours Pre-Soak:** <1
**Date Pre-Soak Initiated:** 12/1/2017
**Depth of Bottom (Below Grade):** 76.00 ft
**Casing Stick-up:** 0 ft
**Total Length of Well Casing** 76.00 ft
**Name of Tester:** AJA
**Date Tested:** 12/1/2017
**Method to Prevent Caving:** Well Casing
**Checked by:** **Date:**

| Status | t-intial | t-final | Δt (min) | Δt (hours) | Initial Depth to Water (ft) | Final Depth to Water (ft) | Initial Head (ft) | Final Head (ft) | Water Level Drop (ft) | Water Level Drop (in) | Measured Infiltration Rate (in/hr) | $CF_t$[2] | $CF_V$[3] | $CF_S$[4] | CF[5] | Design Infiltration Rate (in/hr) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Presoak | 10:00 | 10:05 | 5.00 | 0.08 | 75.00 | 76.00 | 1.00 | 0.00 | 1.00 | 12.0 | 144.0 | - | - | - | - | |
| | 10:06 | 10:18 | 12.12 | 0.20 | 75.00 | 76.00 | 1.00 | 0.00 | 1.00 | 12.0 | 59.4 | - | - | - | - | |
| Time Trial Tests | 10:19 | 10:26 | 7.50 | 0.13 | 75.00 | 76.00 | 1.00 | 0.00 | 1.00 | 12.0 | 96.0 | - | - | - | - | |
| | 10:28 | 10:37 | 9.68 | 0.16 | 75.00 | 76.00 | 1.00 | 0.00 | 1.00 | 12.0 | 74.4 | - | - | - | - | |
| | 10:38 | 10:48 | 10.00 | 0.17 | 75.00 | 76.00 | 1.00 | 0.00 | 1.00 | 12.0 | 72.0 | - | - | - | - | |
| | 10:49 | 10:59 | 10.00 | 0.17 | 75.00 | 75.91 | 1.00 | 0.09 | 0.91 | 10.9 | 65.5 | - | - | - | - | |
| | 11:01 | 11:11 | 10.00 | 0.17 | 75.00 | 75.84 | 1.00 | 0.16 | 0.84 | 10.1 | 60.5 | - | - | - | - | |
| | 11:14 | 11:24 | 10.00 | 0.17 | 75.00 | 75.85 | 1.00 | 0.15 | 0.85 | 10.2 | 61.2 | - | - | - | - | |
| | 11:25 | 11:35 | 10.00 | 0.17 | 75.00 | 75.81 | 1.00 | 0.19 | 0.81 | 9.7 | 58.3 | - | - | - | - | |
| | 11:37 | 11:47 | 10.00 | 0.17 | 75.00 | 75.83 | 1.00 | 0.17 | 0.83 | 10.0 | 59.8 | - | - | - | - | |
| | 11:48 | 11:58 | 10.00 | 0.17 | 75.00 | 75.81 | 1.00 | 0.19 | 0.81 | 9.7 | 58.3 | 1.0 | 1.0 | 3.0 | 3.0 | 19.6 |

notes:

1. Average of the stabilized rate over the last three consecutive readings
2. Correction factor to account for non-vertical flow
3. Correction factor for site variability, number of tests, and thoroughness of subsurface investigation
4. Correction factor for long-term siltation, plugging and maintenance
5. Total correction factor = $CF_t$ x $CF_v$ x $CF_s$

# APPENDIX C

## Prior Nearby PS Suspension Seismic Velocity Logging

GEOVision
geophysical services

# SUSPENSION P & S VELOCITIES
# AND Vs30 AT
# 1150 SOUTH GRAND AVE,
# BORING B-9

**August 31, 2007**
**Report 7364-01**

# SUSPENSION P & S VELOCITIES
# AND Vs30 AT
# 1150 SOUTH GRAND AVE,
# BORING B-9

**Prepared for**

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**Prepared by**

**GEOVision Geophysical Services**
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(951) 549-1234
Project 7364

**August 31, 2007**
**Report 7364-01**

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# FIGURES

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# APPENDICES

## APPENDIX A FIGURES

## APPENDIX A TABLES

# INTRODUCTION

OYO suspension PS velocity measurements were performed in one uncased boring at 1150 South Grand Avenue, Los Angeles, California. Data acquisition was performed on August 10, 2007 by Robert Steller of GEOVision. Data analysis and report preparation were performed by Robert Steller and reviewed by John Diehl. The work was performed under subcontract with GeoDesign, Inc, with Chris Zadoorian as the point of contact for GeoDesign.

This report describes the field measurements, data analysis, and results of this work.

## SCOPE OF WORK

This report presents the results of suspension velocity measurements in one uncased boring, as detailed below. The purpose of these studies was to supplement stratigraphic information obtained from GeoDesign's soil sampling program and to acquire shear wave velocities and compressional wave velocities as a function of depth, as well as to determine Vs30 for the site.

| BORING DESIGNATION | DATE LOGGED | BORING DEPTH (FEET) | LOCATION |
|---|---|---|---|
| B-9 | 8/10/2007 | 160 | CENTER OF PARKING LOT AT<br>1150 SOUTH GRAND AVE |

Table 1. Boring location and logging date

The OYO Model 170 Suspension Logging Recorder and Suspension Logging Probe were used to obtain in-situ horizontal shear and compressional wave velocity measurements at 1.64 ft intervals. The acquired data was analyzed and a profile of velocity versus depth was produced for both compressional and horizontally polarized shear waves.

A detailed reference for the 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 soil velocity measurements were performed using the Model 170 Suspension Logging system, manufactured by OYO Corporation. This system directly determines the average velocity of a 3.28 ft high segment of the soil column surrounding the boring 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 boring 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 3.28 ft, 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 this survey is 19 ft, with the center point of the receiver pair 12.1 ft above the bottom end of the probe. The probe receives control signals from, and sends the amplified receiver signals to, instrumentation on the surface via an armored 7 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.

The entire probe is suspended in the boring by the cable, therefore, source motion is not coupled directly to the boring walls; rather, the source motion creates a horizontally propagating impulsive pressure wave in the fluid filling the boring 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 boring. These waves propagate through the soil and rock surrounding the boring, 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 7.0 ft 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 (meter versus centimeter 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 Model 170 has six channels (two simultaneous recording channels), each with a 12 bit 1024 sample record. The recorded data is displayed on a CRT display and on paper tape output as six channels with a common time scale. Data is stored on 3.5 inch floppy diskettes for further processing. Up to 8 sampling sequences can be summed to improve the signal to noise ratio of the signals.

Review of the displayed data on the CRT or paper tape allows the operator to set the gains, filters, delay time, pulse length (energy), sample rate, and summing number to optimize the quality of the data before recording. Verification of the calibration of the Model 170 digital recorder is performed every twelve months using a NIST traceable frequency source and counter, as outlined in Appendix B.

## MEASUREMENT PROCEDURES

The boring was logged uncased, filled with bentonite based drilling mud. The suspension probe was positioned with the mid-point of the receiver spacing at grade, and the mechanical and electronic depth counters were set to zero. The probe was lowered to the bottom of the boring, stopping at 1.64 ft intervals to collect data, as summarized below.

At each measurement depth the measurement sequence of two opposite horizontal records and one vertical record was performed, and the gains were adjusted as required. The data from each depth was printed on paper tape, checked, and recorded on diskette before moving to the next depth.

| BORING NUMBER | RUN NUMBER | DEPTH RANGE (FEET) | DEPTH AS DRILLED (FEET) | LOST TO SLOUGH (FEET) | SAMPLE INTERVAL (FEET) | DATE LOGGED |
|---|---|---|---|---|---|---|
| B-9 | 1 | 1.6 – 147.0 | 160.0 | 0.9 | 1.64 | 8/10/2007 |
|  |  |  |  |  |  |  |

Table 2. Logging dates and depth ranges

# DATA ANALYSIS

The recorded digital records were analyzed to locate the first minima 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 are used to calculate the P-wave velocity for that 3.28 ft segment of the soil column. When observable, P-wave arrivals on the horizontal axis records are used to verify the velocities determined from the vertical axis data.

The P-wave velocity calculated from the travel time over the 7.0 ft interval from source to receiver 1 (S-R1) is calculated and plotted for quality assurance of the velocity derived from the travel time between receivers. During analysis, the depth values as recorded are increased by 5.15 ft to correspond to the mid-point of the 7.0 ft S-R1 interval, as illustrated in Figure 1. Travel times are obtained by picking the first break of the P-wave signal at receiver 1 and subtracting 3.9 milliseconds, the calculated and experimentally verified delay from source trigger pulse (beginning of record) to source impact. This delay corresponds to the duration of acceleration of the solenoid before impact.

The recorded digital records are studied to establish the presence of 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 FFT - IFFT lowpass filtering was used to remove the higher frequency P-wave signal from the $S_H$-wave signal. Different filter cutoffs are used to separate P- and $S_H$-waves at different depths, ranging from 500 Hz in the slowest zones to 2000 Hz in the regions of highest velocity. At each depth, the filter frequency is 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 boring 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 7.0 ft interval from source to receiver 1 is calculated and plotted for verification of the velocity derived from the travel time between receivers. During analysis, the depth values are increased by 5.15 ft to correspond to the mid-point of the 7.0 ft S-R1 interval. Travel times are obtained by picking the first break of the $S_H$-wave signal at the near receiver and subtracting 3.9 milliseconds, the calculated and experimentally verified delay from the beginning of the record at the source trigger pulse to source impact.

Figure 2 shows an example of R1 - R2 measurements on a filtered sample suspension. In Figure 2, the time difference over the 3.28 ft interval of 2.46 milliseconds for the horizontal signals is equivalent to an $S_H$-wave velocity of 1334 ft/sec. Final $S_H$-wave velocity is the average of the horizontal normal and horizontal reverse (HR) signals. 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 2000 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.

Vs30 was calculated by summing the calculated travel times over each 1.64 ft interval from 9.8 ft (3.0 m) to a depth of 108.3ft (33.0 m).

# RESULTS

Suspension P- and $S_H$-wave velocities are plotted with the calculated Vs30 of 470 m/sec (1540 ft/sec) in Figure 4. The calculated suspension travel time curves are presented with Vs30 in Figure 5. Tabulated measurement depths, pick times and velocities are presented in Table 3.

Calibration procedures and records for the measurement system are presented in Appendix B.

# SUMMARY

## Discussion of Suspension Results

Suspension PS velocity data are ideally collected in an uncased fluid filled boring, drilled with rotary mud (rotary wash) methods, as this boring was.

Suspension PS velocity data quality is judged based upon 5 criteria:

1. Consistent data between receiver to receiver (R1 – R2) and source to receiver (S – R1) data.
2. Consistent relationship between P-wave and $S_H$ -wave (excluding transition to saturated soils)
3. Consistency between data from adjacent depth intervals.
4. Clarity of P-wave and $S_H$-wave onset, as well as damping of later oscillations.
5. Consistency of profile between adjacent borings, if available.

These data show excellent correlation between R1 – R2 and S – R1 data, as well as excellent correlation between P-wave and $S_H$-wave velocities. No adjacent borings were logged. P-wave and $S_H$-wave onsets are generally clear, and later oscillations are well damped. These are excellent quality velocity data.

Below 8.0 ft, the velocity profile is indicative of firm soil or weathered rock. P-wave velocities rise above 5000 ft/sec (1500 m/sec) between depths of 44 and 58 ft, possibly indicating a perched water table in this depth range.

## Discussion of Vs30

Vs30 for this site from 9.8 to 108.3 ft (3.0 – 33.0 m) was calculated at 1540 ft/sec (470 m/sec), classifying it as a NEHRP site class C.

## Quality Assurance

These velocity measurements were performed using industry-standard or better methods for both measurements and analyses. All work was performed under GEOVision 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 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.

## Data Reliability

P- and $S_H$-wave velocity measurement using the Suspension Method gives average velocities over a 3.28 ft 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%. Standardized field procedures and quality assurance checks contribute to the reliability of these data.

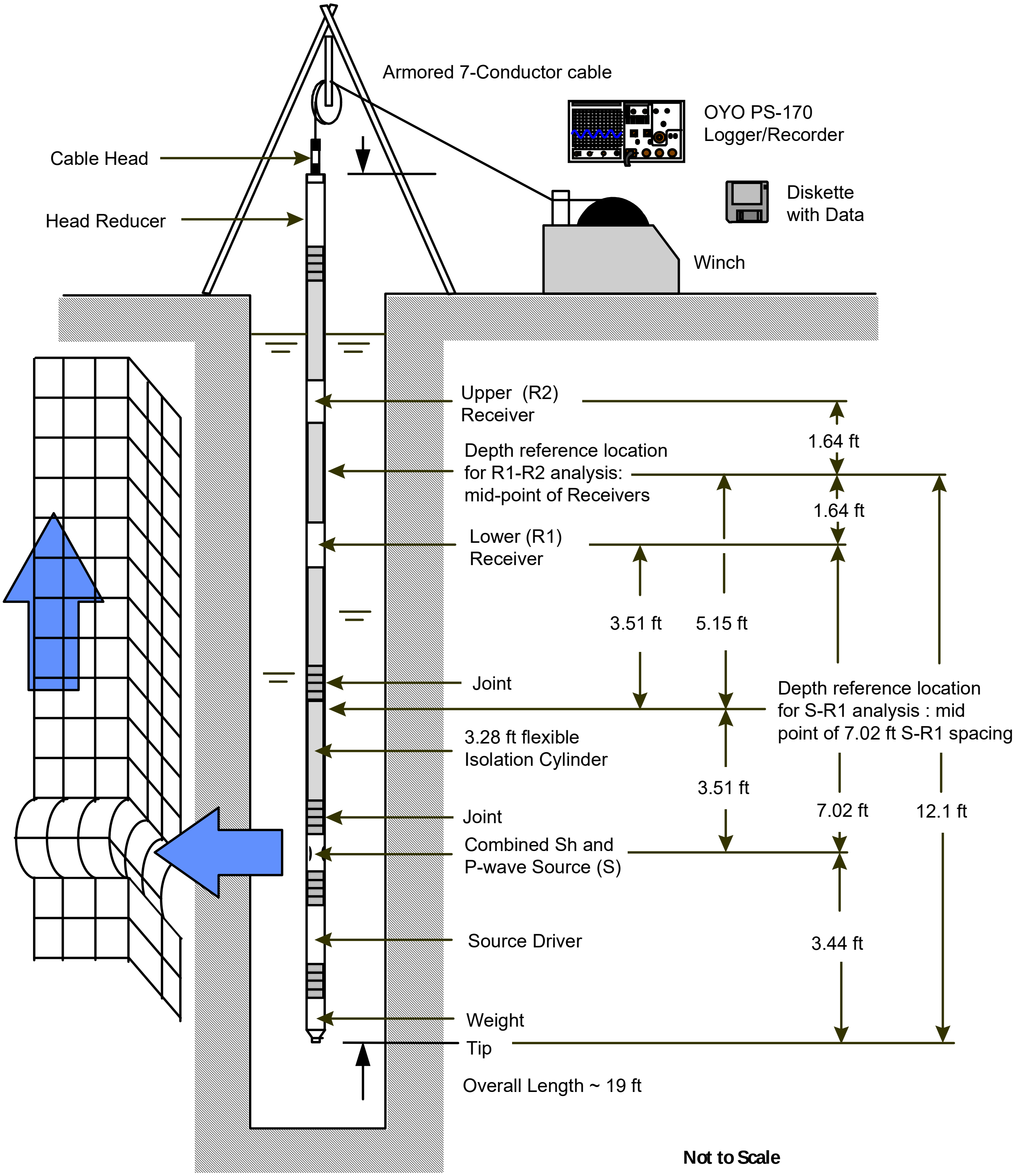

Figure 1. Concept illustration of P-S logging system

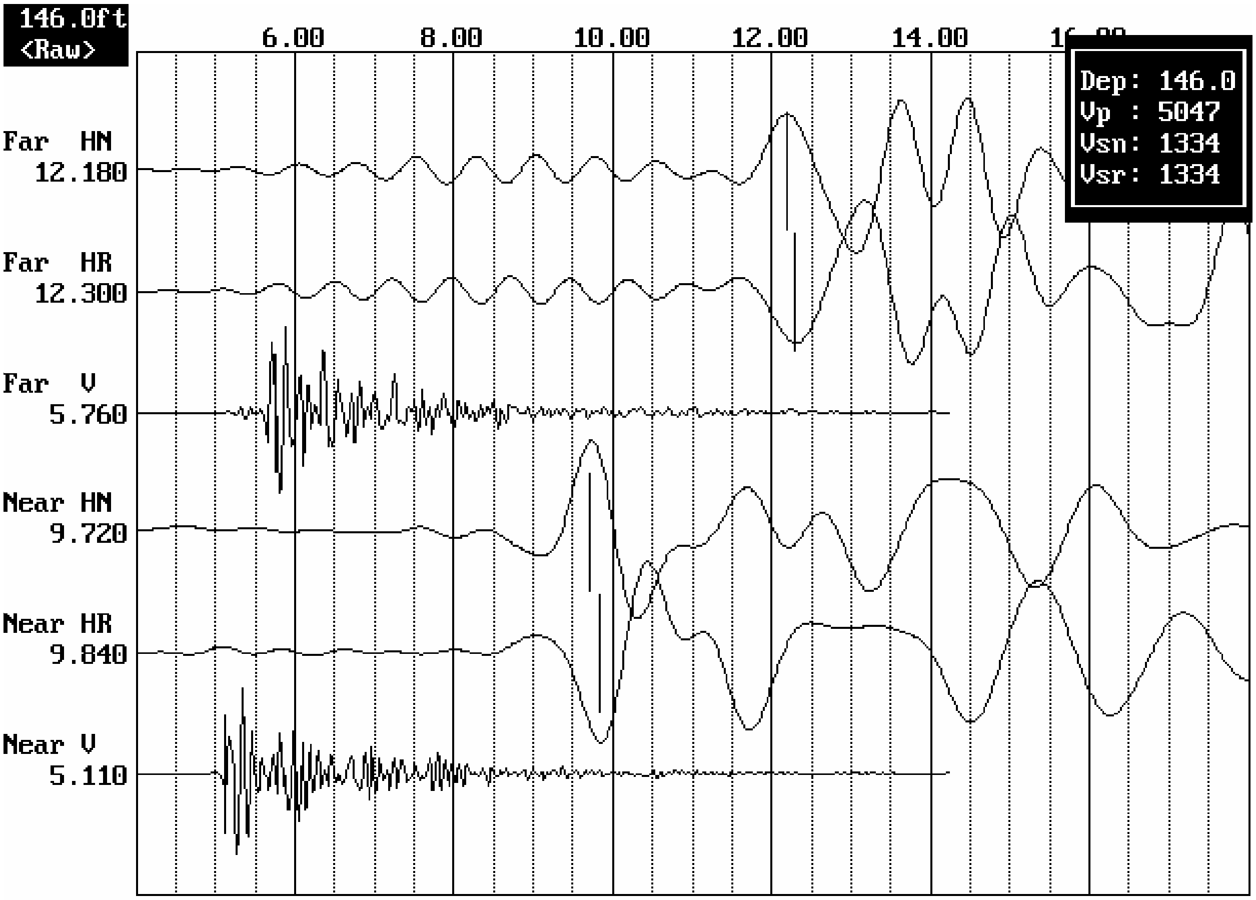

Figure 2. Filtered (2000 Hz lowpass) sample suspension record

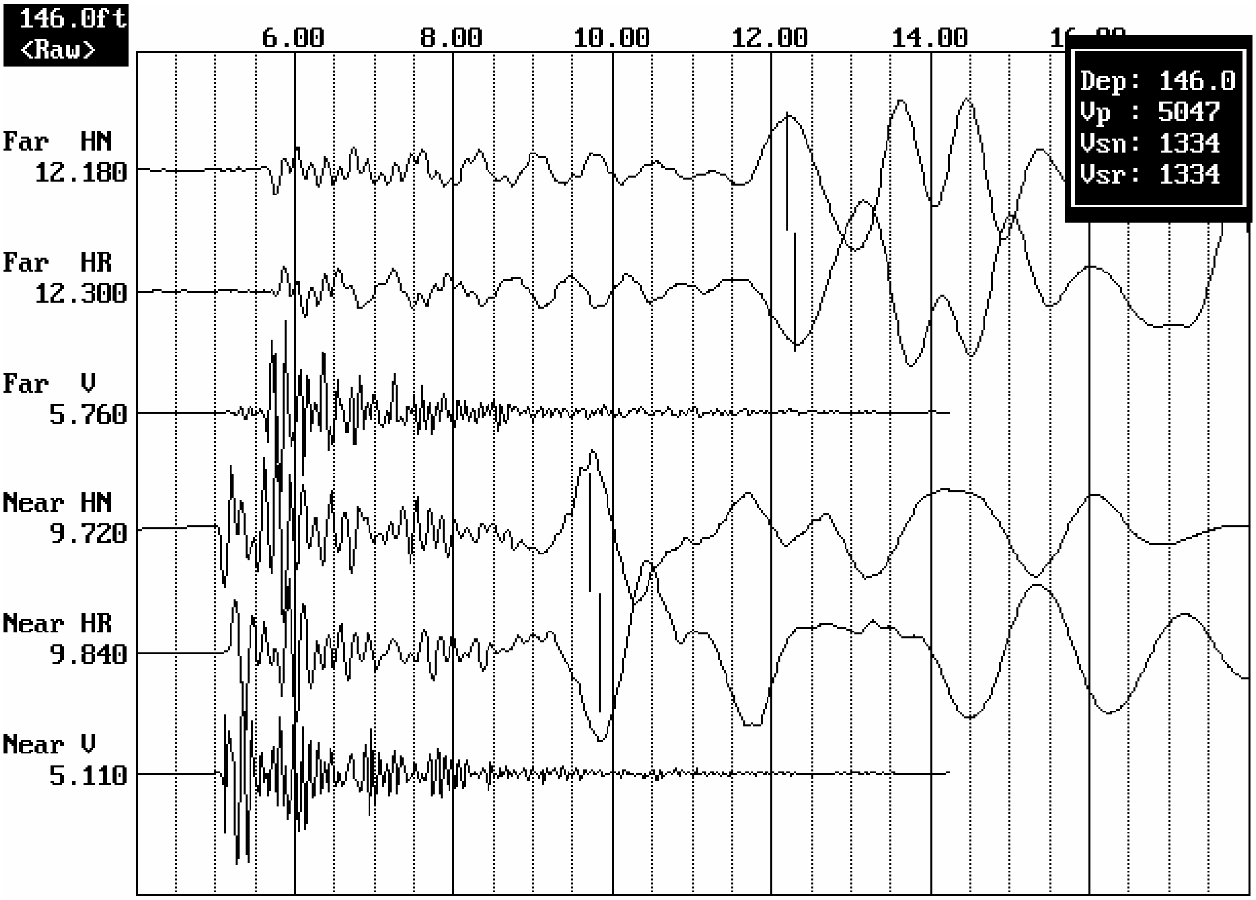

Figure 3. Unfiltered sample suspension record

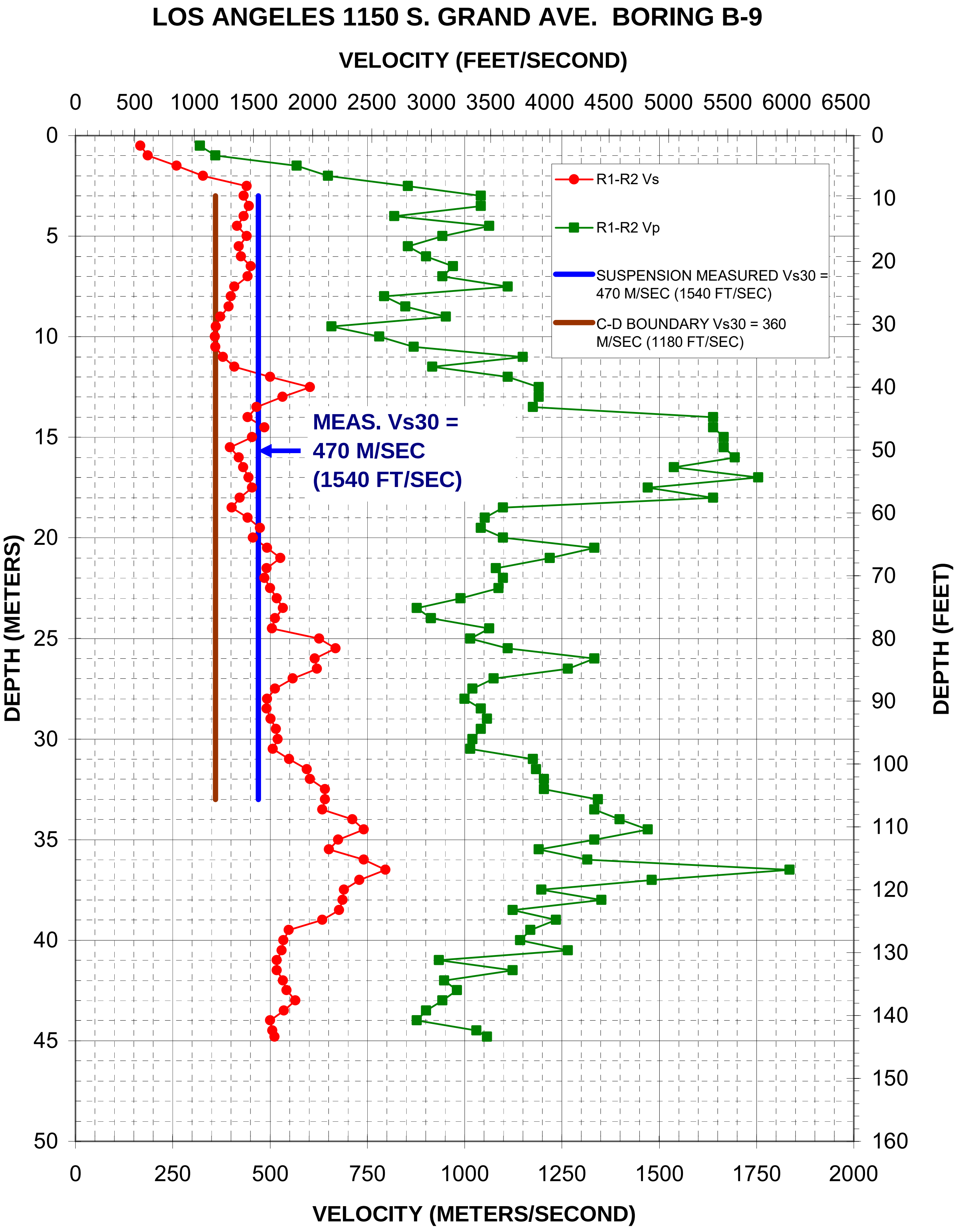

Figure 4. Boring B-9, Suspension P- and $S_H$-wave Velocities with Vs30 values

| Depth | | Pick Times | | | | | | Velocity | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| (m) | (feet) | Far-Hn (millisec) | Far-Hr (millisec) | Far-V (millisec) | Near-Hn (millisec) | Near-Hr (millisec) | Near-V (millisec) | V-$S_H$ (m/sec) | V-P (m/sec) | V-$S_H$ (ft/sec) | V-P (ft/sec) |
| 0.5 | 1.6 | 16.65 | 16.60 | 11.10 | 10.55 | 10.75 | 7.98 | 167 | 321 | 549 | 1052 |
| 1.0 | 3.3 | 15.35 | 14.95 | 9.92 | 9.75 | 9.80 | 7.14 | 186 | 360 | 610 | 1180 |
| 1.5 | 4.9 | 12.70 | 12.90 | 7.42 | 9.00 | 8.90 | 5.66 | 260 | 568 | 852 | 1864 |
| 2.0 | 6.6 | 12.05 | 12.30 | 7.50 | 9.10 | 9.15 | 5.96 | 328 | 649 | 1076 | 2130 |
| 2.5 | 8.2 | 11.50 | 11.34 | 7.28 | 9.12 | 9.18 | 6.11 | 441 | 855 | 1445 | 2804 |
| 3.0 | 9.8 | 11.58 | 11.40 | 6.90 | 9.16 | 9.20 | 5.94 | 433 | 1042 | 1420 | 3418 |
| 3.5 | 11.5 | 11.42 | 11.44 | 6.87 | 9.20 | 9.18 | 5.91 | 446 | 1042 | 1465 | 3418 |
| 4.0 | 13.1 | 11.54 | 11.34 | 7.27 | 9.10 | 9.16 | 6.05 | 433 | 820 | 1420 | 2689 |
| 4.5 | 14.8 | 11.50 | 11.60 | 6.84 | 9.06 | 9.22 | 5.90 | 415 | 1064 | 1361 | 3490 |
| 5.0 | 16.4 | 11.58 | 11.50 | 6.94 | 9.30 | 9.24 | 5.88 | 441 | 943 | 1445 | 3095 |
| 5.5 | 18.0 | 11.50 | 11.62 | 7.28 | 9.14 | 9.22 | 6.11 | 420 | 855 | 1379 | 2804 |
| 6.0 | 19.7 | 11.58 | 11.68 | 7.29 | 9.26 | 9.30 | 6.18 | 426 | 901 | 1396 | 2956 |
| 6.5 | 21.3 | 11.60 | 11.66 | 7.00 | 9.38 | 9.44 | 5.97 | 450 | 971 | 1478 | 3185 |
| 7.0 | 23.0 | 11.76 | 11.76 | 6.85 | 9.46 | 9.54 | 5.79 | 442 | 943 | 1452 | 3095 |
| 7.5 | 24.6 | 12.04 | 12.12 | 6.43 | 9.62 | 9.64 | 5.53 | 408 | 1111 | 1339 | 3645 |
| 8.0 | 26.2 | 12.26 | 12.38 | 7.53 | 9.78 | 9.86 | 6.27 | 400 | 794 | 1312 | 2604 |
| 8.5 | 27.9 | 12.50 | 12.60 | 6.91 | 9.98 | 10.04 | 5.73 | 394 | 847 | 1292 | 2780 |
| 9.0 | 29.5 | 12.48 | 12.52 | 7.09 | 9.78 | 9.84 | 6.04 | 372 | 952 | 1220 | 3125 |
| 9.5 | 31.2 | 12.04 | 12.14 | 7.83 | 9.28 | 9.36 | 6.31 | 361 | 658 | 1184 | 2158 |
| 10.0 | 32.8 | 11.60 | 11.64 | 6.93 | 8.80 | 8.86 | 5.65 | 358 | 781 | 1176 | 2563 |
| 10.5 | 34.4 | 11.06 | 11.14 | 6.94 | 8.28 | 8.36 | 5.79 | 360 | 870 | 1180 | 2853 |
| 11.0 | 36.1 | 10.70 | 10.80 | 6.20 | 8.06 | 8.16 | 5.33 | 379 | 1149 | 1243 | 3771 |
| 11.5 | 37.7 | 10.52 | 10.58 | 6.61 | 8.06 | 8.14 | 5.52 | 408 | 917 | 1339 | 3010 |
| 12.0 | 39.4 | 10.28 | 10.36 | 6.31 | 8.28 | 8.36 | 5.41 | 500 | 1111 | 1640 | 3645 |
| 12.5 | 41.0 | 10.32 | 10.40 | 6.27 | 8.66 | 8.74 | 5.43 | 602 | 1190 | 1976 | 3906 |
| 13.0 | 42.7 | 10.56 | 10.60 | 6.13 | 8.66 | 8.74 | 5.29 | 532 | 1190 | 1745 | 3906 |
| 13.5 | 44.3 | 10.92 | 11.02 | 6.08 | 8.78 | 8.86 | 5.23 | 465 | 1176 | 1526 | 3860 |
| 14.0 | 45.9 | 11.02 | 11.12 | 5.91 | 8.76 | 8.86 | 5.30 | 442 | 1639 | 1452 | 5378 |
| 14.5 | 47.6 | 11.02 | 11.08 | 5.97 | 8.94 | 9.04 | 5.36 | 485 | 1639 | 1593 | 5378 |
| 15.0 | 49.2 | 11.10 | 11.18 | 5.89 | 8.90 | 8.98 | 5.29 | 455 | 1667 | 1491 | 5468 |
| 15.5 | 50.9 | 11.40 | 11.52 | 5.88 | 8.90 | 8.98 | 5.28 | 397 | 1667 | 1302 | 5468 |
| 16.0 | 52.5 | 11.58 | 11.62 | 5.98 | 9.18 | 9.26 | 5.39 | 420 | 1695 | 1379 | 5561 |
| 16.5 | 54.1 | 11.26 | 11.34 | 5.99 | 8.94 | 9.02 | 5.34 | 431 | 1538 | 1414 | 5047 |
| 17.0 | 55.8 | 11.08 | 11.16 | 6.04 | 8.84 | 8.90 | 5.47 | 444 | 1754 | 1458 | 5756 |
| 17.5 | 57.4 | 11.02 | 11.12 | 6.30 | 8.84 | 8.90 | 5.62 | 455 | 1471 | 1491 | 4825 |
| 18.0 | 59.1 | 11.06 | 11.14 | 6.45 | 8.70 | 8.76 | 5.84 | 422 | 1639 | 1384 | 5378 |
| 18.5 | 60.7 | 10.98 | 11.06 | 6.47 | 8.48 | 8.58 | 5.56 | 402 | 1099 | 1318 | 3605 |
| 19.0 | 62.3 | 10.80 | 10.86 | 6.67 | 8.52 | 8.62 | 5.72 | 442 | 1053 | 1452 | 3454 |
| 19.5 | 64.0 | 10.66 | 10.74 | 6.77 | 8.54 | 8.64 | 5.81 | 474 | 1042 | 1555 | 3418 |
| 20.0 | 65.6 | 10.62 | 10.70 | 6.88 | 8.42 | 8.52 | 5.97 | 457 | 1099 | 1498 | 3605 |
| 20.5 | 67.3 | 10.54 | 10.62 | 7.02 | 8.50 | 8.60 | 6.27 | 493 | 1333 | 1616 | 4374 |
| 21.0 | 68.9 | 10.33 | 10.41 | 7.04 | 8.42 | 8.52 | 6.22 | 526 | 1220 | 1727 | 4001 |
| 21.5 | 70.5 | 10.41 | 10.49 | 7.08 | 8.37 | 8.46 | 6.15 | 491 | 1081 | 1612 | 3547 |
| 22.0 | 72.2 | 10.40 | 10.48 | 7.01 | 8.34 | 8.42 | 6.10 | 485 | 1099 | 1593 | 3605 |
| 22.5 | 73.8 | 10.22 | 10.29 | 7.02 | 8.22 | 8.29 | 6.10 | 500 | 1087 | 1640 | 3566 |
| 23.0 | 75.5 | 9.86 | 9.92 | 6.96 | 7.94 | 7.98 | 5.95 | 518 | 990 | 1700 | 3248 |
| 23.5 | 77.1 | 9.72 | 9.82 | 7.03 | 7.85 | 7.94 | 5.89 | 533 | 877 | 1750 | 2878 |
| 24.0 | 78.7 | 9.64 | 9.73 | 7.00 | 7.69 | 7.78 | 5.90 | 513 | 913 | 1682 | 2996 |
| 24.5 | 80.4 | 9.50 | 9.60 | 6.80 | 7.52 | 7.62 | 5.86 | 505 | 1064 | 1657 | 3490 |
| 25.0 | 82.0 | 9.41 | 9.56 | 6.90 | 7.83 | 7.95 | 5.92 | 627 | 1015 | 2057 | 3331 |

Table 3. Boring B-9, Suspension R1-R2 depth,
pick times, and velocities

| Depth | | Pick Times | | | | | | Velocity | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| (m) | (feet) | Far-Hn (millisec) | Far-Hr (millisec) | Far-V (millisec) | Near-Hn (millisec) | Near-Hr (millisec) | Near-V (millisec) | V-$S_H$ (m/sec) | V-P (m/sec) | V-$S_H$ (ft/sec) | V-P (ft/sec) |
| 25.5 | 83.7 | 9.47 | 9.57 | 6.99 | 7.99 | 8.06 | 6.09 | 669 | 1111 | 2195 | 3645 |
| 26.0 | 85.3 | 9.81 | 9.88 | 6.79 | 8.19 | 8.25 | 6.04 | 615 | 1333 | 2019 | 4374 |
| 26.5 | 86.9 | 9.99 | 10.07 | 6.88 | 8.39 | 8.45 | 6.09 | 621 | 1266 | 2038 | 4153 |
| 27.0 | 88.6 | 10.17 | 10.22 | 6.93 | 8.38 | 8.43 | 6.00 | 559 | 1075 | 1833 | 3528 |
| 27.5 | 90.2 | 10.26 | 10.34 | 6.79 | 8.31 | 8.39 | 5.81 | 513 | 1020 | 1682 | 3348 |
| 28.0 | 91.9 | 10.33 | 10.38 | 6.75 | 8.31 | 8.34 | 5.75 | 493 | 1000 | 1616 | 3281 |
| 28.5 | 93.5 | 10.29 | 10.38 | 6.78 | 8.26 | 8.34 | 5.82 | 491 | 1042 | 1612 | 3418 |
| 29.0 | 95.1 | 10.11 | 10.20 | 6.86 | 8.12 | 8.20 | 5.91 | 501 | 1058 | 1645 | 3472 |
| 29.5 | 96.8 | 9.96 | 10.06 | 6.86 | 8.03 | 8.11 | 5.90 | 515 | 1042 | 1691 | 3418 |
| 30.0 | 98.4 | 9.73 | 9.81 | 6.81 | 7.81 | 7.88 | 5.83 | 519 | 1020 | 1704 | 3348 |
| 30.5 | 100.1 | 9.59 | 9.66 | 6.80 | 7.62 | 7.69 | 5.81 | 508 | 1015 | 1665 | 3331 |
| 31.0 | 101.7 | 9.39 | 9.49 | 6.67 | 7.58 | 7.66 | 5.82 | 549 | 1176 | 1803 | 3860 |
| 31.5 | 103.3 | 9.11 | 9.19 | 6.58 | 7.43 | 7.51 | 5.74 | 595 | 1183 | 1953 | 3883 |
| 32.0 | 105.0 | 8.93 | 8.98 | 6.42 | 7.26 | 7.33 | 5.59 | 602 | 1205 | 1976 | 3953 |
| 32.5 | 106.6 | 8.89 | 8.95 | 6.30 | 7.33 | 7.39 | 5.47 | 641 | 1205 | 2103 | 3953 |
| 33.0 | 108.3 | 8.84 | 8.91 | 6.23 | 7.27 | 7.36 | 5.49 | 641 | 1342 | 2103 | 4404 |
| 33.5 | 109.9 | 8.72 | 8.76 | 6.22 | 7.13 | 7.20 | 5.47 | 635 | 1333 | 2083 | 4374 |
| 34.0 | 111.5 | 8.53 | 8.62 | 6.18 | 7.12 | 7.22 | 5.46 | 712 | 1399 | 2335 | 4589 |
| 34.5 | 113.2 | 8.46 | 8.54 | 6.15 | 7.11 | 7.19 | 5.47 | 741 | 1471 | 2430 | 4825 |
| 35.0 | 114.8 | 8.54 | 8.61 | 6.30 | 7.05 | 7.14 | 5.55 | 676 | 1333 | 2217 | 4374 |
| 35.5 | 116.5 | 8.61 | 8.68 | 6.39 | 7.07 | 7.15 | 5.55 | 651 | 1190 | 2137 | 3906 |
| 36.0 | 118.1 | 8.55 | 8.62 | 6.37 | 7.20 | 7.27 | 5.61 | 741 | 1316 | 2430 | 4317 |
| 36.5 | 119.8 | 8.59 | 8.67 | 6.25 | 7.34 | 7.41 | 5.71 | 797 | 1835 | 2614 | 6020 |
| 37.0 | 121.4 | 9.00 | 9.05 | 6.36 | 7.62 | 7.69 | 5.69 | 730 | 1481 | 2395 | 4861 |
| 37.5 | 123.0 | 9.24 | 9.31 | 6.69 | 7.78 | 7.87 | 5.85 | 690 | 1198 | 2263 | 3929 |
| 38.0 | 124.7 | 9.39 | 9.45 | 6.71 | 7.93 | 8.00 | 5.97 | 687 | 1351 | 2255 | 4434 |
| 38.5 | 126.3 | 9.62 | 9.69 | 6.92 | 8.14 | 8.22 | 6.03 | 678 | 1124 | 2224 | 3686 |
| 39.0 | 128.0 | 9.78 | 9.86 | 6.98 | 8.18 | 8.31 | 6.17 | 635 | 1235 | 2083 | 4050 |
| 39.5 | 129.6 | 10.01 | 10.10 | 7.10 | 8.19 | 8.27 | 6.24 | 548 | 1170 | 1798 | 3837 |
| 40.0 | 131.2 | 10.10 | 10.25 | 7.17 | 8.27 | 8.34 | 6.29 | 535 | 1143 | 1754 | 3750 |
| 40.5 | 132.9 | 9.97 | 10.04 | 6.90 | 8.08 | 8.16 | 6.11 | 531 | 1266 | 1740 | 4153 |
| 41.0 | 134.5 | 10.07 | 10.13 | 7.25 | 8.14 | 8.20 | 6.18 | 518 | 935 | 1700 | 3066 |
| 41.5 | 136.2 | 10.11 | 10.20 | 7.17 | 8.19 | 8.26 | 6.28 | 518 | 1124 | 1700 | 3686 |
| 42.0 | 137.8 | 10.13 | 10.21 | 7.22 | 8.25 | 8.34 | 6.17 | 533 | 948 | 1750 | 3110 |
| 42.5 | 139.4 | 10.13 | 10.21 | 7.13 | 8.28 | 8.37 | 6.11 | 542 | 980 | 1778 | 3217 |
| 43.0 | 141.1 | 10.15 | 10.23 | 7.01 | 8.38 | 8.46 | 5.95 | 565 | 943 | 1854 | 3095 |
| 43.5 | 142.7 | 10.17 | 10.22 | 6.86 | 8.30 | 8.36 | 5.75 | 536 | 901 | 1759 | 2956 |
| 44.0 | 144.4 | 10.29 | 10.35 | 6.61 | 8.29 | 8.35 | 5.47 | 500 | 877 | 1640 | 2878 |
| 44.5 | 146.0 | 10.28 | 10.36 | 6.25 | 8.31 | 8.38 | 5.28 | 506 | 1031 | 1661 | 3382 |
| 44.8 | 147.0 | 10.30 | 10.39 | 6.18 | 8.35 | 8.43 | 5.23 | 512 | 1058 | 1678 | 3472 |
| | | | | | | | | | | | |

Table 3, continued. Boring B-9, Suspension R1-R2 depth, pick times, and velocities

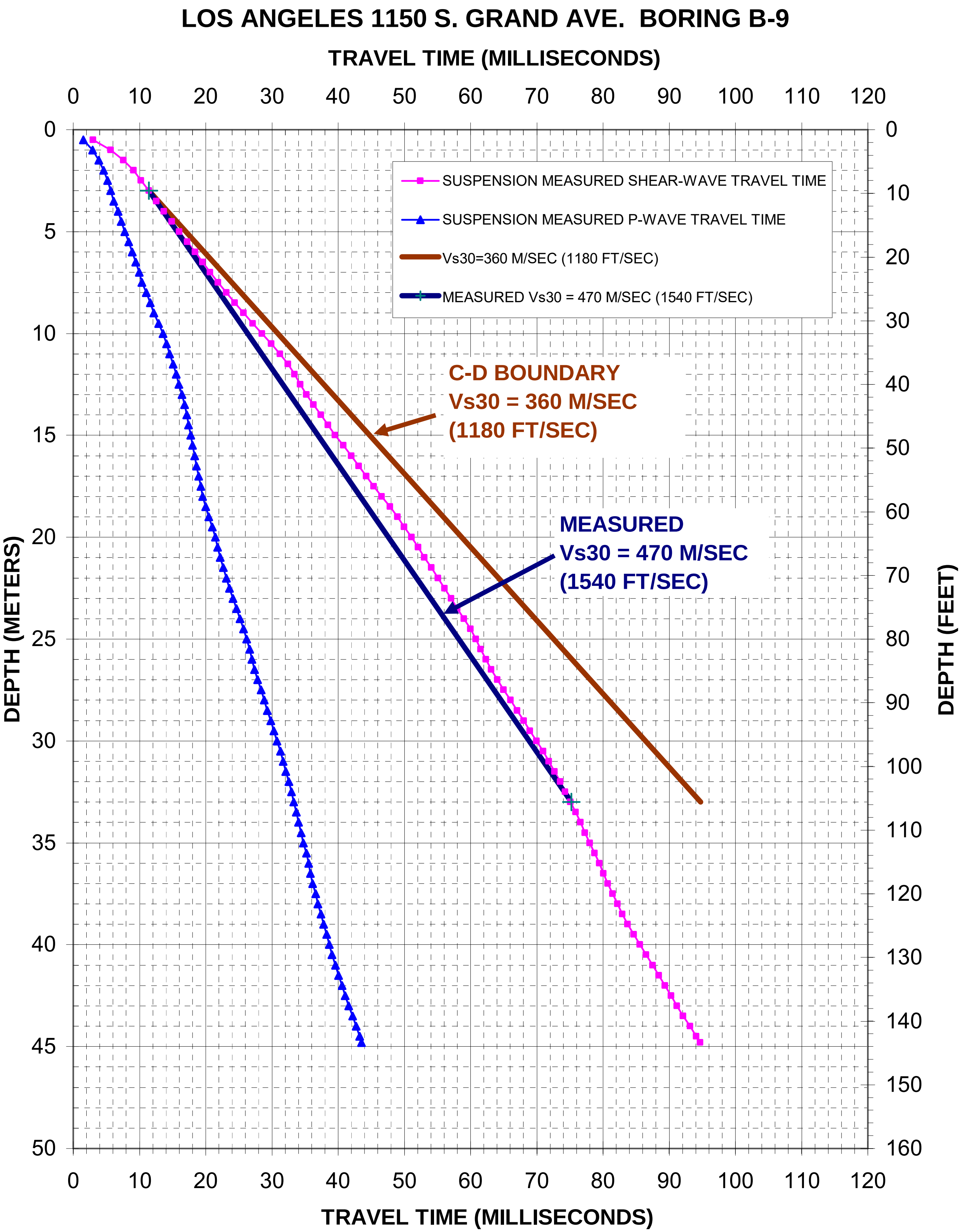

Figure 5. Boring B-9, Suspension P- and $S_H$-wave travel times with Vs30 values

# APPENDIX A

# SUSPENSION VELOCITY MEASUREMENT
# QUALITY ASSURANCE SUSPENSION SOURCE
# TO RECEIVER ANALYSIS RESULTS

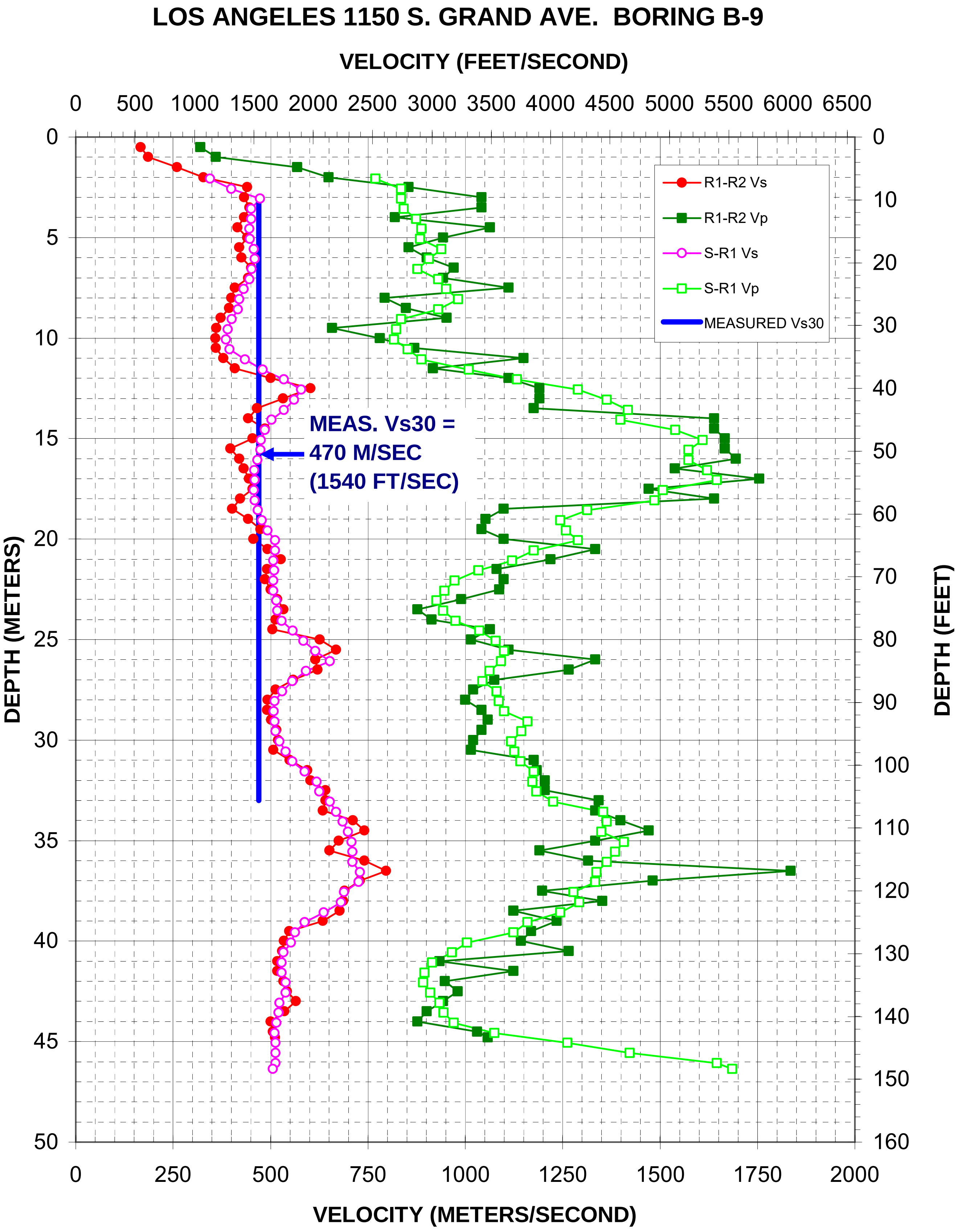

Figure A-1. Boring B-9, R1 - R2 high resolution analysis and S-R1 quality assurance analysis P- and $S_H$-wave data

|  | Velocity |  |  | Velocity |  |
|---|---|---|---|---|---|
| Depth (meters) | V-$S_H$ (m/sec) | V-p (m/sec) | Depth (feet) | V- $S_H$ (ft/sec) | V-p (ft/sec) |
| 2.1 | 345 | 770 | 6.79 | 1132 | 2526 |
| 2.6 | 400 | 836 | 8.43 | 1312 | 2743 |
| 3.1 | 473 | 836 | 10.07 | 1553 | 2743 |
| 3.6 | 451 | 843 | 11.71 | 1478 | 2764 |
| 4.1 | 451 | 873 | 13.35 | 1478 | 2866 |
| 4.6 | 446 | 888 | 14.99 | 1463 | 2913 |
| 5.1 | 448 | 884 | 16.63 | 1469 | 2901 |
| 5.6 | 457 | 939 | 18.27 | 1500 | 3079 |
| 6.1 | 459 | 907 | 19.91 | 1507 | 2975 |
| 6.6 | 451 | 877 | 21.56 | 1481 | 2877 |
| 7.1 | 446 | 930 | 23.20 | 1463 | 3053 |
| 7.6 | 431 | 951 | 24.84 | 1416 | 3120 |
| 8.1 | 420 | 982 | 26.48 | 1377 | 3221 |
| 8.6 | 416 | 930 | 28.12 | 1366 | 3053 |
| 9.1 | 401 | 836 | 29.76 | 1315 | 2743 |
| 9.6 | 391 | 823 | 31.40 | 1281 | 2700 |
| 10.1 | 386 | 817 | 33.04 | 1267 | 2680 |
| 10.6 | 395 | 853 | 34.68 | 1295 | 2797 |
| 11.1 | 435 | 888 | 36.32 | 1427 | 2913 |
| 11.6 | 480 | 1009 | 37.96 | 1574 | 3312 |
| 12.1 | 535 | 1132 | 39.60 | 1755 | 3715 |
| 12.6 | 578 | 1289 | 41.24 | 1898 | 4230 |
| 13.1 | 560 | 1363 | 42.88 | 1838 | 4472 |
| 13.6 | 535 | 1417 | 44.52 | 1755 | 4650 |
| 14.1 | 502 | 1399 | 46.16 | 1648 | 4589 |
| 14.6 | 485 | 1540 | 47.80 | 1592 | 5051 |
| 15.1 | 476 | 1609 | 49.44 | 1560 | 5279 |
| 15.6 | 475 | 1574 | 51.08 | 1557 | 5162 |
| 16.1 | 466 | 1574 | 52.72 | 1530 | 5162 |
| 16.6 | 458 | 1621 | 54.36 | 1503 | 5319 |
| 17.1 | 459 | 1646 | 56.00 | 1507 | 5401 |
| 17.6 | 457 | 1507 | 57.64 | 1500 | 4944 |
| 18.1 | 459 | 1486 | 59.28 | 1507 | 4876 |
| 18.6 | 467 | 1313 | 60.93 | 1533 | 4307 |
| 19.1 | 478 | 1244 | 62.57 | 1567 | 4082 |
| 19.6 | 493 | 1259 | 64.21 | 1618 | 4130 |
| 20.1 | 512 | 1289 | 65.85 | 1680 | 4230 |
| 20.6 | 512 | 1176 | 67.49 | 1680 | 3858 |
| 21.1 | 507 | 1120 | 69.13 | 1664 | 3676 |
| 21.6 | 510 | 1034 | 70.77 | 1672 | 3392 |
| 22.1 | 507 | 973 | 72.41 | 1664 | 3191 |
| 22.6 | 507 | 947 | 74.05 | 1664 | 3107 |
| 23.1 | 516 | 926 | 75.69 | 1692 | 3039 |
| 23.6 | 518 | 943 | 77.33 | 1700 | 3093 |
| 24.1 | 528 | 975 | 78.97 | 1734 | 3199 |
| 24.6 | 557 | 1036 | 80.61 | 1828 | 3400 |
| 25.1 | 585 | 1078 | 82.25 | 1918 | 3537 |
| 25.6 | 615 | 1100 | 83.89 | 2018 | 3610 |
| 26.1 | 652 | 1092 | 85.53 | 2141 | 3582 |
| 26.6 | 591 | 1062 | 87.17 | 1940 | 3484 |
| 27.1 | 556 | 1044 | 88.81 | 1824 | 3425 |
| 27.6 | 530 | 1081 | 90.45 | 1740 | 3546 |
| 28.1 | 511 | 1086 | 92.09 | 1676 | 3564 |
| 28.6 | 508 | 1100 | 93.73 | 1668 | 3610 |
| 29.1 | 511 | 1160 | 95.37 | 1676 | 3805 |
| 29.6 | 513 | 1144 | 97.01 | 1684 | 3755 |
| 30.1 | 523 | 1117 | 98.65 | 1717 | 3666 |
| 30.6 | 539 | 1126 | 100.30 | 1769 | 3695 |
| 31.1 | 556 | 1141 | 101.94 | 1824 | 3745 |
| 31.6 | 588 | 1176 | 103.58 | 1929 | 3858 |
| 32.1 | 618 | 1173 | 105.22 | 2029 | 3847 |
| 32.6 | 626 | 1182 | 106.86 | 2053 | 3879 |
| 33.1 | 652 | 1226 | 108.50 | 2141 | 4023 |
| 33.6 | 669 | 1354 | 110.14 | 2194 | 4444 |
| 34.1 | 686 | 1363 | 111.78 | 2250 | 4472 |
| 34.6 | 699 | 1350 | 113.42 | 2294 | 4430 |
| 35.1 | 709 | 1408 | 115.06 | 2325 | 4619 |
| 35.6 | 711 | 1385 | 116.70 | 2333 | 4544 |
| 36.1 | 711 | 1363 | 118.34 | 2333 | 4472 |
| 36.6 | 730 | 1338 | 119.98 | 2396 | 4388 |
| 37.1 | 727 | 1333 | 121.62 | 2384 | 4374 |
| 37.6 | 689 | 1278 | 123.26 | 2261 | 4192 |
| 38.1 | 682 | 1293 | 124.90 | 2236 | 4242 |
| 38.6 | 637 | 1244 | 126.54 | 2090 | 4082 |
| 39.1 | 588 | 1160 | 128.18 | 1929 | 3805 |
| 39.6 | 563 | 1123 | 129.82 | 1848 | 3686 |
| 40.1 | 553 | 1005 | 131.46 | 1814 | 3296 |
| 40.6 | 534 | 966 | 133.10 | 1751 | 3170 |
| 41.1 | 528 | 915 | 134.74 | 1734 | 3000 |
| 41.6 | 528 | 895 | 136.38 | 1734 | 2938 |
| 42.1 | 539 | 892 | 138.02 | 1769 | 2925 |
| 42.6 | 539 | 911 | 139.67 | 1769 | 2988 |
| 43.1 | 523 | 934 | 141.31 | 1717 | 3066 |
| 43.6 | 521 | 945 | 142.95 | 1708 | 3100 |
| 44.1 | 516 | 971 | 144.59 | 1692 | 3184 |
| 44.6 | 511 | 1075 | 146.23 | 1676 | 3528 |
| 45.1 | 513 | 1263 | 147.87 | 1684 | 4142 |
| 45.6 | 513 | 1422 | 149.51 | 1684 | 4665 |
| 46.1 | 513 | 1646 | 151.15 | 1684 | 5401 |
| 46.4 | 506 | 1685 | 152.13 | 1660 | 5528 |

Table A-1. Boring B-9, S - R1 quality assurance analysis P- and $S_H$-wave data

# APPENDIX B

# OYO 170 VELOCITY LOGGING SYSTEM
# NIST TRACEABLE
# CALIBRATION PROCEDURE

# CALIBRATION PROCEDURE FOR

# GEOVision SEISMIC RECORDER/LOGGER

Reviewed 4/6/06

## Objective

The timing/sampling accuracy of seismic recorders or data loggers is required for several GEOVision field procedures including Seismic Refraction, Downhole Seismic Velocity Logging, and P-S Suspension Logging. This procedure describes the method for measuring the timing accuracy of a seismic data logger, such as the OYO Model 170, OYO/Robertson Model 3403, Geometrics Strataview or Geometrics Geode. The objective of this procedure is to verify that the timing accuracy of the recorder is accurate to within 1%.

## Frequency of Calibration

The calibration of each GEOVision seismic data logger is twelve (12) months. In the case of rented seismic data loggers, calibration must be performed prior to use.

## Test Equipment Required

The following equipment is required. Item #2 must have current NIST traceable calibration.

1. Function generator, Krohn Hite 5400B or equivalent

2. Frequency counter, HP 5315A or equivalent

3. Test cables, from item 1 to item 2, and from item 1 to subject data logger.

## Procedure

This procedure is designed to be performed using the accompanying Seismograph Calibration Data Sheet with the same revision number. All data must be entered and the procedure signed by the technician performing the test.

1. Record all identification data on the form provided.

2. Connect function generator to data logger (such as OYO Model 170) using test cable

3. Connect the function generator to the frequency counter using test cable.

4. Set up generator to produce a 100.0 Hz, 0.25 volt (amplitude is approximate, modify as necessary to yield less than full scale waveforms on logger display) peak square wave or sine wave. Verify frequency using the counter and initial space on the data sheet.

5. Initialize data logger and record a data record of at least 0.1 second using a 100 microsecond or less sample period.

6. Measure the recorded square wave frequency by measuring the duration of 9 cycles of data. This measurement can be made using the data logger display device, or by printing out a paper tape. If a paper tape can be printed, the resulting printout must be attached to this procedure. Record the data in the space provided.

7. Repeat steps 5 and 6 three more times using separate files.

## Criteria

The duration for 9 cycles in any file must be 90.0 milliseconds plus or minus 0.9 milliseconds, corresponding to an average frequency for the nine cycles of 100.0 Hz plus or minus 1 Hz (obtained by dividing 9 cycles by the duration in milliseconds).

If the results are outside this range, the data logger must be marked with a GEOVision REJECT tag until it can be repaired and retested.

If results are acceptable affix label indicating the initials of the person performing the calibration, the date of calibration, and the due date for the next calibration (12 months).

## Procedure Approval

Approved by:

John G. Diehl

Name

President

Title

Signature

April 6, 2006

Date

Client Approval (if required):

Name

Title

Signature

Date

EDISON ESI℠
A SOUTHERN CALIFORNIA EDISON® Company
**Metrology**
7300 Fenwick Lane
Westminster, CA 92683
Phone: 866-723-2257

# Calibration Report

*NVLAP Accredited*
*Calibration*

**GEOVision Geophysical Services**
1151 Pomona Road, Unit P
Corona, CA 92882

TEST NUMBER
527436

NVLAP®
Lab Code:105014-0

**Manufacturer:** Oyo
**Model Number:** 03331-0000
**Description:** Seismograph,
**Asset Number:** 15014
**Serial Number:** 15014
**PO Number:** 7087-070315-01

**Condition As Found:** In Tolerance
**Condition As Left:** In Tolerance
**Calibration Date:** 04/13/2007
**Calibration Due Date:** 04/13/2008
**Calibration Interval:** 12 Months

**Remarks:**

The UUT (unit under test) was calibrated using the customer's procedure. The UUT was operated by the customer's personnel and data collection was observed by SCE personnel. The UUT was found to be in tolerance to customer supplied specifications. Frequency is accredited. Please see attached data.

## Standards Utilized

| I.D. No. | Mfg. | Model No. | Description | Cal. Date | Due Date |
|---|---|---|---|---|---|
| S1-03092 | Hewlett Packard | 5335A OPT 020, 30,40 | Counter, Universal, | 12/12/2006 | 06/12/2007 |
| S1-03355 | Hewlett Packard | 3325B OPT 001, 002 | Generator, Function, Synthesizer | 11/08/2006 | 11/08/2007 |
| S1-03686 | Fluke | 910 | Standard, Frequency, Controlled, Gps | 01/18/2007 | 01/18/2008 |

**Procedure:** Customer
**Temperature:** 23° C
**Humidity:** 38% RH
**Test No.:** 527436

| Calibration Performed By: | | | Quality Reviewer: | |
|---|---|---|---|---|
| Branson, Craig A | Metrologist | 714-895-0714 | | 4/13/07 |
| Name | Title | Phone | Name | Date |

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GEOVision
geophysical services

# SEISMOGRAPH CALIBRATION DATA SHEET REV 4/6/06

**INSTRUMENT DATA**

SYSTEM MFR: OYO MODEL NO.: 3331
SERIAL NO.: 15014 CALIBRATION DATE: 04/13/2007
BY: ROBERT STELLER DUE DATE: 04/13/2008

COUNTER MFR: HEWLETT PACKARD MODEL NO.: 5335A
SERIAL NO.: 2626A10854 CALIBRATION DATE: 12/12/2006
BY: SCE #S1-03092 DUE DATE: 06/12/2007

FCTN GEN MFR: HEWLETT PACKARD MODEL NO.: 3325B
SERIAL NO.: 2847A14447 CALIBRATION DATE: 11/08/2006
BY: SCE #S1-03355 DUE DATE: 11/08/2007

**SYSTEM SETTINGS:**

GAIN: 10
FILTER: 20 KHZ
RANGE: 100 MILLISEC
DELAY: 0
STACK: 1 (STD) 1
PULSE: 1.6
DISPLAY: NA
SYSTEM: DATE = CORRECT DATE & TIME 04/13/2007, 09:21AM

**PROCEDURE:**

SET FREQUENCY TO 100.0HZ SQUAREWAVE WITH AMPLITUDE APPROXIMATELY 0.25 VOLT PEAK. RECORD BOTH ON DISK AND PAPER TAPE, IF AVAILABLE. ANALYZE AND PRINT WAVEFORMS FROM ANALYSIS UTILITY. ATTACH PAPER COPIES OF PRINTOUT AND PAPER TAPES, IF AVAILABLE, TO THIS FORM. AVERAGE FREQUENCY MUST BE BETWEEN 99.0 AND 101.0 HZ.

AS FOUND 100.0 AS LEFT 100.0

| WAVEFORM | FILE NO | FREQUENCY | TIME FOR 9 CYCLES Hn | TIME FOR 9 CYCLES Hr | TIME FOR 9 CYCLES V | AVERAGE FREQ. |
|---|---|---|---|---|---|---|
| SQUARE | 201 | 100.0 | 90.0 | 90.0 | 90.0 | 100.0 |
| SQUARE | 202 | 100.0 | 90.0 | 90.0 | 90.0 | 100.0 |
| SINE | 203 | 100.0 | 90.0 | 90.0 | 90.0 | 100.0 |
| SINE | 204 | 100.0 | 90.0 | 90.0 | 90.0 | 100.0 |

CALIBRATED BY: ROBERT STELLER 04/13/2007
NAME DATE SIGNATURE

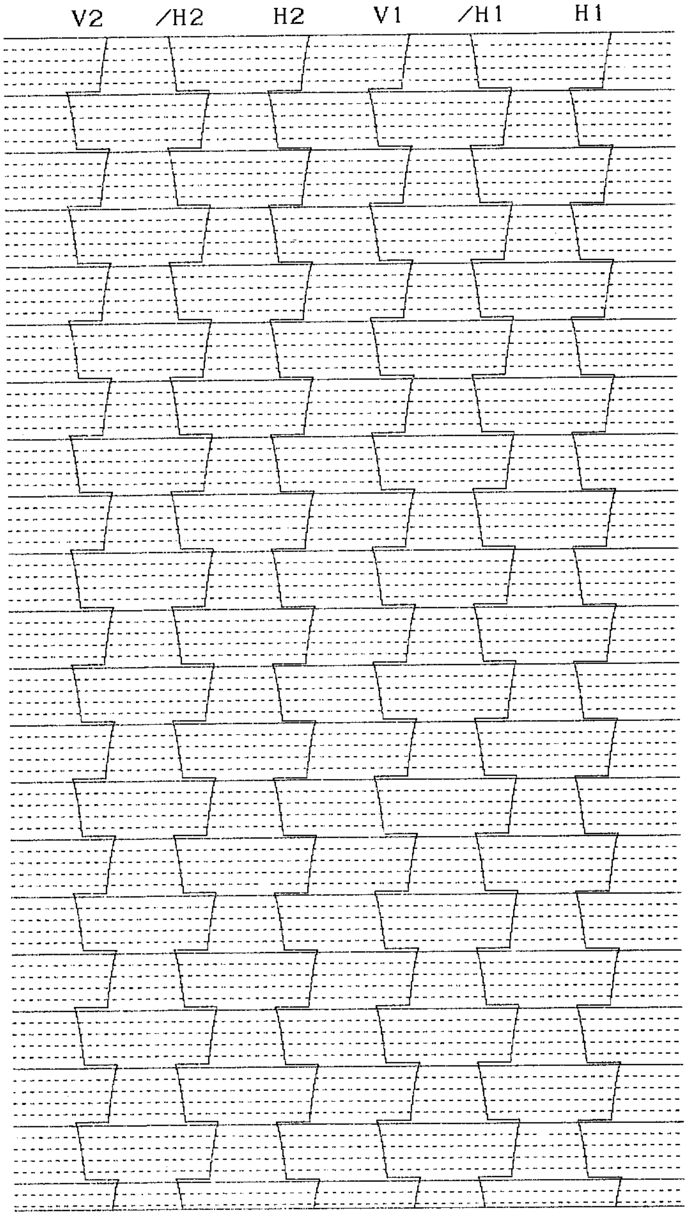
OYO S/N 15014

```
Suspension 170 1.42

ID_NO.       : 201
HOLE NO.     :   0
DEPTH        :     0.0 [m]
DATE         : 13/04/07  09:23:34 AM
H-SAMPLE RATE: 100 [μSEC]
V-SAMPLE RATE: 100 [μSEC]
PULSE WIDTH  : 1.6 [mSEC]
DELAY TIME   :   0 [mSEC]

              H1   /H1    V1    H2   /H2    V2
            ----------------------------------
GAIN        :X 10 X 10 X 10 X 10 X 10 X 10
LCF[Hz]     :   5     5     5     5     5     5
HCF[Hz]     : 20K   20K   20K   20K   20K   20K
STACK       :   1     1     1     1     1     1

TRACE SIZE   :  1
H-TIME SCALE: 1.00 [mSEC/LINE]
V-TIME SCALE: 1.00 [mSEC/LINE]
```



---

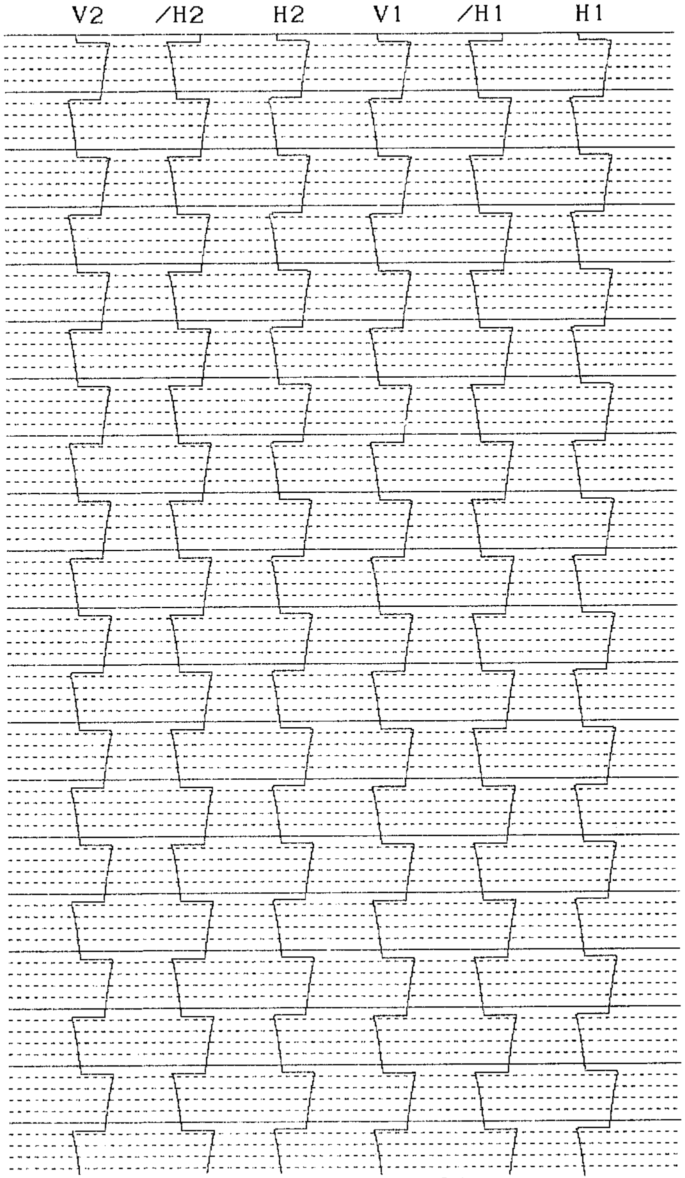
OYO S/N 15014

```
Suspension 170 1.42

ID_NO.       : 202
HOLE NO.     :   0
DEPTH        :     0.0 [m]
DATE         : 13/04/07  09:24:47 AM
H-SAMPLE RATE: 100 [μSEC]
V-SAMPLE RATE: 100 [μSEC]
PULSE WIDTH  : 1.6 [mSEC]
DELAY TIME   :   0 [mSEC]

              H1   /H1    V1    H2   /H2    V2
            ----------------------------------
GAIN        :X 10 X 10 X 10 X 10 X 10 X 10
LCF[Hz]     :   5     5     5     5     5     5
HCF[Hz]     : 20K   20K   20K   20K   20K   20K
STACK       :   1     1     1     1     1     1

TRACE SIZE   :  1
H-TIME SCALE: 1.00 [mSEC/LINE]
V-TIME SCALE: 1.00 [mSEC/LINE]
```



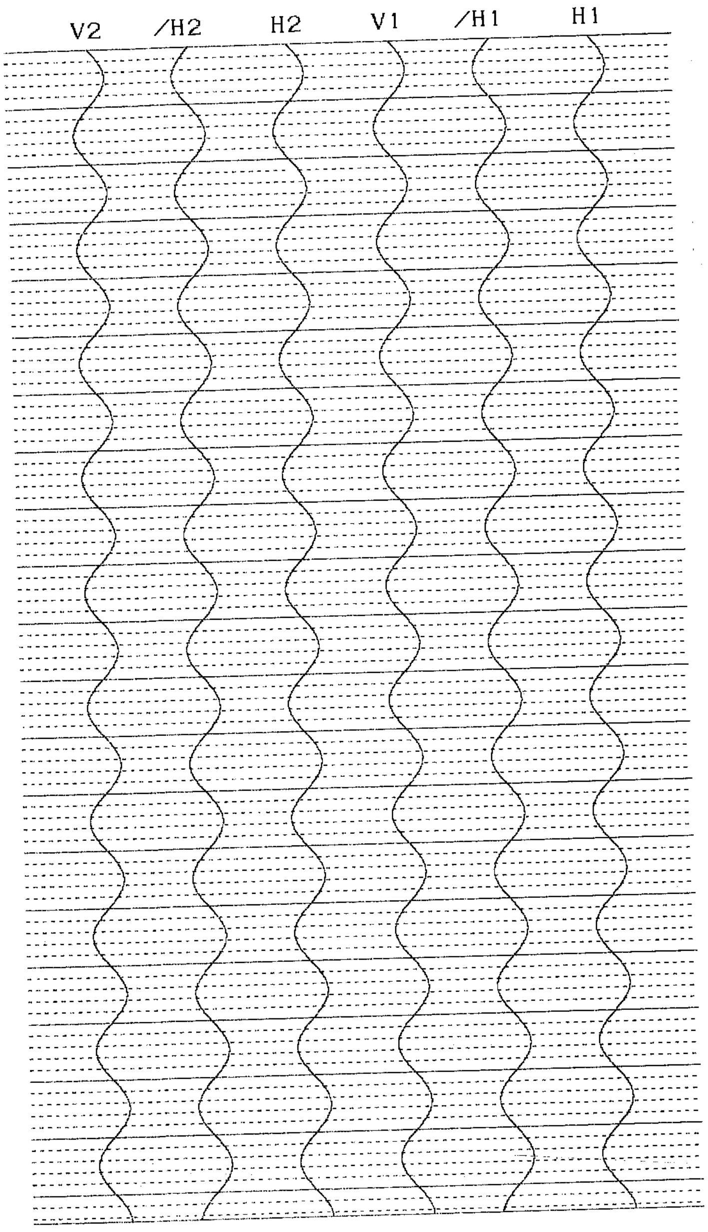
OYO S/N 15014

Suspension 170 1.42

ID_NO. : 203
HOLE NO. : 0
DEPTH : 0.0 [m]
DATE : 13/04/07 09:25:23 AM
H-SAMPLE RATE: 100 [μSEC]
V-SAMPLE RATE: 100 [μSEC]
PULSE WIDTH : 1.6 [mSEC]
DELAY TIME : 0 [mSEC]

| | H1 | /H1 | V1 | H2 | /H2 | V2 |
|---|---|---|---|---|---|---|
| GAIN | :X 10 | X 10 | X 10 | X 10 | X 10 | X 10 |
| LCF [Hz] | : 5 | 5 | 5 | 5 | 5 | 5 |
| HCF [Hz] | : 20K | 20K | 20K | 20K | 20K | 20K |
| STACK | : 1 | 1 | 1 | 1 | 1 | 1 |

TRACE SIZE : 1
H-TIME SCALE: 1.00 [mSEC/LINE]
V-TIME SCALE: 1.00 [mSEC/LINE]



---

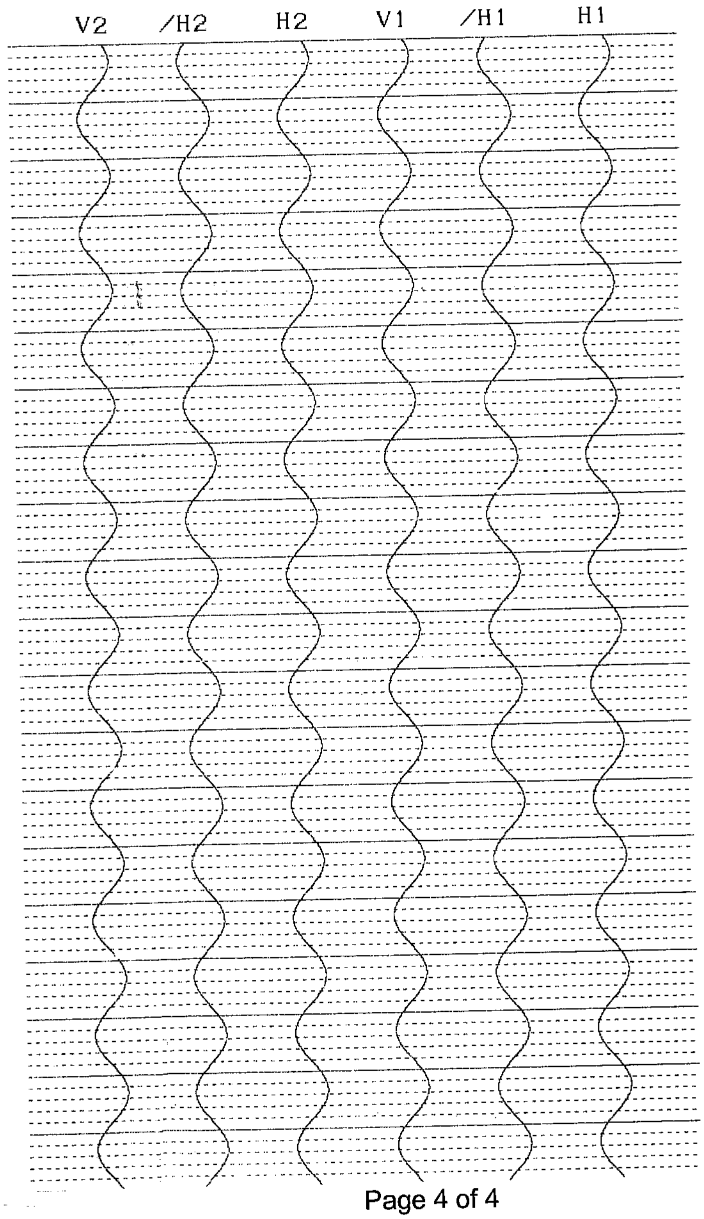
OYO S/N 15014

Suspension 170 1.42

ID_NO. : 204
HOLE NO. : 0
DEPTH : 0.0 [m]
DATE : 13/04/07 09:26:03 AM
H-SAMPLE RATE: 100 [μSEC]
V-SAMPLE RATE: 100 [μSEC]
PULSE WIDTH : 1.6 [mSEC]
DELAY TIME : 0 [mSEC]

| | H1 | /H1 | V1 | H2 | /H2 | V2 |
|---|---|---|---|---|---|---|
| GAIN | :X 10 | X 10 | X 10 | X 10 | X 10 | X 10 |
| LCF [Hz] | : 5 | 5 | 5 | 5 | 5 | 5 |
| HCF [Hz] | : 20K | 20K | 20K | 20K | 20K | 20K |
| STACK | : 1 | 1 | 1 | 1 | 1 | 1 |

TRACE SIZE : 1
H-TIME SCALE: 1.00 [mSEC/LINE]
V-TIME SCALE: 1.00 [mSEC/LINE]



# GEOVision Suspension PS probe Receiver 1–Receiver 2 (R1-R2) spacing verification

Performed by Robert Steller on September 23, 2006

| | R2 center to R1 center hanging dry | R2 center to R1 center hanging submerged | R1 bottom to source center hanging submerged with 1m isolation tube S/N 280068 |
|---|---|---|---|
| Receiver S/N 30086 | 40.2in<br>1.02m | 40.0in<br>1.02m | 76.0in<br>1.93m |
| | | | |
| Receiver S/N 20042 | 39.8in<br>1.01m | 39.6in<br>1.01m | 75.7in<br>1.92m |
| | | | |
| Receiver S/N 12008 | 40.2in<br>1.02m | 40.0in<br>1.02m | 76.0in<br>1.93m |
| | | | |
| | | | |
| | | | |

All measurements taken with a Lufkin 3.7m flexible steel tape model number HV1034DM, marked in mm and 100th of feet. Probe suspended in 3-inch diameter clear PVC pipe, using chain clamp placed between bottom and center of Receiver 2 hard section (See Figure). Probe "bounced" to establish unrestricted hanging length before measurement. Probe allowed to relax for 5 minutes prior to each measurement. Water level set to submerge bottom of Receiver 2 hard section.. Estimated accuracy due to hysterisis in rubber section approximately +/- 0.01' or +/- 0.003m.

# DTLA SITE 3-1-01

# BOREHOLE B-8

# SUSPENSION PS VELOCITIES

**December 06, 2017**

**Report 17436-01 rev 0**

# DTLA SITE 3-1-01

# BOREHOLE B-8

# SUSPENSION PS VELOCITIES

**Prepared for**

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**Project 16139**

**December 06, 2017**

**Report 17436-01 rev 0**

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## APPENDICES

## INTRODUCTION

**GEO***Vision* acquired borehole geophysical data in one borehole at 1100 Olive St. in Los Angeles, California. The work was performed for GeoDesign, Inc. Data, analysis and report were reviewed by a **GEO***Vision* Professional Geophysicist or Engineer.

## SCOPE OF WORK

This report presents results of Suspension PS velocity data acquired in one borehole on November 30, 2017 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 PS 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 and compressional-wave source, 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 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 at least every twelve months using a NIST traceable frequency source and counter, as presented in Appendix B.

# MEASUREMENT PROCEDURES

## Suspension Velocity Measurement Procedures

Borehole B-8 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 file accompanies 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:

$$\boldsymbol{\nu} \;=\; \frac{\left(\frac{V_s}{V_p}\right)^2 - 0.5}{\left(\frac{V_s}{V_p}\right)^2 - 1.0}$$

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.

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

# RESULTS

## Suspension Velocity Results

Suspension R1-R2 P- and $S_H$-wave velocities for borehole B-8 are plotted in Figure 4 and 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 displayed in this figure 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. Drilling fluid could not be maintained above 6.5 feet, thus data could not be acquired above this depth.

Overall, Suspension PS velocity data quality is judged on 5 criteria, as summarized below.

| | **Criteria** | **B-08** |
|---|---|---|
| **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<br>Saturation in B-08 occurs at about 145 ft. |
| **4** | Clarity of P-wave and $S_H$-wave onset, as well as damping of later oscillations. | Generally S-wave and P-wave onsets were clear with P-wave arrivals slightly more difficult to identify near ground surface. |
| **5** | Consistency of profile between adjacent borings, if available. | NA |

## Quality Assurance

These borehole geophysical measurements were performed using industry-standard or better methods for measurements and analysis. 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

J Jordan

12/06/2017

J. Jonathan Jordan
Sr Staff Geophysicist
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Date

Reviewed and approved by

VICTOR GONZALEZ
PGp 1074

12/06/2017

Victor Gonzalez
California Professional Geophysicist, PGp. 1074
**GEO***Vision* Geophysical Services

Date

* 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 DESIGNATION | DATES LOGGED | COORDINATES | | ELEVATION (TOP OF WELL CASING) (1) (FEET) |
|---|---|---|---|---|
| | | LATITUDE | LONGITUDE | |
| B-8 | 11/30/2017 | 34.04038 | -118.26085 | |

(1) Coordinates estimated from phone GPS. Elevation not provided

## Table 2. Logging dates and depth ranges

| BOREHOLE NUMBER | TOOL AND RUN NUMBER | DEPTH RANGE (FEET) | OPEN HOLE (FEET) | SAMPLE INTERVAL (FEET) | DATE LOGGED |
|---|---|---|---|---|---|
| B-8 | SUSPENSION DOWN01 | 1.6 – 187.0 | 200 | 1.6 | 11/30/2017 |

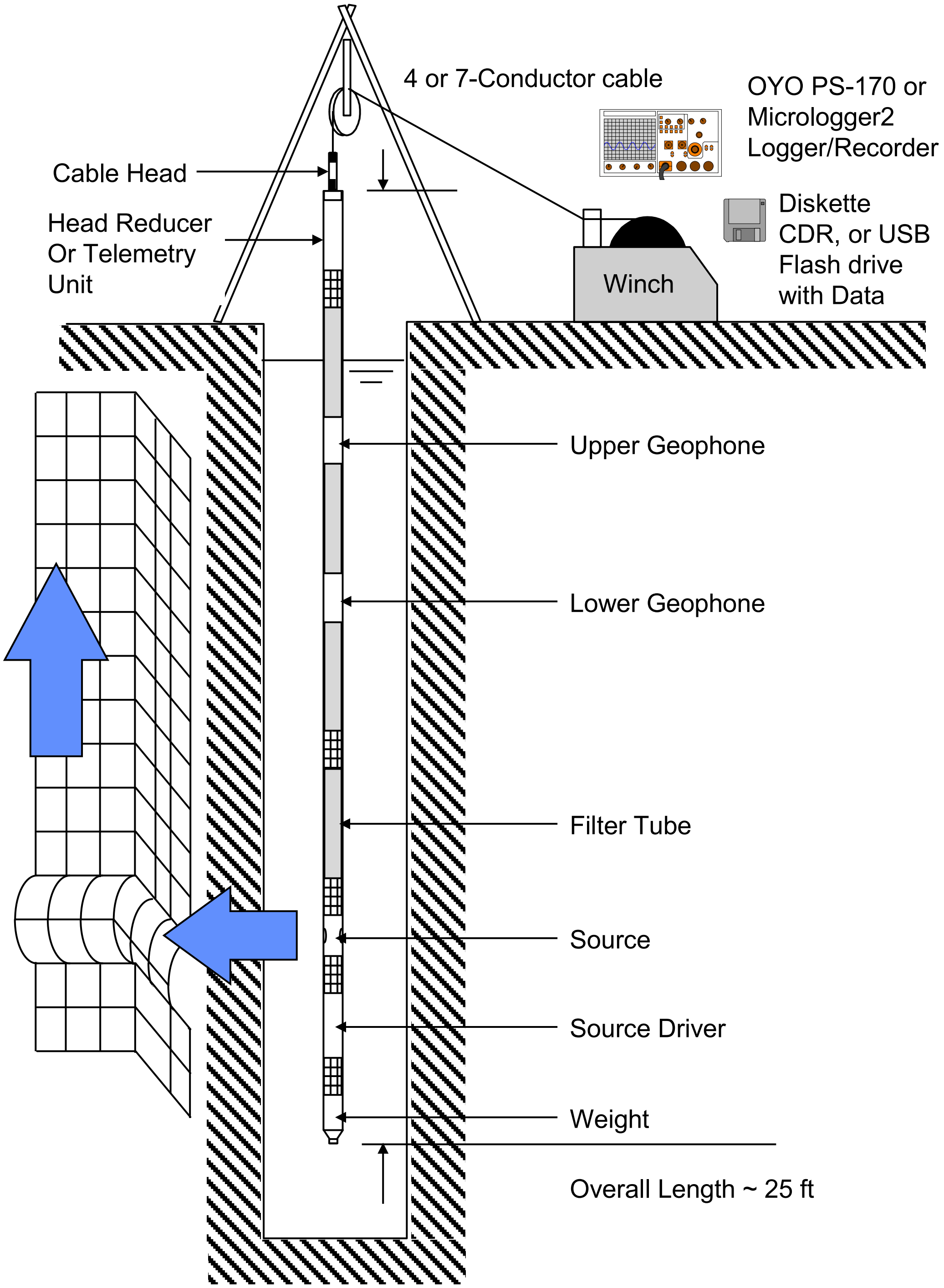

Figure 1: Concept illustration of P-S logging system

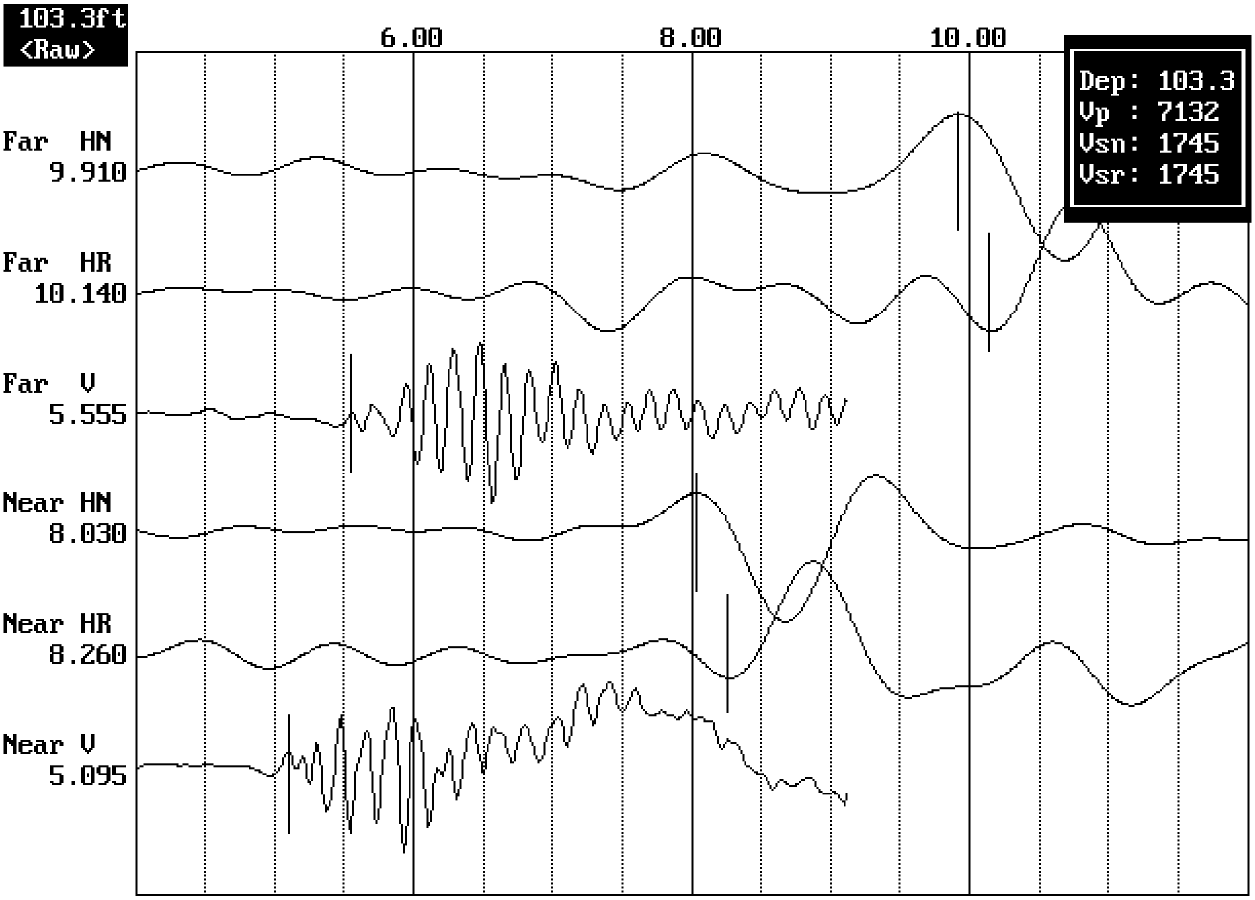

Figure 2: Example of filtered (1400 Hz lowpass) suspension record

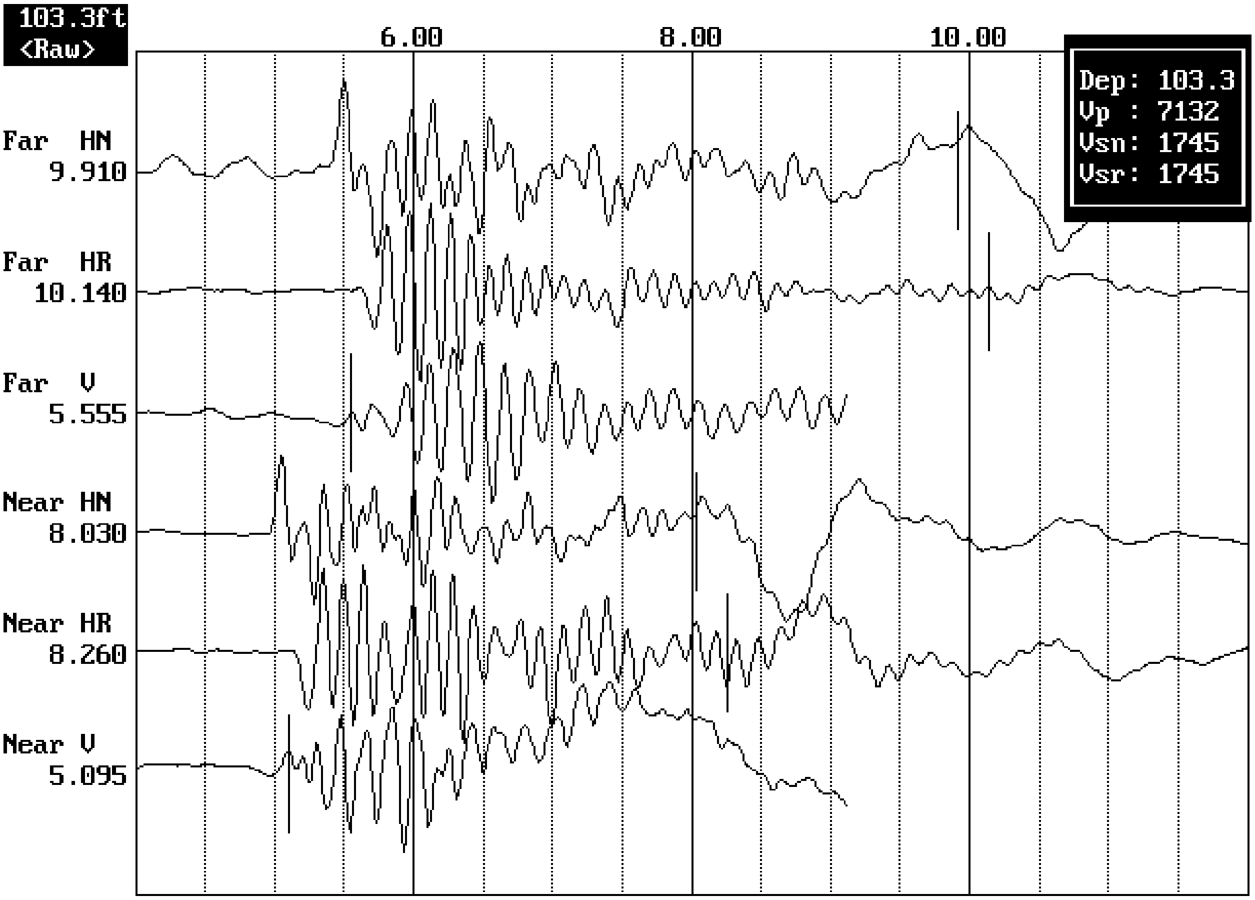

Figure 3. Example of unfiltered suspension record

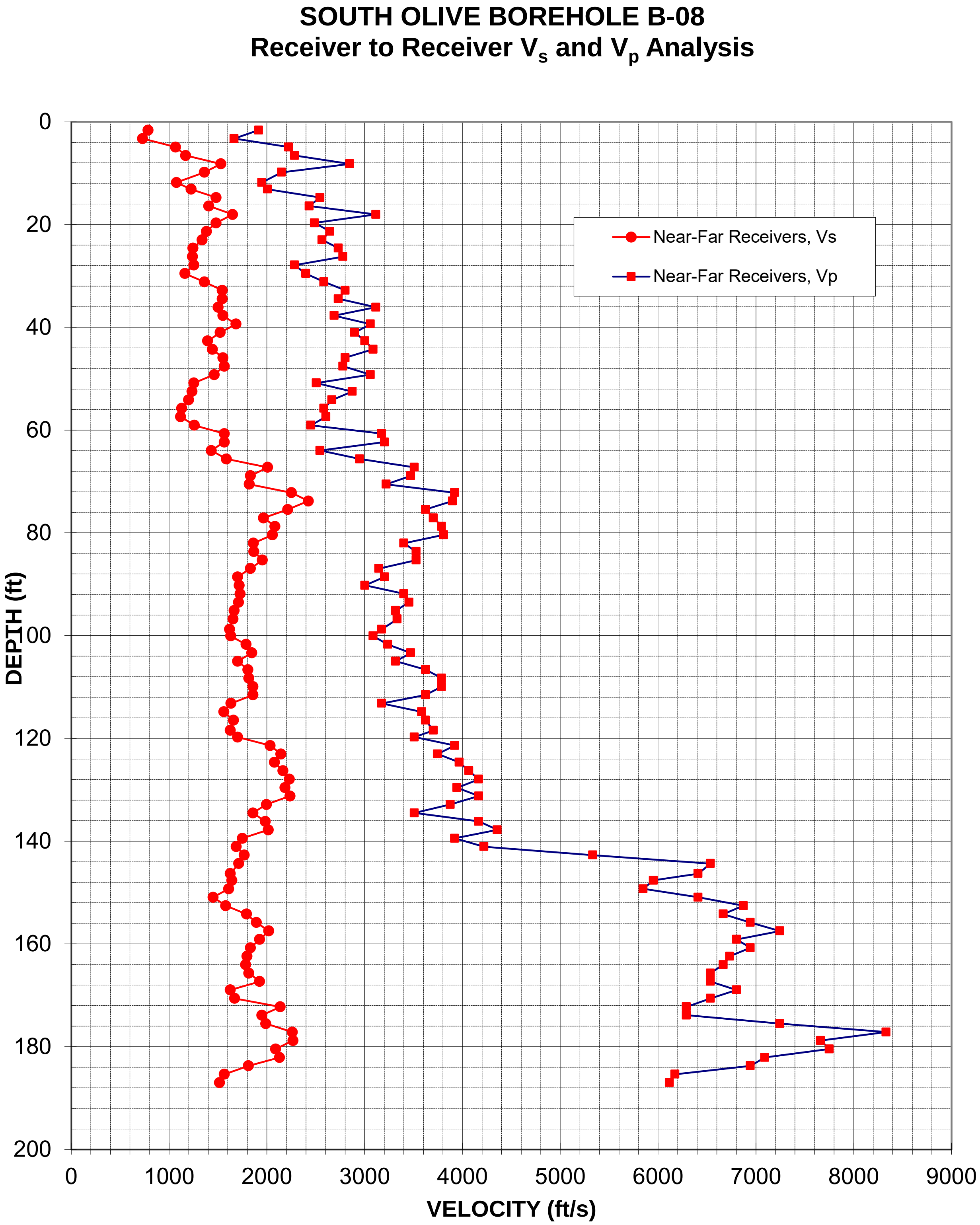

Figure 4: Borehole B-8, Suspension R1-R2 P- and $S_H$-wave velocities

## Table 3. Borehole B-8, 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-8**

| American Units | | | |
|---|---|---|---|
| Depth at Midpoint Between Receivers | Velocity | | Poisson's Ratio |
| | $V_s$ | $V_p$ | |
| (ft) | (ft/s) | (ft/s) | |
| 1.6 | 780 | 1920 | 0.40 |
| 3.3 | 730 | 1670 | 0.38 |
| 4.9 | 1070 | 2220 | 0.35 |
| 6.6 | 1170 | 2280 | 0.32 |
| 8.2 | 1530 | 2850 | 0.30 |
| 9.8 | 1360 | 2150 | 0.17 |
| 11.8 | 1080 | 1950 | 0.28 |
| 13.1 | 1230 | 2010 | 0.20 |
| 14.8 | 1480 | 2540 | 0.24 |
| 16.4 | 1410 | 2430 | 0.25 |
| 18.0 | 1650 | 3120 | 0.30 |
| 19.7 | 1480 | 2490 | 0.23 |
| 21.3 | 1380 | 2650 | 0.31 |
| 23.0 | 1340 | 2560 | 0.31 |
| 24.6 | 1240 | 2730 | 0.37 |
| 26.3 | 1240 | 2780 | 0.38 |
| 27.9 | 1250 | 2280 | 0.28 |
| 29.5 | 1160 | 2400 | 0.35 |
| 31.2 | 1360 | 2580 | 0.31 |
| 32.8 | 1540 | 2800 | 0.28 |
| 34.5 | 1540 | 2730 | 0.27 |
| 36.1 | 1500 | 3120 | 0.35 |
| 37.7 | 1550 | 2690 | 0.25 |
| 39.4 | 1680 | 3060 | 0.28 |
| 41.0 | 1520 | 2900 | 0.31 |
| 42.7 | 1390 | 3000 | 0.36 |
| 44.3 | 1440 | 3090 | 0.36 |
| 45.9 | 1550 | 2800 | 0.28 |
| 47.6 | 1560 | 2780 | 0.27 |
| 49.2 | 1460 | 3060 | 0.35 |
| 50.9 | 1250 | 2510 | 0.33 |
| 52.5 | 1230 | 2870 | 0.39 |
| 54.1 | 1200 | 2670 | 0.37 |
| 55.8 | 1130 | 2580 | 0.38 |
| 57.4 | 1120 | 2600 | 0.39 |
| 59.1 | 1260 | 2450 | 0.32 |

| Metric Units | | | |
|---|---|---|---|
| Depth at Midpoint Between Receivers | Velocity | | Poisson's Ratio |
| | $V_s$ | $V_p$ | |
| (m) | (m/s) | (m/s) | |
| 0.5 | 240 | 580 | 0.40 |
| 1.0 | 220 | 510 | 0.38 |
| 1.5 | 330 | 680 | 0.35 |
| 2.0 | 360 | 700 | 0.32 |
| 2.5 | 470 | 870 | 0.30 |
| 3.0 | 410 | 660 | 0.17 |
| 3.6 | 330 | 590 | 0.28 |
| 4.0 | 370 | 610 | 0.20 |
| 4.5 | 450 | 780 | 0.24 |
| 5.0 | 430 | 740 | 0.25 |
| 5.5 | 500 | 950 | 0.30 |
| 6.0 | 450 | 760 | 0.23 |
| 6.5 | 420 | 810 | 0.31 |
| 7.0 | 410 | 780 | 0.31 |
| 7.5 | 380 | 830 | 0.37 |
| 8.0 | 380 | 850 | 0.38 |
| 8.5 | 380 | 700 | 0.28 |
| 9.0 | 350 | 730 | 0.35 |
| 9.5 | 410 | 790 | 0.31 |
| 10.0 | 470 | 850 | 0.28 |
| 10.5 | 470 | 830 | 0.27 |
| 11.0 | 460 | 950 | 0.35 |
| 11.5 | 470 | 820 | 0.25 |
| 12.0 | 510 | 930 | 0.28 |
| 12.5 | 460 | 880 | 0.31 |
| 13.0 | 430 | 920 | 0.36 |
| 13.5 | 440 | 940 | 0.36 |
| 14.0 | 470 | 850 | 0.28 |
| 14.5 | 480 | 850 | 0.27 |
| 15.0 | 450 | 930 | 0.35 |
| 15.5 | 380 | 760 | 0.33 |
| 16.0 | 380 | 880 | 0.39 |
| 16.5 | 370 | 810 | 0.37 |
| 17.0 | 340 | 790 | 0.38 |
| 17.5 | 340 | 790 | 0.39 |
| 18.0 | 380 | 750 | 0.32 |

## Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio
## Based on Receiver-to-Receiver Travel Time Data - Borehole B-8

| American Units | | | |
|---|---|---|---|
| Depth at Midpoint Between Receivers | Velocity | | Poisson's Ratio |
| | $V_s$ | $V_p$ | |
| (ft) | (ft/s) | (ft/s) | |
| 60.7 | 1560 | 3170 | 0.34 |
| 62.3 | 1560 | 3210 | 0.34 |
| 64.0 | 1430 | 2540 | 0.27 |
| 65.6 | 1590 | 2950 | 0.30 |
| 67.3 | 2010 | 3510 | 0.26 |
| 68.9 | 1830 | 3470 | 0.31 |
| 70.5 | 1820 | 3220 | 0.26 |
| 72.2 | 2250 | 3920 | 0.25 |
| 73.8 | 2420 | 3900 | 0.18 |
| 75.5 | 2210 | 3620 | 0.20 |
| 77.1 | 1970 | 3700 | 0.30 |
| 78.7 | 2080 | 3790 | 0.28 |
| 80.4 | 2060 | 3810 | 0.29 |
| 82.0 | 1860 | 3400 | 0.29 |
| 83.7 | 1870 | 3530 | 0.31 |
| 85.3 | 1960 | 3530 | 0.28 |
| 86.9 | 1830 | 3140 | 0.24 |
| 88.6 | 1700 | 3210 | 0.30 |
| 90.2 | 1720 | 3000 | 0.26 |
| 91.9 | 1730 | 3400 | 0.33 |
| 93.5 | 1710 | 3450 | 0.34 |
| 95.1 | 1670 | 3320 | 0.33 |
| 96.8 | 1650 | 3330 | 0.34 |
| 98.8 | 1620 | 3170 | 0.32 |
| 100.1 | 1630 | 3090 | 0.31 |
| 101.7 | 1790 | 3240 | 0.28 |
| 103.4 | 1850 | 3470 | 0.30 |
| 105.0 | 1700 | 3320 | 0.32 |
| 106.6 | 1810 | 3620 | 0.33 |
| 108.3 | 1820 | 3790 | 0.35 |
| 109.9 | 1860 | 3790 | 0.34 |
| 111.6 | 1860 | 3620 | 0.32 |
| 113.2 | 1630 | 3170 | 0.32 |
| 114.8 | 1560 | 3580 | 0.38 |
| 116.5 | 1660 | 3620 | 0.37 |
| 118.4 | 1630 | 3700 | 0.38 |
| 119.8 | 1700 | 3510 | 0.35 |
| 121.4 | 2030 | 3920 | 0.32 |

| Metric Units | | | |
|---|---|---|---|
| Depth at Midpoint Between Receivers | Velocity | | Poisson's Ratio |
| | $V_s$ | $V_p$ | |
| (m) | (m/s) | (m/s) | |
| 18.5 | 480 | 970 | 0.34 |
| 19.0 | 480 | 980 | 0.34 |
| 19.5 | 440 | 780 | 0.27 |
| 20.0 | 480 | 900 | 0.30 |
| 20.5 | 610 | 1070 | 0.26 |
| 21.0 | 560 | 1060 | 0.31 |
| 21.5 | 560 | 980 | 0.26 |
| 22.0 | 690 | 1200 | 0.25 |
| 22.5 | 740 | 1190 | 0.18 |
| 23.0 | 680 | 1100 | 0.20 |
| 23.5 | 600 | 1130 | 0.30 |
| 24.0 | 640 | 1150 | 0.28 |
| 24.5 | 630 | 1160 | 0.29 |
| 25.0 | 570 | 1040 | 0.29 |
| 25.5 | 570 | 1080 | 0.31 |
| 26.0 | 600 | 1080 | 0.28 |
| 26.5 | 560 | 960 | 0.24 |
| 27.0 | 520 | 980 | 0.30 |
| 27.5 | 520 | 920 | 0.26 |
| 28.0 | 530 | 1040 | 0.33 |
| 28.5 | 520 | 1050 | 0.34 |
| 29.0 | 510 | 1010 | 0.33 |
| 29.5 | 500 | 1020 | 0.34 |
| 30.1 | 490 | 970 | 0.32 |
| 30.5 | 500 | 940 | 0.31 |
| 31.0 | 540 | 990 | 0.28 |
| 31.5 | 560 | 1060 | 0.30 |
| 32.0 | 520 | 1010 | 0.32 |
| 32.5 | 550 | 1100 | 0.33 |
| 33.0 | 550 | 1150 | 0.35 |
| 33.5 | 570 | 1150 | 0.34 |
| 34.0 | 570 | 1100 | 0.32 |
| 34.5 | 500 | 970 | 0.32 |
| 35.0 | 480 | 1090 | 0.38 |
| 35.5 | 510 | 1100 | 0.37 |
| 36.1 | 500 | 1130 | 0.38 |
| 36.5 | 520 | 1070 | 0.35 |
| 37.0 | 620 | 1200 | 0.32 |

## Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio Based on Receiver-to-Receiver Travel Time Data - Borehole B-8

| American Units | | | |
|---|---|---|---|
| Depth at Midpoint Between Receivers | Velocity | | Poisson's Ratio |
| | $V_s$ | $V_p$ | |
| (ft) | (ft/s) | (ft/s) | |
| 123.0 | 2140 | 3750 | 0.26 |
| 124.7 | 2080 | 3970 | 0.31 |
| 126.3 | 2160 | 4070 | 0.30 |
| 128.0 | 2230 | 4170 | 0.30 |
| 129.6 | 2190 | 3940 | 0.28 |
| 131.2 | 2240 | 4170 | 0.30 |
| 132.9 | 2000 | 3880 | 0.32 |
| 134.5 | 1860 | 3510 | 0.31 |
| 136.2 | 1980 | 4170 | 0.35 |
| 137.8 | 2010 | 4360 | 0.36 |
| 139.4 | 1750 | 3920 | 0.38 |
| 141.1 | 1690 | 4220 | 0.40 |
| 142.7 | 1770 | 5330 | 0.44 |
| 144.4 | 1710 | 6540 | 0.46 |
| 146.3 | 1630 | 6410 | 0.47 |
| 147.6 | 1640 | 5950 | 0.46 |
| 149.3 | 1610 | 5850 | 0.46 |
| 150.9 | 1450 | 6410 | 0.47 |
| 152.6 | 1580 | 6870 | 0.47 |
| 154.2 | 1790 | 6670 | 0.46 |
| 155.8 | 1890 | 6940 | 0.46 |
| 157.5 | 2020 | 7250 | 0.46 |
| 159.1 | 1930 | 6800 | 0.46 |
| 160.8 | 1830 | 6940 | 0.46 |
| 162.4 | 1800 | 6730 | 0.46 |
| 164.0 | 1780 | 6670 | 0.46 |
| 165.7 | 1820 | 6540 | 0.46 |
| 167.3 | 1930 | 6540 | 0.45 |
| 169.0 | 1630 | 6800 | 0.47 |
| 170.6 | 1670 | 6540 | 0.47 |
| 172.2 | 2140 | 6290 | 0.43 |
| 173.9 | 1950 | 6290 | 0.45 |
| 175.5 | 1990 | 7250 | 0.46 |
| 177.2 | 2260 | 8330 | 0.46 |
| 178.8 | 2270 | 7660 | 0.45 |
| 180.5 | 2090 | 7750 | 0.46 |
| 182.1 | 2130 | 7090 | 0.45 |
| 183.7 | 1810 | 6940 | 0.46 |

| Metric Units | | | |
|---|---|---|---|
| Depth at Midpoint Between Receivers | Velocity | | Poisson's Ratio |
| | $V_s$ | $V_p$ | |
| (m) | (m/s) | (m/s) | |
| 37.5 | 650 | 1140 | 0.26 |
| 38.0 | 630 | 1210 | 0.31 |
| 38.5 | 660 | 1240 | 0.30 |
| 39.0 | 680 | 1270 | 0.30 |
| 39.5 | 670 | 1200 | 0.28 |
| 40.0 | 680 | 1270 | 0.30 |
| 40.5 | 610 | 1180 | 0.32 |
| 41.0 | 570 | 1070 | 0.31 |
| 41.5 | 600 | 1270 | 0.35 |
| 42.0 | 610 | 1330 | 0.36 |
| 42.5 | 530 | 1200 | 0.38 |
| 43.0 | 510 | 1290 | 0.40 |
| 43.5 | 540 | 1630 | 0.44 |
| 44.0 | 520 | 1990 | 0.46 |
| 44.6 | 500 | 1950 | 0.47 |
| 45.0 | 500 | 1810 | 0.46 |
| 45.5 | 490 | 1780 | 0.46 |
| 46.0 | 440 | 1950 | 0.47 |
| 46.5 | 480 | 2090 | 0.47 |
| 47.0 | 550 | 2030 | 0.46 |
| 47.5 | 580 | 2120 | 0.46 |
| 48.0 | 620 | 2210 | 0.46 |
| 48.5 | 590 | 2070 | 0.46 |
| 49.0 | 560 | 2120 | 0.46 |
| 49.5 | 550 | 2050 | 0.46 |
| 50.0 | 540 | 2030 | 0.46 |
| 50.5 | 550 | 1990 | 0.46 |
| 51.0 | 590 | 1990 | 0.45 |
| 51.5 | 500 | 2070 | 0.47 |
| 52.0 | 510 | 1990 | 0.47 |
| 52.5 | 650 | 1920 | 0.43 |
| 53.0 | 590 | 1920 | 0.45 |
| 53.5 | 610 | 2210 | 0.46 |
| 54.0 | 690 | 2540 | 0.46 |
| 54.5 | 690 | 2340 | 0.45 |
| 55.0 | 640 | 2360 | 0.46 |
| 55.5 | 650 | 2160 | 0.45 |
| 56.0 | 550 | 2120 | 0.46 |

## Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio Based on Receiver-to-Receiver Travel Time Data - Borehole B-8

| American Units | | | |
|---|---|---|---|
| **Depth at Midpoint Between Receivers** | **Velocity** | | **Poisson's Ratio** |
| | **$V_s$** | **$V_p$** | |
| (ft) | (ft/s) | (ft/s) | |
| 185.4 | 1560 | 6170 | 0.47 |
| 187.0 | 1520 | 6120 | 0.47 |

| Metric Units | | | |
|---|---|---|---|
| **Depth at Midpoint Between Receivers** | **Velocity** | | **Poisson's Ratio** |
| | **$V_s$** | **$V_p$** | |
| (m) | (m/s) | (m/s) | |
| 56.5 | 480 | 1880 | 0.47 |
| 57.0 | 460 | 1860 | 0.47 |

**Notes:** "-" means no data available at that particular interval of depth.

# APPENDIX A

# SUSPENSION VELOCITY MEASUREMENT QUALITY ASSURANCE SUSPENSION SOURCE TO RECEIVER ANALYSIS RESULTS

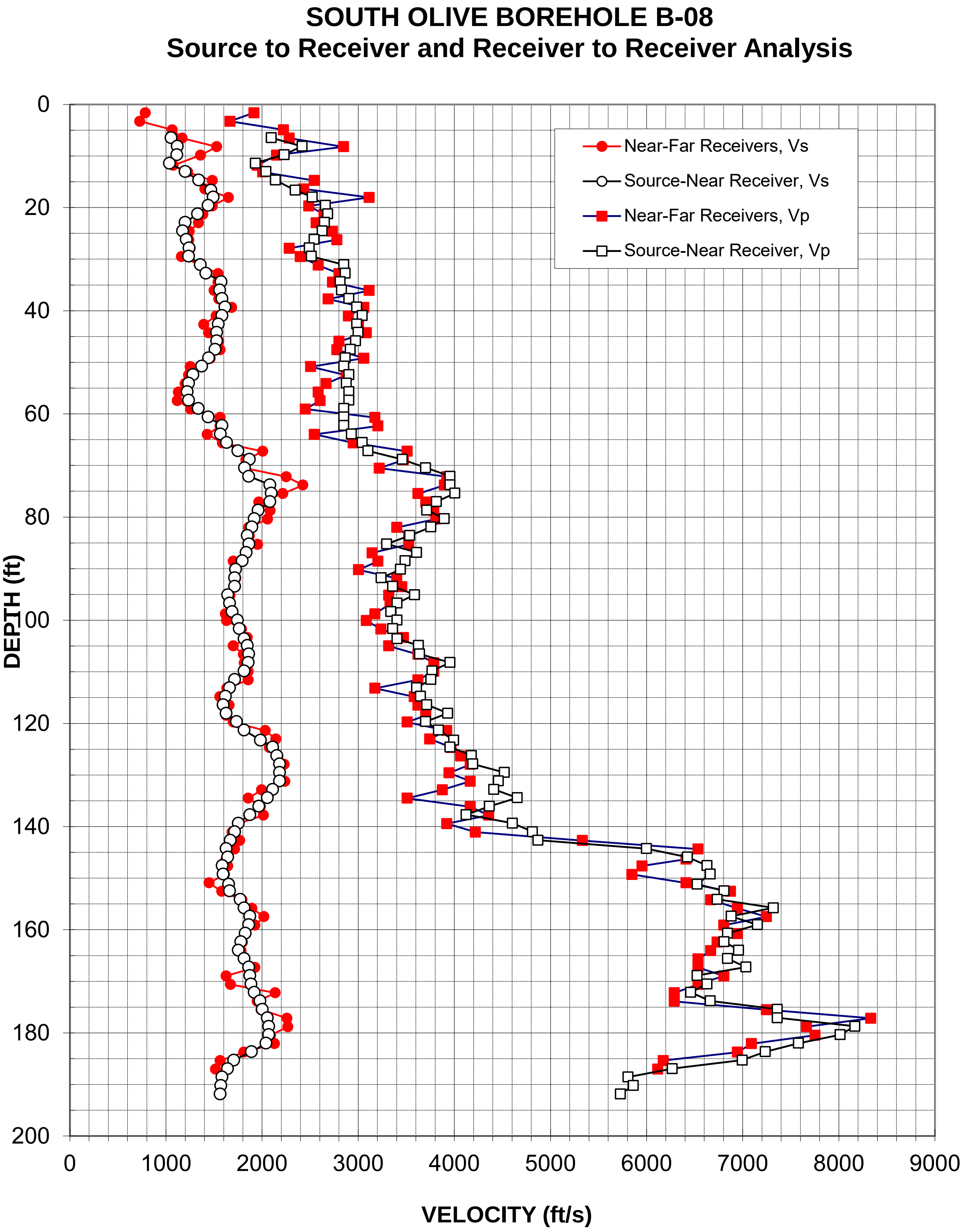

Figure A-1: Borehole B-8, Suspension S-R1 P- and $S_H$-wave velocities

# Table A-1. Borehole B-8, 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-8**

| American Units | | | |
|---|---|---|---|
| Depth at Midpoint Between Source and Near Receiver | Velocity | | Poisson's Ratio |
| | $V_s$ | $V_p$ | |
| (ft) | (ft/s) | (ft/s) | |
| 6.5 | 1050 | 2100 | 0.33 |
| 8.1 | 1120 | 2420 | 0.36 |
| 9.8 | 1110 | 2230 | 0.33 |
| 11.4 | 1040 | 1930 | 0.30 |
| 13.0 | 1200 | 2040 | 0.24 |
| 14.7 | 1340 | 2140 | 0.18 |
| 16.6 | 1470 | 2340 | 0.18 |
| 18.0 | 1490 | 2520 | 0.23 |
| 19.6 | 1440 | 2660 | 0.29 |
| 21.2 | 1330 | 2680 | 0.34 |
| 22.9 | 1200 | 2650 | 0.37 |
| 24.5 | 1170 | 2630 | 0.38 |
| 26.2 | 1210 | 2540 | 0.35 |
| 27.8 | 1240 | 2490 | 0.34 |
| 29.4 | 1240 | 2510 | 0.34 |
| 31.1 | 1360 | 2850 | 0.35 |
| 32.7 | 1420 | 2860 | 0.34 |
| 34.4 | 1570 | 2810 | 0.27 |
| 36.0 | 1560 | 2830 | 0.28 |
| 37.6 | 1580 | 2900 | 0.29 |
| 39.3 | 1610 | 2990 | 0.29 |
| 40.9 | 1580 | 3040 | 0.31 |
| 42.6 | 1540 | 2990 | 0.32 |
| 44.2 | 1530 | 3000 | 0.32 |
| 45.8 | 1530 | 2970 | 0.32 |
| 47.5 | 1510 | 2920 | 0.32 |
| 49.1 | 1450 | 2860 | 0.33 |
| 50.8 | 1370 | 2850 | 0.35 |
| 52.4 | 1280 | 2900 | 0.38 |
| 54.0 | 1240 | 2880 | 0.39 |
| 55.7 | 1220 | 2900 | 0.39 |
| 57.3 | 1240 | 2900 | 0.39 |
| 59.0 | 1340 | 2850 | 0.36 |
| 60.6 | 1440 | 2850 | 0.33 |
| 62.2 | 1580 | 2850 | 0.28 |
| 63.9 | 1570 | 2930 | 0.30 |

| Metric Units | | | |
|---|---|---|---|
| Depth at Midpoint Between Source and Near Receiver | Velocity | | Poisson's Ratio |
| | $V_s$ | $V_p$ | |
| (m) | (m/s) | (m/s) | |
| 2.0 | 320 | 640 | 0.33 |
| 2.5 | 340 | 740 | 0.36 |
| 3.0 | 340 | 680 | 0.33 |
| 3.5 | 320 | 590 | 0.30 |
| 4.0 | 370 | 620 | 0.24 |
| 4.5 | 410 | 650 | 0.18 |
| 5.1 | 450 | 710 | 0.18 |
| 5.5 | 460 | 770 | 0.23 |
| 6.0 | 440 | 810 | 0.29 |
| 6.5 | 410 | 820 | 0.34 |
| 7.0 | 370 | 810 | 0.37 |
| 7.5 | 360 | 800 | 0.38 |
| 8.0 | 370 | 770 | 0.35 |
| 8.5 | 380 | 760 | 0.34 |
| 9.0 | 380 | 770 | 0.34 |
| 9.5 | 410 | 870 | 0.35 |
| 10.0 | 430 | 870 | 0.34 |
| 10.5 | 480 | 860 | 0.27 |
| 11.0 | 480 | 860 | 0.28 |
| 11.5 | 480 | 890 | 0.29 |
| 12.0 | 490 | 910 | 0.29 |
| 12.5 | 480 | 930 | 0.31 |
| 13.0 | 470 | 910 | 0.32 |
| 13.5 | 470 | 910 | 0.32 |
| 14.0 | 470 | 910 | 0.32 |
| 14.5 | 460 | 890 | 0.32 |
| 15.0 | 440 | 870 | 0.33 |
| 15.5 | 420 | 870 | 0.35 |
| 16.0 | 390 | 890 | 0.38 |
| 16.5 | 380 | 880 | 0.39 |
| 17.0 | 370 | 890 | 0.39 |
| 17.5 | 380 | 890 | 0.39 |
| 18.0 | 410 | 870 | 0.36 |
| 18.5 | 440 | 870 | 0.33 |
| 19.0 | 480 | 870 | 0.28 |
| 19.5 | 480 | 890 | 0.30 |

## Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio Based on Source-to-Receiver Travel Time Data - Borehole B-8

| American Units | | | |
|---|---|---|---|
| **Depth at Midpoint** | **Velocity** | | **Poisson's Ratio** |
| **Between Source and Near Receiver** | **$V_s$** | **$V_p$** | |
| (ft) | (ft/s) | (ft/s) | |
| 65.5 | 1630 | 3040 | 0.30 |
| 67.2 | 1750 | 3100 | 0.27 |
| 68.8 | 1870 | 3460 | 0.29 |
| 70.5 | 1820 | 3700 | 0.34 |
| 72.1 | 1860 | 3960 | 0.36 |
| 73.7 | 2080 | 3960 | 0.31 |
| 75.4 | 2100 | 4010 | 0.31 |
| 77.0 | 2080 | 3810 | 0.29 |
| 78.7 | 1960 | 3710 | 0.31 |
| 80.3 | 1920 | 3900 | 0.34 |
| 81.9 | 1900 | 3760 | 0.33 |
| 83.6 | 1850 | 3540 | 0.31 |
| 85.2 | 1860 | 3300 | 0.26 |
| 86.9 | 1830 | 3610 | 0.33 |
| 88.5 | 1790 | 3490 | 0.32 |
| 90.1 | 1720 | 3440 | 0.33 |
| 91.8 | 1720 | 3240 | 0.30 |
| 93.4 | 1720 | 3360 | 0.32 |
| 95.1 | 1640 | 3590 | 0.37 |
| 96.7 | 1660 | 3400 | 0.34 |
| 98.3 | 1690 | 3340 | 0.33 |
| 100.0 | 1740 | 3400 | 0.32 |
| 101.6 | 1760 | 3360 | 0.31 |
| 103.6 | 1810 | 3400 | 0.30 |
| 104.9 | 1850 | 3630 | 0.33 |
| 106.5 | 1860 | 3640 | 0.32 |
| 108.2 | 1860 | 3960 | 0.36 |
| 109.8 | 1810 | 3770 | 0.35 |
| 111.5 | 1720 | 3760 | 0.37 |
| 113.1 | 1660 | 3610 | 0.37 |
| 114.7 | 1620 | 3650 | 0.38 |
| 116.4 | 1590 | 3710 | 0.39 |
| 118.0 | 1630 | 3930 | 0.40 |
| 119.7 | 1730 | 3700 | 0.36 |
| 121.3 | 1810 | 3840 | 0.36 |
| 123.3 | 1980 | 3990 | 0.34 |
| 124.6 | 2110 | 3960 | 0.30 |
| 126.2 | 2150 | 4180 | 0.32 |
| 127.9 | 2180 | 4190 | 0.31 |

| Metric Units | | | |
|---|---|---|---|
| **Depth at Midpoint** | **Velocity** | | **Poisson's Ratio** |
| **Between Source and Near Receiver** | **$V_s$** | **$V_p$** | |
| (m) | (m/s) | (m/s) | |
| 20.0 | 500 | 930 | 0.30 |
| 20.5 | 530 | 950 | 0.27 |
| 21.0 | 570 | 1050 | 0.29 |
| 21.5 | 550 | 1130 | 0.34 |
| 22.0 | 570 | 1210 | 0.36 |
| 22.5 | 630 | 1210 | 0.31 |
| 23.0 | 640 | 1220 | 0.31 |
| 23.5 | 630 | 1160 | 0.29 |
| 24.0 | 600 | 1130 | 0.31 |
| 24.5 | 580 | 1190 | 0.34 |
| 25.0 | 580 | 1150 | 0.33 |
| 25.5 | 560 | 1080 | 0.31 |
| 26.0 | 570 | 1000 | 0.26 |
| 26.5 | 560 | 1100 | 0.33 |
| 27.0 | 550 | 1060 | 0.32 |
| 27.5 | 530 | 1050 | 0.33 |
| 28.0 | 520 | 990 | 0.30 |
| 28.5 | 520 | 1020 | 0.32 |
| 29.0 | 500 | 1090 | 0.37 |
| 29.5 | 510 | 1040 | 0.34 |
| 30.0 | 510 | 1020 | 0.33 |
| 30.5 | 530 | 1040 | 0.32 |
| 31.0 | 540 | 1020 | 0.31 |
| 31.6 | 550 | 1040 | 0.30 |
| 32.0 | 560 | 1110 | 0.33 |
| 32.5 | 570 | 1110 | 0.32 |
| 33.0 | 570 | 1210 | 0.36 |
| 33.5 | 550 | 1150 | 0.35 |
| 34.0 | 520 | 1150 | 0.37 |
| 34.5 | 510 | 1100 | 0.37 |
| 35.0 | 490 | 1110 | 0.38 |
| 35.5 | 490 | 1130 | 0.39 |
| 36.0 | 500 | 1200 | 0.40 |
| 36.5 | 530 | 1130 | 0.36 |
| 37.0 | 550 | 1170 | 0.36 |
| 37.6 | 600 | 1220 | 0.34 |
| 38.0 | 640 | 1210 | 0.30 |
| 38.5 | 660 | 1270 | 0.32 |
| 39.0 | 670 | 1280 | 0.31 |

## Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio Based on Source-to-Receiver Travel Time Data - Borehole B-8

| American Units | | | |
|---|---|---|---|
| Depth at Midpoint Between Source and Near Receiver | Velocity | | Poisson's Ratio |
| | $V_s$ | $V_p$ | |
| (ft) | (ft/s) | (ft/s) | |
| 129.5 | 2180 | 4520 | 0.35 |
| 131.1 | 2180 | 4460 | 0.34 |
| 132.8 | 2110 | 4410 | 0.35 |
| 134.4 | 2060 | 4650 | 0.38 |
| 136.1 | 1970 | 4370 | 0.37 |
| 137.7 | 1870 | 4120 | 0.37 |
| 139.3 | 1750 | 4600 | 0.42 |
| 141.0 | 1720 | 4810 | 0.43 |
| 142.6 | 1670 | 4870 | 0.43 |
| 144.3 | 1630 | 6000 | 0.46 |
| 145.9 | 1640 | 6430 | 0.46 |
| 147.6 | 1590 | 6630 | 0.47 |
| 149.2 | 1590 | 6660 | 0.47 |
| 151.2 | 1650 | 6530 | 0.47 |
| 152.5 | 1660 | 6810 | 0.47 |
| 154.1 | 1770 | 6730 | 0.46 |
| 155.8 | 1810 | 7320 | 0.47 |
| 157.4 | 1870 | 6880 | 0.46 |
| 159.0 | 1860 | 7150 | 0.46 |
| 160.7 | 1820 | 6840 | 0.46 |
| 162.3 | 1780 | 6810 | 0.46 |
| 164.0 | 1750 | 6960 | 0.47 |
| 165.6 | 1810 | 6840 | 0.46 |
| 167.2 | 1860 | 7030 | 0.46 |
| 168.9 | 1870 | 6530 | 0.46 |
| 170.5 | 1880 | 6630 | 0.46 |
| 172.2 | 1920 | 6460 | 0.45 |
| 173.8 | 1980 | 6660 | 0.45 |
| 175.4 | 2000 | 7360 | 0.46 |
| 177.1 | 2060 | 7360 | 0.46 |
| 178.7 | 2070 | 8170 | 0.47 |
| 180.4 | 2070 | 8010 | 0.46 |
| 182.0 | 2040 | 7580 | 0.46 |
| 183.6 | 1890 | 7230 | 0.46 |
| 185.3 | 1710 | 6990 | 0.47 |
| 186.9 | 1640 | 6270 | 0.46 |
| 188.6 | 1580 | 5810 | 0.46 |
| 190.2 | 1570 | 5860 | 0.46 |
| 191.8 | 1560 | 5730 | 0.46 |

| Metric Units | | | |
|---|---|---|---|
| Depth at Midpoint Between Source and Near Receiver | Velocity | | Poisson's Ratio |
| | $V_s$ | $V_p$ | |
| (m) | (m/s) | (m/s) | |
| 39.5 | 670 | 1380 | 0.35 |
| 40.0 | 670 | 1360 | 0.34 |
| 40.5 | 640 | 1340 | 0.35 |
| 41.0 | 630 | 1420 | 0.38 |
| 41.5 | 600 | 1330 | 0.37 |
| 42.0 | 570 | 1260 | 0.37 |
| 42.5 | 530 | 1400 | 0.42 |
| 43.0 | 520 | 1470 | 0.43 |
| 43.5 | 510 | 1480 | 0.43 |
| 44.0 | 500 | 1830 | 0.46 |
| 44.5 | 500 | 1960 | 0.46 |
| 45.0 | 480 | 2020 | 0.47 |
| 45.5 | 490 | 2030 | 0.47 |
| 46.1 | 500 | 1990 | 0.47 |
| 46.5 | 510 | 2070 | 0.47 |
| 47.0 | 540 | 2050 | 0.46 |
| 47.5 | 550 | 2230 | 0.47 |
| 48.0 | 570 | 2100 | 0.46 |
| 48.5 | 570 | 2180 | 0.46 |
| 49.0 | 560 | 2090 | 0.46 |
| 49.5 | 540 | 2070 | 0.46 |
| 50.0 | 530 | 2120 | 0.47 |
| 50.5 | 550 | 2090 | 0.46 |
| 51.0 | 570 | 2140 | 0.46 |
| 51.5 | 570 | 1990 | 0.46 |
| 52.0 | 570 | 2020 | 0.46 |
| 52.5 | 580 | 1970 | 0.45 |
| 53.0 | 600 | 2030 | 0.45 |
| 53.5 | 610 | 2240 | 0.46 |
| 54.0 | 630 | 2240 | 0.46 |
| 54.5 | 630 | 2490 | 0.47 |
| 55.0 | 630 | 2440 | 0.46 |
| 55.5 | 620 | 2310 | 0.46 |
| 56.0 | 580 | 2210 | 0.46 |
| 56.5 | 520 | 2130 | 0.47 |
| 57.0 | 500 | 1910 | 0.46 |
| 57.5 | 480 | 1770 | 0.46 |
| 58.0 | 480 | 1790 | 0.46 |
| 58.5 | 480 | 1750 | 0.46 |

# APPENDIX B

# BOREHOLE GEOPHYSICAL LOGGING SYSTEMS - NIST TRACEABLE CALIBRATION RECORDS

MICRO PRECISION CALIBRATION

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

ilac-MRA ANAB ACCREDITED ISO/IEC 17025 CALIBRATION LABORATORY
AC-1969.03

# Certificate of Calibration

**Date: Oct 9, 2017**

**Cert No. 512200813056866**

**Customer:**

GEOVISION

1124 OLYMPIC DRIVE
CORONA CA 92881

Work Order #: LA-90029989
Purchase Order #: 17341-170929-01

MPC Control #: AM6767
Asset ID: 160023
Gage Type: LOGGER
Manufacturer: OYO
Model Number: 3403
Size: N/A
Temp/RH: 68.8°F / 40.5%
Location: Calibration performed at MPC facility

Serial Number: 160023
Department: N/A
Performed By: NIKOLAS GROHMAN
Received Condition: IN TOLERANCE
Returned Condition: IN TOLERANCE
Cal. Date: September 29, 2017
Cal. Interval: 12 MONTHS
Cal. Due Date: September 29, 2018

**Calibration Notes:**

This Certificate Supersedes Cert No. 512200813056466

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: 5KKSA84231
Calibrated To 4:1 Accuracy Ratio

Calibration performed in accordance with approved GEOVision calibration procedures included in work Instruction No. 17.

Software: ML PS 4.00 Suspension Logger, GVLog.jar ( 2004 ) and pslog.exe ver 1.00 software.

## Standards Used to Calibrate Equipment

| I.D. | Description. | Model | Serial | Manufacturer | Cal. Due Date | Traceability # |
|---|---|---|---|---|---|---|
| DB8748 | GPS TIME AND FREQUENCY RECEIVER | 58503A | 3625A01225 | HEWLETT PACKARD | Jun 16, 2019 | 512200812919221 |
| LAS0018 | ARB / FUNC GENERATOR | 33250A | US40001522 | AGILENT | Dec 7, 2017 | 512200812632023 |
| BD7715 | UNIVERSAL COUNTER | 53131A | 3416A05377 | HEWLETT PACKARD | Oct 3, 2017 | 512200812523201 |

Calibrating Technician:

NIKOLAS GROHMAN

QC Approval:

Tyler McKeen

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/IEC 17025:2005, ANSI/NCSL Z540-1-1994, ANSI/NCSL Z540.3-2006, 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
2165 N. Glassell St.,
Orange, CA 92865
714-901-5659

AC-1969.03

# Certificate of Calibration

**Date: Oct 9, 2017** **Cert No. 512200813056866**

## Procedures Used in this Event

| Procedure Name | Description |
|---|---|
| GEOVISION SEISMIC | Seismic Logger/Recorder Calibration Procedure, Rev. 2.1 |

Calibrating Technician: NIKOLAS GROHMAN

QC Approval: Tyler McKeen

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/IEC 17025:2005, ANSI/NCSL Z540-1-1994, ANSI/NCSL Z540.3-2006, 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.

# REPORT OF GEOTECHNICAL ENGINEERING SERVICES

for

# PROPOSED HIGH-RISE RESIDENTIAL TOWER DEVELOPMENT – SITE 3

## 1100 South Olive Street
## Los Angeles, California

*Prepared For:*

**DTLA South Park Properties Propco I, LLC**
**1150 S. OLIVE ST. SUITE 2250**
**LOS ANGELES CA 90015**

*Prepared By:*

**Langan Engineering and Environmental Services, Inc.**
**18575 Jamboree Road, Suite 150**
**Irvine, California 92612**

**December 16, 2020**
**Langan Project No.: 700070701**

LANGAN

Technical Excellence
Practical Experience
Client Responsiveness

December 16, 2020

Andrew Dutton and Kevin Lindquist
DTLA South Park Properties Propco I, LLC
1150 South Olive Street, Suite 2250
Los Angeles CA 90015

**Preliminary Geotechnical Report**
**Proposed High-Rise Residential Tower Development – Site 3**
**1100 South Olive Street**
**Los Angeles, California**
**Langan Project: 700070701**

Dear Mr. Dutton and Lindquist:

Langan Engineering & Environmental Services, Inc. is pleased to submit this geotechnical investigation report for the proposed high-rise residential tower (aka Site 3) to be constructed at 1100 South Olive Street in Los Angeles, California.

Our services were performed in general accordance with our proposal dated November 13, 2020 that you authorized on December 7, 2020.

◆ ◆ ◆

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

Sincerely,
Langan Engineering & Environmental Services, Inc.

REGISTERED PROFESSIONAL ENGINEER
CHRISTOPHER JOHN ZADOORIAN
GE2493
signed 12/16/20
GEOTECHNICAL
STATE OF CALIFORNIA

Chris Zadoorian, G.E.
Associate

AJA:SR:SHW:CJZ:
Document ID: \\langan.com\data\IRV\data7\700070701\Project Data_Discipline\Geotechnical\Reports\700070701 -11202020-geor-site 3 - aja.docx

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## FIGURES

## APPENDICES

# 1.0 INTRODUCTION

## 1.1 GENERAL

This preliminary report was prepared for the proposed high-rise residential tower development to be constructed at 1100 South Olive Street (DTLA Site 3) in Los Angeles, California. The site location is shown on Figure 1.

## 1.2 EXISTING SITE CONDITIONS

### 1.2.1 General

The approximately 1.25-arce site is located at the southern corner of the intersection between South Olive Street and West 11th Street in downtown Los Angeles as shown on Figure 2.

The site is currently an asphaltic concrete (AC)-paved surface parking lot and the ground surface level slopes gently to the south and ranges from approximately Elevation 246 feet at the northwest corner to approximately 244 at the southwest corner.

Existing structures are located along the southeast, southwest, and northwest sides of the site as shown on Figure 2. These structures include existing one- and two-story masonry buildings located at 1127 and 1111 South Hill Street; a 32-story, high-rise tower located at 1155 South Hill Street (AT&T Center Tower); and an existing six-level parking garage located at 1127 – 1143 South Olive Street. In addition, another high-rise tower development is planned at 1105 South Olive Street. South Olive Street borders the property on the northwest while West 11th Street borders the property on the northeast. Each adjacent structure is described briefly herein.

As discussed in Section 1.6 below, a prior report of geotechnical investigation dated June 20, 2018 was prepared by GeoDesign, Inc. (GDI) for the proposed development and the prior GDI report presents information regarding the adjacent developments as discussed in the following sections.

### 1.2.2 Two-Story Masonry Building (1111 South Hill Street)

As-built plans were not available for the existing two-story building that is located immediately southeast of the site; however, based on observations and research performed by GDI, the existing building is two stories above grade, of masonry construction, and was constructed circa 1955. It is not clear if the existing building includes subterranean parking levels; therefore, we have assumed that the lowest finished floor level of this building is established at approximately the existing ground surface level, Elevation 246.

A copy of the original report of geotechnical investigation dated April 8, 1954 prepared by L. T. Evans for the existing building was referenced in by GDI. GDI indicated that the existing building is supported on spread and continuous footings using an allowable bearing pressure of 8,000 psf.

### 1.2.3 One-Story Masonry Bank of America Building (1127 South Hill Street)

Plans were not available for the other existing building that is located immediately southeast of the site; however, based on our visual observations, the existing building is one story above grade and of wood-frame and masonry block construction. The existing building has a relatively small footprint and was established at approximately the existing ground surface, Elevation 245.

A copy of a report of compacted fill dated March 25, 1975 prepared by Converse, Davis and Associates, that summarized grading activities performed at the site was referenced by GDI. Based on GDI's review of the compaction report, the existing building is supported on spread and continuous footings established in properly compacted fill and designed using an allowable bearing pressure of 2,500 psf.

### 1.2.4 32-Story AT&T Center Tower (1155 South Hill Street)

Plans were not available for the existing 32-story AT&T Center Tower that is located southwest of the site; however, based on our visual observations and research, the existing reinforced concrete building is 32 stories above grade and was completed in 1965.

The existing building includes at least two subterranean levels; therefore, we have assumed that the lowest finished floor level of this building is established approximately 25 feet BGS, corresponding to approximately Elevation 220.

### 1.2.5 Six-Level Parking Garage (1127 - 1143 South Olive Street)

GDI referenced as-built plans dated December 11, 1964 prepared by William L. Pereira & Associates, for the existing six-level parking garage located northwest of the site. GDI indicated that the existing parking garage is supported on a combined foundation system that consists of CIDH shafts and deepened spread footings.

The drilled shafts were installed along the northeast to southwest building grid lines (Grid Lines A and J) and deepened spread footings were used for the remainder of the parking garage. Based on GDIs review of the as-built plans, drilled shafts along the north building grid line (Grid Line J) consist of 24-inch-diameter CIDH shafts that are 30 and 37 feet in length below the bottom of the pile cap.

The lowest finish floor level of the existing parking garage is established at approximately Elevation 248.4.

### 1.2.6 Proposed 51-Story Tower (1105 South Olive Street)

We prepared a preliminary geotechnical report dated December 15, 2020 for a proposed 51-story tower to be constructed at 1105 South Olive Street, northwest of the site. The proposed tower will be approximately 603-foot-tall over six subterranean parking levels. The lowest finished floor level for the subterranean parking will be approximately 60 feet BGS, corresponding to approximately Elevation 185.

Based on the preliminary information available, we anticipate that the proposed tower will be supported on a mat foundation on the order of 10 feet thick. We anticipate the average applied bearing pressure on the mat foundation will be on the order of 10,000 psf.

## 1.3 HISTORICAL SITE DEVELOPMENT

Sanborn maps and aerial photographs dating back to 1888 to provide general background information regarding the historical site development. Based on GDIs review of this information, initial residential development at the site occurred by 1906 and consisted of flats and houses on the property. Between 1928 and 1947 automotive shops, commercial buildings, and/or surface parking lots were present at the site.

Adjacent buildings located at 1111 South Hill Street and 1155 South Hill Street are shown on the aerial photograph dated 1965. By 1965 the automotive shop and commercial buildings were removed and replaced by the current surface parking lot.

The site has remained relatively unchanged since 1965.

## 1.4 PROPOSED DEVELOPMENT

You furnished us with conceptual plans dated May 4, 2018, prepared by CallisonRTKL for the proposed development.

Based on our review of the conceptual plans and the draft structural engineering basis of design, the proposed development will include construction of a 60-story, 698-foot-tall, mixed-use tower and a contiguous 5-level podium structure, each constructed over six subterranean parking levels. The lowest finished floor level for the subterranean parking garage will be approximately 60 feet BGS, corresponding to approximately Elevation 186.

Preliminary foundation plans and structural loading were unavailable at the time of this report. However, based on similar projects, we estimate the average dead-plus-live foundation loading for the tower may be on the order of 12,000 to 15,000 psf and typical dead-plus-live column loading for podium structure may be on the order of 1,000 to 1,500 kips.

We also anticipate that the tower will be supported on a mat foundation that may be on the order of 10 feet thick, so the bottom of the mat foundation will be established at approximately Elevation 176.

Temporary excavations up to approximately 70 feet deep will be required for the proposed development and as a result temporary shoring will be required. The temporary shoring design and construction will include provisions for support of adjacent structures.

The site is also located in a LADBS-designated methane zone. As such, methane mitigation provisions will be required in accordance with LADBS Ordinance No. 175790. The required level of methane mitigation will be based on the results of a soil gas survey.

On-site groundwater infiltration will be implemented as part of Standard Urban Storm Water Mitigation Plan mitigation measures. We anticipate that stormwater infiltration will be accomplished through the use of deep drywells, likely within the building footprint.

## 1.5 SEISMIC DESIGN APPROACH

It's our understanding that the proposed residential tower will be designed using a non-prescriptive seismic design approach as permitted in Section 104.11 Alternative materials, design and methods of construction and equipment of the 2019 California Building Code (CBC). This approach allows the use of alternative methods to determine the design lateral forces on the tower in lieu of code-prescriptive CBC provisions. LADBS approved the CBC provision in LADBS Information Bulletin P/BC 2017-123 titled Alternative Design Procedure for Seismic Analysis and Design of Tall Buildings and Buildings Utilizing Complex Structural Systems.

In general, the alternative design approach will result in a building that remains serviceable when subjected to frequent earthquakes and a building that does not experience collapse during an extremely rare seismic event.

Seismic design for the parking garage will be performed in general accordance with LABC-prescriptive methods and/or utilizing site specific ground motions in accordance with ASCE-7-16, Chapter 11-4.8.

Site specific ground motion studies in accordance with ASCE-7-16 will be performed during the design development phase of the project along with earthquake acceleration time histories for use in the alternative design procedure.

## 1.6 PRIOR GEOTECHNICAL INVESTIGATION

### 1.6.1 Prior On-Site Borings and Laboratory Testing

You furnished us a prior geotechnical report dated June 20, 2018 prepared by GDI for our review. GDI's explorations at the site include nine borings (B-1 through B-9) at the approximate locations shown on Figure 2.

The results of pertinent prior data is summarized herein followed by our conclusions and recommendations for the proposed development.

### 1.6.2 Statement of Responsibility for Prior Data

We have reviewed the data presented in the GDI report and nearby P-S logging and have determined that the information presented is suitable for use in developing the conclusions and preliminary geotechnical design recommendations presented herein. As such, we assume the professional responsibility for the use and interpretation of the prior GDI data.

## 2.0 SITE EXPLORATIONS AND SUBSURFACE CONDITIONS

## 2.1 SUBSURFACE CONDITIONS

GDI drilled six borings (B-1 through B-6) as part of prior work at the site in 2007. The borings were drilled to depths between 75.3 and 126.0 feet BGS using a hollow-stem auger drill rig.

To supplement the prior data and to perform field percolation testing, GDI drilled three additional borings (B-7 through B-9) at the site to depths between 75.8 and 200.9 BGS. Borings B-7 and B-9 were drilled using a hollow-stem auger drill rig and boring B-8 was drilled using a mud rotary drill rig. The locations of all explorations at the site are shown on Figure 2.

AC pavement ranging from approximately 3 to 6 inches in thickness was encountered in the borings. A 2-inch-thick layer of base materials was logged in boring B-9 beneath the AC pavement and the base is underlain by 4 inches of PCC in this boring. Boring B-4 also encountered 3 inches of PCC beneath the AC pavement.

Fill typically ranging to depths between approximately 3 and 8 feet BGS was logged in the prior borings beneath the pavement section. The fill material consists predominately of stiff to very stiff silt and sandy silt and medium dense silty sand and clayey sand. Brick fragments were logged in the fill at several locations. The fill is likely related to the prior episodes of development at the site and will be removed as part of the planned excavation.

Native soil encountered below the fill or directly below the pavement section generally consists of an upper layer of medium dense to very dense sand with gravel and cobbles typically logged to depths between approximately 22 and 38 feet BGS.

The upper soil is typically underlain by alternating layers of stiff to hard silt, sandy silt, and sandy clay and dense to very dense sand and silty sand with gravel to depths of approximately 135 feet BGS.

An approximately 13-foot-thick layer of very dense gravel with cobbles was encountered at a depth of 135 feet BGS in boring B-8.

Soft to medium hard siltstone and sandstone bedrock was encountered below the gravel layer to the completion depth of the exploration B-8 at 201 feet BGS. The bedrock is intensely to slightly weathered, with decreasing weathering with depth.

Generalized cross sections are presented on Figures 3 through 6, which depict the subsurface conditions at the site.

Logs for the prior borings B-1 through B-9 are presented in Appendix A.

## 2.2 GROUNDWATER CONDITIONS

### 2.2.1 General

The distinction between the groundwater table and groundwater seepage is significant for sites in downtown Los Angeles as intermittent and typically discontinuous, relatively shallow zones of groundwater seepage are often present. Intermittent zones of groundwater seepage would require mitigation during the construction phase of the project if encountered within the depth of the proposed excavation; however, in general, this condition would not require any permanent design considerations. Groundwater seepage, the global groundwater table, and the historical high groundwater level are each discussed below.

### 2.2.2 Groundwater Seepage

Groundwater seepage was not observed in the explorations at the site; however, it has been our experience that groundwater seepage, though present, is not always evident in hollow-stem auger and/or mud rotary borings. In addition, the frequency and intensity of groundwater seepage varies seasonally, typically in proportion to the rainfall levels. In this part of Los Angeles, perched groundwater seepage on silt and clay layers is typically sporadic, and in many cases explorations that are in close proximity to each other may encounter highly variable groundwater conditions. It is also typical for the perched water to dissipate relatively quickly once encountered.

### 2.2.3 Groundwater Table

Groundwater was logged at a depth of 130 feet BGS in GDI's boring B-8. This depth correlates well with other nearby deep borings we have drilled.

### 2.2.4 Historical High Groundwater Level

Based on our review of the Seismic Hazard Zone Report for the Hollywood 7.5-Minute Quadrangle (CGS, 1998), the historical high groundwater level is approximately 110 feet BGS.

## 2.3 FIELD PERCOLATION TESTING

GDI performed field percolation testing in general conformance with the Borehole Percolation Test Procedure outlined in the Guidelines for Design, Investigation and Reporting Low Impact Development Stormwater Infiltration (Section GS200.2 of County of Los Angeles Administrative Manual, December 2014) in borings B-7 and B-9 at depths of 75.8 and 76.5 feet BGS, respectively.

The test procedure consisted of drilling 8-inch-diameter boreholes to the corresponding test depths below the existing ground surface, placing a 2-inch-diameter perforated pipe in the holes, and backfilling the annulus with clean gravel to avoid caving in the test zone.

To perform the necessary testing, a 2-inch-diameter PVC pipe was installed within the hollow-stem auger simultaneously as the auger was being withdrawn from the hole. The lower 5 feet of the PVC pipe was screened and an end cap was installed at the bottom of the pipe. To prevent caving of the boring side wall, filter pack gravel was placed around the PVC pipe as the hollow-stem auger was withdrawn.

The testing consisted of introducing water to the subsurface soil through the PVC pipe and measuring the rate of infiltration. Prior to the start of the test, pre-soaking was performed in each boring. During the two-hour pre-soaking period, the water level in each test dropped more than 12 inches within 30 minutes or less; therefore, the pre-soaking was considered complete in accordance with the City of Los Angeles guidelines.

Field percolation testing was initiated following the completion of the pre-soak process. Water was added and the water level drop was recorded for 10-minute intervals until stabilized rates were obtained after the fourth and fifth intervals, respectively. A stabilized rate is considered to be reached when the highest and lowest readings from three consecutive readings were within 10 percent of each other.

A reduction factor was used to adjust the infiltration rate to account for the discharge of water through the gravel pack and into the sides of the borehole.

The testing indicates a design infiltration rate for the on-site soil of approximately 15 inches per hour at a depth of approximately 75 feet BGS. The results of percolation testing are presented in Appendix B.

After the completion of the percolation test, the PVC pipe was removed from the boring and the boring was backfilled with the soil cuttings. Excess soil cuttings resulting from the percolation testing were placed in 55-gallon drums and subsequently hauled from the site by a licensed materials hauler.

### 2.4 P-S SUSPENSION SEISMIC VELOCITY LOGGING

P-S suspension logging was performed in GDI's boring B-8. The testing was conducted by GEOVision, Inc. of Corona, California.

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 performed using P-S suspension logging techniques, the average shear wave velocity for the upper 100 feet is approximately 1,630 feet per second (500 meters per second.)

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

## 3.0 PRIOR LABORATORY TESTING

Geotechnical laboratory testing was performed on select samples from the prior GDI borings including the following:

- In-place moisture and density
- Consolidation
- Direct shear

Results of geotechnical testing performed on samples collected from the prior borings are presented in Appendix A.

## 4.0 GEOLOGIC AND SEISMIC HAZARDS EVALUATION

### 4.1 GEOLOGIC SETTING

The site is located within the Peninsular Ranges geomorphic province, which is characterized by northwest/southeast-trending alignments of mountains and hills and intervening basins, reflecting the influence of northwest-trending major faults and folds controlling the general geologic structural fabric of the region. This province extends northwest from Baja California into the Los Angeles Basin and west into the offshore area, including the Catalina and Channel islands. The Los Angeles Basin lies in the northern part of the Peninsular Ranges province. The Newport-Inglewood fault zone, a northwest-trending structural zone expressed at the surface by a series of discontinuous low hills, is located approximately 6 miles west-southwest of the site. The relationship of the site to local geologic features is depicted on Figure 7.

## 4.2 SOIL

Based on soil mapping available from USDA (2017), the site is located in an area indicated as "urban land with slopes of 0 to 5%." The site is developed with a parking lot. Past agricultural use of the site is not documented. Site improvements are anticipated to include drainage controls and protective features to minimize soil erosion. The potential for erosion is considered low.

## 4.3 GEOLOGIC MATERIALS

A local geologic map is presented on Figure 7 that shows the geologic units in the site region. As shown on published geologic mapping (Lamar, 1970; Dibblee, 1991; and Campbell et al, 2014), the site is underlain by Holocene and Pleistocene age alluvium.

The soil encountered in current and prior borings at the site is generally consistent with the mapped and published descriptions. As discussed previously, the soil encountered at the site includes areas of fill associated with prior site development underlain by native soil and bedrock at depth. The native soil includes dense to very dense sand, gravel, and silty sand and stiff to hard, clayey sediments underlain by Puente Formation siltstone and sandstone bedrock. Bedrock was encountered at a depth of approximately 148 feet BGS in boring B-8, conducted at the site.

## 4.4 MINERAL RESOURCES

The aggregate resource potential for the area of the site is addressed in a report titled *Mineral Land Classification of the Greater Los Angeles Area, Part IV: Classification of Sand and Gravel Resource Areas, San Gabriel Production-Consumption Region* (CGS, 1982). The report addresses the sand and gravel resource potential according to the presence or absence of significant sand and gravel deposits for use in construction-grade aggregate. The resource quality of surrounding lands is reported according to the following MRZ classification system:

- MRZ-1: Areas where adequate information indicates that significant mineral deposits are not present or where it is judged that little likelihood exists for their presence.
- MRZ-2: Areas where adequate information indicates mineral deposits are present or where it is judged that a high likelihood for their presence exists.
- MRZ-3: Areas containing mineral deposits, the significance of which cannot be evaluated from available data.
- MRZ-4: Areas where available information is inadequate for assignment to any other MRZ.

The site is situated in primarily developed terrain underlain by consolidated sediments. Economically significant sources of aggregate material were not observed within the site. The site is placed within MRZ-1 defined as "adequate information indicates that significant mineral deposits are not present."

As the project area is not presently used for mineral resource extraction and does not contain identified sources of aggregate materials, the proposed project will not result in the loss of availability of any known mineral resources. Therefore, significant impacts are not anticipated.

## 4.5 LANDSLIDES

According to the County of Los Angeles GIS database and the CGS Seismic Hazard Zone map for the Hollywood quadrangle, the site is not located within an area identified as having a potential for slope instability. The site is situated in level terrain that lacks significant natural relief or slopes. The potential for landslide or slope instability is considered low.

## 4.6 FAULTS

### 4.6.1 General

The tectonics of the Southern California area are dominated by the interaction of the North American tectonic plate and the Pacific tectonic plate, which are sliding past each other in a transform motion. Although some of the motion may be accommodated by rotation of crustal blocks such as the Transverse Ranges geomorphic province (Dickinson, 1996), the San Andreas fault zone is thought to represent the major surface expression of the tectonic boundary and is thought to be accommodating most of the transform motion between the Pacific plate and North American plate. Some of the plate motion is accommodated along other northwest-trending, strike-slip faults that are related to the San Andreas system, such as the San Jacinto, Newport-Inglewood, Elsinore-Whittier, Palos Verdes, and offshore faults. Figure 8 shows the regional faults with respect to the site location.

The site does not lie within or immediately adjacent to an Alquist-Priolo Earthquake Fault Zone designated by the State of California to include traces of suspected active faulting. Alquist-Priolo Earthquake Fault Zones have been established for regional faults, including portions of the Newport-Inglewood fault zone and Hollywood fault zone located approximately 6 miles west-southwest and 4.8 miles north-northwest of the site, respectively. The site is not located within a preliminary Fault Rupture Study Area designated by City of Los Angeles (2017). The potential for fault surface rupture to occur at the site is considered very low.

### 4.6.2 Hollywood Fault

The Hollywood fault is located approximately 4.8 miles north-northwest of the site. The Hollywood fault is an oblique, left-lateral, reverse fault that places crystalline basement rock of the Santa Monica Mountains over alluvial fan deposits of the northern Los Angeles Basin. Subsurface and geomorphic investigations indicate that the fault extends along the southern flank of the Santa Monica Mountains, from the Los Angeles River to northwestern Beverly Hills. A magnitude 6.4 capability is postulated for the fault based on the fault length and estimated slip rates (Field et al., 2008). An alignment of bedrock outcrops along Sunset Boulevard, previously thought to represent the surface trace of the fault, was found to be a paleo seacliff with the active trace of the fault being located farther to the south (Lindvall et al., 2001). The Hollywood fault is included in a state-designated zone to mitigate surface rupture effects in the built environment.

While some literature references the Hollywood-Santa Monica fault, these references are describing the combined faults along the Santa Monica Mountain front. These faults are treated as different faults and are modelled as separate faults in terms of characteristic magnitudes, distances from a site, and subsurface geometry. These fault-specific characteristics are used in determining the level of ground shaking at a site. The reason that the faults are modelled separately is that there is a separation between the faults at the extension of the northwest-trending Newport-Inglewood fault.

For the purposes of determining the potential for ground surface rupture at a site, neither the Hollywood fault nor the Santa Monica fault are relevant due to the distance from each fault to the site.

### 4.6.3 Newport-Inglewood Fault

The Newport-Inglewood fault zone is a system of northwest-trending, right-lateral, en echelon faults located approximately 6 miles west-southwest of the site. The discontinuous surface expression along a series of aligned hills and topographic rises suggests a youthful stage of development for this zone. The Newport-Inglewood fault extends offshore and trends into the Rose Canyon fault system, toward the south. The 1933 Long Beach earthquake is attributed to a segment of the Newport-Inglewood fault. A magnitude 7.4 capability is postulated based on fault length and a scenario rupture along the combined Newport-Inglewood and Rose Canyon fault systems.

#### 4.6.4 Raymond Fault

The Raymond fault is located approximately 5.3 miles north of the site. The Raymond fault is an approximately 23-kilometer-long feature exhibiting south-facing scarps and predominant left lateral motion (Weaver and Dolan, 2000). The Raymond fault is active, with trenching studies that have shown the most recent rupture on the Raymond fault to have occurred 1,000 to 2,000 years ago (Weaver and Dolan, 2000). A potential for magnitude 6.7 earthquakes is postulated for the Raymond fault based on the dimensions of the fault plane area. Portions of the Raymond fault are included within state-designated Alquist-Priolo Earthquake Fault Zones.

#### 4.6.5 Verdugo Fault

The Verdugo fault is located approximately 7.3 miles north-northeast of the site. The Verdugo fault is a northeast-dipping, reverse fault that trends along base of the San Rafael Hills and Verdugo Mountains and merges southeasterly to the Raymond fault zone (Weber et al., 1980). Probable magnitudes between 6.0 and 6.8 are estimated based on fault length.

#### 4.6.6 San Andreas Fault

The Mojave segment of the San Andreas fault zone is located along the northern margin of the San Gabriel Mountains, approximately 35 miles northeast of the site. The San Andreas fault is thought to represent the major surface expression of the tectonic boundary between the Pacific plate and the North American plate. A magnitude 7.4 earthquake is estimated for the Mojave segment of the San Andreas fault based on magnitude-length relations.

#### 4.6.7 Blind Thrust Faults

The Los Angeles Basin is underlain locally by a system of buried thrust faults that terminate at a depth of approximately 3 kilometers. These buried thrust faults include the Puente Hills thrust (PHT) system and the Upper Elysian Park fault.

The PHT is a system of buried thrust fault ramps that extend from beneath Los Angeles to the Puente Hills of eastern Los Angeles County and Orange County. Identified by subsurface data, including seismic reflection profiles, petroleum well data, and precisely located seismicity, the PHT is expressed at the surface as a series of contractional folds. Fault segments designated for the PHT include the Los Angeles, Santa Fe Springs, and Coyote Hills segments (Shaw and Shearer, 1999).

The Los Angeles segment of the PHT underlies downtown Los Angeles at a depth of approximately 4 kilometers.

This buried fault system is estimated to be capable of producing earthquakes with magnitudes of 6.5 to 6.6 on individual segments or a magnitude 7.1 earthquake as a group (Shaw et al., 2002). The Santa Fe Springs segment of the PHT is postulated to be the source of the 1987 magnitude 5.9 Whittier Narrows earthquake. A study using borehole data collected from sediments overlying the central segment of the PHT indicates that subtle folding extends to the near-surface locally and reveals four events in the past 11,000 years (Dolan et al., 2003).

The active Upper Elysian Park fault is located approximately 4.8 kilometers from the site as measured to the closest portion of the fault plane. The Upper Elysian Park thrust is a blind thrust fault located above the Los Angeles segment of the PHT system. The Elysian Park anticline and associated escarpments (MacArthur Park, Coyote Pass) and Montebello anticline provide evidence of recent activity on this fault. The vertically projected plane boundaries of this fault, as depicted by Shaw et al. (2002), are located northeast of the site. The plane of this structure plunges to the north-northeast at angles between 45 and 60 degrees (Oskin et al., 2000). This buried fault is postulated to be capable of producing earthquakes with magnitudes of 6.2 to 6.7.

These faults do not present a surface rupture hazard although they are capable of producing strong ground shaking as evidenced by the 1987 magnitude 5.9 Whittier Narrows earthquake.

## 4.7 HISTORICAL EARTHQUAKES

The site is located within the seismically active southern California region. Table 1 summarizes the historical seismic events in the site region. The locations of historical seismic events of magnitude 5.0 or greater that have occurred since 1800 are shown on Figure 8. Some of the most significant seismic events in the region are provided in Table 1 below.

**Table 1 – Summary of Historical Earthquakes**

| Event I.D. | Date | Magnitude | Distance from Site (miles) | Direction from Site |
|---|---|---|---|---|
| **Hector Mine** | 10/16/1999 | 7.1 | 120 | ENE |
| **Big Bear** | 6/28/1992 | 6.4 | 83 | E |
| **Landers** | 6/28/1992 | 7.3 | 105 | E |
| **Upland** | 2/28/1990 | 5.4 | 33 | ENE |
| **Sierra Madre** | 6/28/1991 | 5.8 | 23 | NE |
| **Chino Hills** | 7/29/2008 | 5.4 | 29 | ESE |
| **Whittier Narrows** | 10/1/1987 | 5.9 | 11 | E |
| **Sylmar** | 2/9/1971 | 6.6 | 27 | NNW |
| **Tehachapi** | 7/21/1952 | 7.3 | 44 | NW |
| **Northridge** | 1/17/1994 | 6.7 | 20 | NW |
| **Long Beach** | 3/10/1933 | 6.4 | 31 | SE |

The Long Beach, Whittier Narrows, Sylmar, and Northridge earthquakes attest to the potential for future seismic events in the southern California region to produce strong ground shaking. Any of the active faults of the region are capable of producing strong ground shaking during earthquakes. Construction according to applicable building codes can mitigate or lessen the potential for damage to site facilities.

## 4.8 GEOTECHNICAL HAZARDS

### 4.8.1 Fault Rupture

The site does not lie within or immediately adjacent to an Alquist-Priolo Earthquake Fault Zone, designated by the State of California to include traces of suspected active faulting. The site is not included in a city-designated fault hazard zone. The potential for surface fault rupture at the site is considered to be very low.

### 4.8.2 Strong Ground Motions

The site is located within a seismically active region; therefore, strong ground shaking may occur during the design life of the proposed project. However, this hazard is common in Southern

California and can be mitigated by designing the proposed structures in accordance with the current Los Angeles Building Code (LABC).

### 4.8.3 Slope Stability and Landslides

According to City of Los Angeles Navigate LA (2017) and CGS (1998), the site is not located within an area identified as having a potential for slope instability. Significant natural slopes are not present on the site and the potential for slope instability and/or landslides is very low.

### 4.8.4 Erosion

The site is mantled by artificial fill soil overlying native sediments with moderate fines. The site has been developed with buildings and associated flatwork since at least 1948. The planned development is anticipated to include improvements that will conceal site soil; therefore, the potential for erosion is considered very low.

### 4.8.5 Liquefaction and Seismically Induced Settlement

Liquefaction is a process in which strong ground shaking causes saturated soil to lose its strength and behave as a fluid. Ground failure associated with liquefaction can result in severe damage to structures. The geologic conditions for increased susceptibility to liquefaction are (1) shallow groundwater (generally less than 50 feet in depth), (2) the presence of unconsolidated, sandy alluvium, typically Holocene in age, and (3) strong ground shaking. All three of these conditions must be present for liquefaction to occur.

The site is not located within an area identified as having a potential for liquefaction by CGS (1999). Based on an historical high groundwater depth and the dense nature of the native soil at the site, liquefaction is not considered a hazard at the site.

### 4.8.6 Tsunamis, Inundation, Seiche, and Flooding

The site is not located in a coastal area; therefore, tsunamis are not considered a hazard at the site. According to the County of Los Angeles General Plan (1990), the site is not located within a potential inundation area for seismically induced dam/reservoir failure.

A portion of the site is located in an area mapped as having a one percent chance of flooding to a depth of less than one foot (Zone X), based on a review of FEMA FIRM panel 06037C1617G (2018).

### 4.8.7 Subsidence

Land subsidence may be induced from withdrawal of oil, gas, or water from wells. Based on a search of the CalGEM (formerly known as Division of Oil, Gas, and Geothermal Resources [DOGGR]) GIS Well Finder online tool the site is located within the Los Angeles Downtown oil/gas field. Several plugged wells lie within a half mile of the site. According to our review of the available information from CalGEM, the likelihood of land subsidence caused by oil or gas withdrawal from oil wells is low.

### 4.8.8 Expansive and Corrosive Soil

Plasticity index values available from USDA (2017) indicate the soil at the site is non-plastic. The soil at the site is generally considered non-expansive based on the reported plasticity index values. Samples collected from the current and prior nearby borings did not exhibit expansive characteristics. Chemical testing performed for a prior site investigation indicate a "negligible" anticipated exposure to sulfate attack.

### 4.8.9 Oil Wells and Methane Gas

As stated above, the site is located within the boundaries of the Los Angeles Downtown oil field. The closest well is located approximately a quarter mile south-southwest of the site. This consists of a plugged and abandoned drill hole designated as API No. 0403730413.

The site is located in a "methane zone" designated by the City of Los Angeles Navigate LA (2017). This zone identifies areas that have a potential for accumulation of methane and other volatile gases to occur in subsurface strata. As such, methane mitigation provisions will be required in accordance with LADBS Ordinance No. 175790. The required level of methane mitigation will be based on the results of a soil gas survey.

### 4.8.10 Volcanic Eruption

The site is not located in an area of recent volcanic eruption. The potential for volcanic activity at the site is very low.

### 4.8.11 Radon Gas

The site is not located in an area of high radon potential for indoor areas (CGS, 2005). The potential for radon gas accumulation is considered low.

### 4.8.12 Off-Site Impacts

Potential geotechnical impacts to off-site areas are not anticipated due to requirements regarding grading permitting, erosion control, and avoidance of non-permitted disturbance to off-site areas required by local regulations.

## 5.0 CONCLUSIONS

### 5.1 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 Los Angeles and can be mitigated by following LABC seismic design requirements.

The site is also located within a LADBS-designated methane zone and appropriate methane mitigation provisions are required in accordance with the LADBS Ordinance.

### 5.2 FOUNDATIONS

The soil anticipated at the foundation level of the proposed development generally consists of very dense sand with areas of very stiff silt. These soils are suitable for supporting the proposed tower on a mat foundation and the proposed podium on a mat foundation or spread and continuous footings.

Recommendations for foundation design are presented in Section 6.1.

### 5.3 SHORING, EXCAVATION AND PERMANENT BELOW-GRADE WALLS

Temporary shoring consisting of soldier piles or sheet piles with tiebacks will be required to provide support for the mass excavation that may extend on the order of 70 feet BGS. Temporary shoring may encounter gravel and cobbles typically in the upper approximately 10 to 34 feet BGS. In addition, drilling could encounter localized zones of perched water.

If the planned excavation will extend below a 1H:1V plane from the bottom of adjacent footings, surcharge loading will be required on temporary shoring and below-grade buildings walls. This will include the six-story parking garage to the south and possibly other buildings surrounding the site.

Prior to the start of construction, tieback locations and inclinations should be checked to verify that they do not interfere with existing foundations or buried utilities. Remnants of prior developments may also be present at the site within the planned mass excavation.

Our preliminary surcharge loading for permanent building walls below grade and temporary shoring are presented in Sections 6.2 and 6.4

### 5.4 STORM WATER INFILTRATION

Stormwater infiltration is feasible at the site provided infiltration is performed in a manner that does not adversely impact the performance of the foundation system and is in conformance with City of Los Angeles Bureau of Sanitation guidelines and regulations. Deep drywells are feasible within the footprint of the proposed tower and parking garage provided the recommendations presented herein are followed.

Our recommendations for storm water infiltration are presented in Section 6.7.

### 5.5 FLOOR SLAB SUPPORT

The tower floor slab will be established above the top of the mat foundation; therefore, the slab in this area can be supported on properly compacted fill soil. Beneath the podium, the floor slab may be established on the stiff and dense native soil present at the site or a mat foundation.

Recommendations for floor slab support are presented in Section 6.5.

### 5.6 ON-SITE MATERIAL

Large-sized particles will be generated from the on-site excavations, particularly within the upper 10 to 34 feet. These large-sized particles will generally be difficult to re-use on site without processing to meet the specifications presented herein for fill material. The remainder of on-site materials are suitable for re-use in required fills.

Concrete and asphalt generated from the required demolition are suitable for re-use in compacted fills provided the concrete and asphalt are processed and placed as recommended herein.

## 6.0 RECOMMENDATIONS

### 6.1 FOUNDATIONS

#### 6.1.1 General

The tower may be supported on a mat foundation and the podium and parking garage may be supported on spread footings each established in the dense to very dense and stiff to hard native soil present at the planned foundation levels.

Foundation loading information was not available at the time this report was prepared; therefore, preliminary recommendations are presented below based on assumed foundation loading. Once specific foundation loading is available, we will need to re-evaluate the recommendations presented below and issue an addendum either confirming or updating these preliminary foundation recommendations.

#### 6.1.2 Mat Foundation

Based on similar projects, we anticipate the average applied bearing pressure on the order of 15,000 psf may be applied by the mat foundation. Noting that the planned excavation will result in a pressure release on the order 7,200 psf, the net average applied bearing pressure from the mat foundation will be on the order of 7,800 psf.

We estimate that total settlement of the tower mat foundation will be on the order of 1.5 inches or less and that differential settlement of the mat foundation will be on the order of ½ inch or less.

For preliminary design of the mat foundation, a soil modulus of subgrade reaction equal to 120 pci for the stiff to hard and dense to very dense native soil may be used to compute the deformation of the mat foundation.

The recommended modulus value should be used to evaluate the deformation of the mat foundation for compatibility with the settlement estimates presented above.

### 6.1.3 Spread Footings

The proposed podium and parking garage may be supported on spread and continuous footings established in the dense to very dense and stiff to hard native soil. Spread and continuous 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 10,000 psf and can be increased by 500 psf for each additional foot of width and/or embedment depth to a maximum value of 13,500 psf. The additional 500 psf increase is assumed to begin below 2 feet from the lowest adjacent grade or floor level.

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.

Based on data available from similar projects, we anticipate dead-plus-live column loading for the proposed parking garage will be on the order of 400 to 1,300 kips. Based on these assumed values, we estimate total settlement for the proposed spread footings to be on the order of 1 inch or less and differential settlement across the parking garage and tower building footprint to be on the order of ½ inch or less, as the influence of the mat foundation would also contribute to settlement of the spread footings.

### 6.1.4 Lateral Resistance

For mat foundations and spread and continuous footings, lateral loading may be resisted by foundations using an ultimate passive pressure of 800 psf for footings where the concrete is placed directly against the undisturbed stiff or dense alluvial soil.

An ultimate coefficient of friction equal to 0.6 may be used when calculating resistance to sliding for foundations bearing on undisturbed stiff to hard or dense to very dense native soil. However, if a methane barrier is placed beneath the foundation, the ultimate coefficient of friction shall be reduced to 0.25.

The ultimate passive resistance and the ultimate frictional resistance may be used in combination provided the ultimate passive resistance is reduced to 400 psf to account for the magnitude of deformation required to mobilize the full passive resistance when considering short-term seismic and wind loading.

### 6.1.5 On-Site Infiltration

On-site stormwater infiltration should be performed at least 40 feet below the bottom of the mat foundation and at least 30 feet below the bottom of the footings and may extend to at least a depth of 115 feet BGS, which is at least 10 feet above the seasonal average groundwater level based on the data from our current and prior borings at the site.

## 6.2 BELOW-GRADE WALLS

### 6.2.1 Design Lateral Earth Pressure

For static conditions, drained below-grade building walls should be designed to resist a trapezoidal-shaped at-rest lateral earth pressure distribution equal to 28H psf as shown on Figure 10.

For seismic loading conditions, drained below-grade building walls should be designed to resist a triangular-shaped active lateral earth pressure distribution equal to 35H psf in conjunction and a triangular-shaped seismic lateral earth pressure distribution equal to 15H psf as shown on Figure 11.

The upper 10 feet of the below-grade building walls should also be designed to resist a uniform lateral pressure of 100 psf to account for normal traffic loading as shown on Figures 10 and 11.

The load combination (active and seismic earth pressure) and the shape of the seismic pressure distribution are each based on Seismic Earth Pressures on Cantilevered Retaining Structures (Atik and Sitar, 2010) and Seismic Earth Pressures: Fact or Fiction (Lew, Sitar, and Atik, 2010).

Although not currently planned, if the surface at the top of the wall is sloped, the recommended lateral earth pressures should be increased as indicated in Table 2.

**Table 2. Lateral Earth Pressures Increases**

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

### 6.2.2 Surcharge Loading from Adjacent Structures and Vehicular Loads

The planned excavations will extend below the foundations for the adjacent buildings located at 1111 South Hill Street. The remaining adjacent buildings are a sufficient distance (greater than 60 feet) from the site; therefore, the planned excavation will not extend below those existing or proposed foundations.

Based on the information available from the original geotechnical report, we recommend a uniform lateral earth pressure equal to 600 psf be applied to the northern portion of the proposed east below-grade building wall.

Please note the assumed bearing values for the six-level parking garage and 32-story AT&T Center Tower will require validation from the structural engineer and it may be necessary to update these surcharge recommendations once foundation loading information is available.

To account for vehicular surcharge loads at each of the below-grade walls, we recommend utilizing a lateral load distribution of 100 psf within the upper 10 feet.

### 6.2.3 Wall Back-Drainage Provisions

Permanent retaining walls should be constructed with adequate back-drainage to prevent the buildup of hydrostatic pressure behind the walls. Typically, a pre-fabricated geo-composite drainage board is fixed to the shoring wall and the below-grade building wall is constructed by the placement of shotcrete directly against the drainage board.

In addition to drainage boards, the Grading Division requires the installation of rock pockets consisting of 1 cubic foot of crushed rock spaced at 8-foot centers around the perimeter of the below-grade building walls to promote drainage.

The Grading Division requires each rock pocket to be drained at each 8-foot center back into the building and discharged to a suitable outlet. Drainage and discharge provisions should be included by others on the design drawings.

The impact of the Grading Division's back-drainage requirements on the design and construction of the proposed development is complicated by an LADBS requirement related to methane mitigation. The methane mitigation requirement is to include vent risers at each penetration through the below-grade building wall for the purpose of mitigating the potential for methane gas to enter the building through the penetrations. The combined result of the two requirements is that an alternative method to provide back-drainage in a manner that meets requirement may be desirable.

One alternative method includes perimeter drainage elements at the base of the below-grade building walls. Per LADBS requirements, the subject pipe only needs to be drained into the building at fewer locations and conveyed to the building sump system.

Therefore, the number of vent risers required as a function of the wall back-drainage system could be significantly reduced.

## 6.3 FREE-STANDING RETAINING WALLS

### 6.3.1 Foundations

Free-standing retaining wall foundations should be established on at least 2 feet of properly compacted fill soil. Wall foundations supported on properly compacted fill and established at least 2 feet below the lowest adjacent grade may be designed using an allowable bearing pressure of 4,000 psf. To resist lateral loading, a coefficient of friction equal to 0.4 may be used in conjunction with a passive pressure of 300 psf.

### 6.3.2 Design Lateral Earth Pressures

Drained, 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 2.

### 6.3.3 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.

### 6.3.4 Temporary Vertical Cuts and Construction Slopes

If necessary, temporary, unsurcharged slopes should not exceed a 1H: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 dense native material but should not exceed 4 feet in height.

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.

## 6.4 TEMPORARY SHORING

### 6.4.1 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 where the maximum value is 26H psf as shown on Figure 12. Internally braced shoring should be designed to resist a trapezoidal earth pressure distribution where the maximum value is equal to 24H psf as shown on Figure 13.

In addition to the lateral earth pressures from the weight of the retained soil, surcharge loading from the adjacent building foundations will develop on the temporary shoring wall as well. As recommended in Section 6.2.2, based on the information available from the original geotechnical report, we recommend a uniform lateral earth pressure equal to 600 psf be applied to the northerly portion of the proposed east shoring wall.

For cantilevered and braced shoring design, where the surface at the top of the shoring is sloped, the recommended lateral earth pressures should be increased as indicated in Table 2.

The design of temporary shoring walls should consider the location of construction cranes and other potentially heavy equipment or loads that may act against the shoring system.

### 6.4.2 Soldier Pile Design and Installation

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 400 psf per foot of depth, up to a maximum of 4,000 psf of depth. 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 piles can be backfilled with controlled low strength material (CLSM) in conformance with the City of Los Angeles Department of Building and Safety information bulletin P/BC 2014-121, 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 required depth of embedment should be determined based on the provisions of and Section 1806.3.4 and Section 1807.3.2.1 of the 2020 Los Angeles Building Code.

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 400 psf for drilled solider piles. For soldier piles that are vibrated into the supporting soil, the frictional resistance between the soldier piles and the soil below the excavated level may be taken as 800 psf.

Installation of soldier piles by vibration techniques should not be installed where sensitive structures or utilities are present. We recommend the feasibility of vibration installation be evaluated by the shoring contractor.

### 6.4.3 Sheet Piles

The allowable lateral bearing value (passive value) of the native soil below the level of excavation may be assumed to be 400 psf per foot of depth, up to a maximum of 4,000 psf of depth. To resist vertical loading from tiebacks, a coefficient of friction between the sheets and the retained earth of 0.4 can be used. Due to the small surface area of the sheets, we do not recommend assuming end resistance.

It will be difficult/labor intensive to install back of wall drainage behind the sheet piles if they will be used as the permanent walls. If back of wall drainage is not provided, it will be necessary to design the basement for hydrostatic pressures.

### 6.4.4 Timber Lagging Design

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 6 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.

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 35 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.

### 6.4.5 Tiebacks

The capacities of anchors should be determined by testing the initial anchors as outlined below. We anticipate that gravity-filled anchors will achieve an allowable bond strength of 1 kips 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 used, we estimate 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 use 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.

Total deflection during the 24-hour, 200 percent tests should not exceed 12 inches during loading. 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. Anchor 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 the production anchors should be pre-tested to at least 150 percent of the design load. 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.

### 6.4.6 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 footings (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 6,000 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 movement of the shoring, the Rakers should be tightly wedged against the footings and/or shoring system.

### 6.4.7 Monitoring

Some means of monitoring the performance of the shoring system is recommended. Monitoring should consist of periodic surveying of the lateral and vertical locations of the tops of all the soldier piles. When 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.

Additionally, we recommend an existing condition survey be performed to document the condition of the adjacent buildings along the eastern and southern edge of the proposed building. The survey should include photographs and placement of monitoring devices (crack monitors, for instance) if appropriate and should be performed prior to the start of the shoring installation.

### 6.4.8 Shoring Construction Consideration

Due to the presence of localized fill material, cobbles, and boulders; granular soil that may be subject to caving; and potential groundwater seepage perched on fine-grained layers at depth, difficult drilling is expected for soldier pile and tieback installations.

## 6.5 FLOOR SLABS

The tower floor slab will be established above the top of the mat foundation and the remainder of the podium could be established on a mat or stiff to dense soil. Where slabs are constructed on mats they can be supported on properly compacted fill soil. Floor slabs bearing on properly compacted fill and/or stiff or dense native alluvial soil can support up to an estimated 400 psf area loading.

Typically, it is common to include a capillary break beneath building floor slabs that will have moisture-sensitive flooring. If used, the capillary break should consist of 6 inches of gravel underlying a 15-mil HDPE membrane.

However, since the lowest levels will be used for parking, and will not include moisture-sensitive flooring, the capillary break section may be omitted at the owner’s discretion.

## 6.6 SEISMIC DESIGN

### 6.6.1 LABC Seismic Design Parameters

In accordance ASCE-7-16, Chapter 20, and based on the results of the prior PS-logging performed immediately adjacent to the site, the seismic site classification is C.

For preliminary design purposes, Table 3 presents LABC-prescribed seismic design parameters for Site Class C noting that we will develop a site specific response spectrum (SSRS) in accordance with our performance-based design services described briefly in Section 6.6.2.

**Table 3 –Seismic Design Parameters**

| **Criteria** | **Value** |
|---|---|
| MCE$_R$ Ground Motion at Short Periods, $S_s$ | 1.939 |
| MCE$_R$ Ground Motion at 1Second Period, $S_1$ | 0.689 |
| Site Class | C |
| Site-Modified Spectral Acceleration Value at Short Periods, $S_{MS}$ | 2.326 |
| Site-Modified Spectral Acceleration Value at 1 Second Period, $S_{M1}$ | 0.965 |
| Design Spectral Response Acceleration at short periods, $S_{DS}$ | 1.551 |
| Design Spectral Response Acceleration at 1 second period, $S_{D1}$ | 0.643 |
| MCE$_G$ Peak Ground Acceleration, $PGA_M$ | 0.994 |

Our SSRS will be developed for two levels of ground shaking. These levels of ground shaking correspond to the Risk-Targeted Maximum Considered Earthquake (MCER) and Design Earthquake (DE) determined in accordance with ASCE 7-16

### 6.6.2 Performance-Based Design Parameters

As part of our services during the design development phase of the project, we will develop SSRS and generate earthquake time history records in accordance with the provisions outlined in LADBS Information Bulletin P/BC 2017-123 titled Alternative Design Procedure for Seismic Analysis and Design of Tall Buildings and Buildings Utilizing Complex Structural Systems.

Time-history record will be developed in general accordance with Chapter 16 of ASCE-7-16 in collaboration with the project structural engineer and the geotechnical and structural peer review consultants.

## 6.7 STORM WATER INFILTRATION

In accordance with the City of Los Angeles Bureau of Sanitation Low Impact Development guidelines, the invert of on-site infiltration must be at least 10 feet above the seasonal high groundwater level. Considering that the seasonal high level is at least 125 feet BGS based on the data from our current and prior borings, the invert of stormwater infiltration systems should be 115 feet BGS or above.

As recommended in Section 6.1.5, stormwater infiltration should be performed at least 40 feet below the bottom of the mat foundation and at least 30 feet below the bottom of the parking garage spread footings assuming a design infiltration rate of 15 inches per hour.

Drywells should be spaced at least 10 feet from proposed spread footings.

## 6.8 SITE PREPARATION

### 6.8.1 General

Site preparation for this project will primarily include exposing the bottom of foundations and floor slabs and preparing soil at the bottom of trenches, behind below-grade walls, and behind free-standing site retaining walls to receive backfill. For foundation and floor slab support, the exposed bottoms do not require special preparation, except when disturbed by construction activities.

In this case, loose or otherwise disturbed soil should be removed and either replaced with structural concrete for footing bottoms or re-compacted prior to the placement of concrete for floor slabs. 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 6.8.2.

### 6.8.2 Groundwater Seepage

If groundwater seepage is encountered within excavations, the seepage should be collected and disposed of in accordance with regulatory guidelines. If seepage results in disturbance and/or softening of the excavation bottom, the disturbed material should be removed and replaced with 1-inch-minus crushed rock to provide a firm and unyielding base.

## 6.9 GRADING CONSIDERATIONS

### 6.9.1 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.

### 6.9.2 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 D1557. Cohesive fill, though not anticipated for this project, should be compacted to at least 90 percent of the maximum dry density, as determined by ASTM D1557, 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.

### 6.9.3 Site Drainage

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

## 6.10 MATERIALS FOR FILL

### 6.10.1 General

The fill material should be free of organic matter and other deleterious material and, in general, should consist of particles no larger than 6 inches in largest dimension. It should be noted that cobbles exceeding the size tolerances were observed in the upper approximately 5 to 25 feet of the on-site material. 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.

### 6.10.2 On-Site Native Soil

The on-site native granular soil is suitable for use in the required fills provided particles larger than 3 inches in largest dimension are removed. Where larger-sized material is used, the percentage of these materials in a representative section of the fill should be limited to 5 percent.

The on-site native silty soil is not considered suitable for use in structural fills or within 2 feet of floor slabs or other flatwork, but may be used as secondary fill in landscaping areas.

### 6.10.3 Imported Granular Material

Imported fill material should have a sand equivalent of at least 35 and should be approved by our firm prior to import to the site.

## 7.0 SUPPLEMENTAL FIELD EXPLORATIONS AND ANALYSIS

As noted herein, the recommendations presented herein are considered preliminary pending the initiation of the design development phase of the project. At that time, our assumptions regarding foundation loading for the proposed tower and parking garage should be re-evaluated, along with foundation loading and corresponding surcharge loading from the adjacent structures.

Supplemental field explorations are also recommended to confirm the preliminary recommendations presented herein remain applicable.

## 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
- Installation of drywells

## 9.0 LIMITATIONS

We have prepared this preliminary report for use by DTLA South Park Properties Propco I, 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.

## 10.0 CLOSING

We sincerely appreciate the opportunity to provide professional services for this project and look forward to working with you on this project. Please contact us at your convenience to discuss any questions you may have regarding this report.

Sincerely,
**Langan Engineering and Environmental Services, Inc.**

ANDREW J. ATRY
C 84728
signed 12/16/20

Andrew J. Atry, PE
Project Engineer

Shaun Wilkins

SHAUN H. WILKINS
No. 2665
signed 12/16/20

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Senior Project Geologist

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GE2493
signed 12/16/20

Christopher J. Zadoorian, GE
Associate

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# FIGURES

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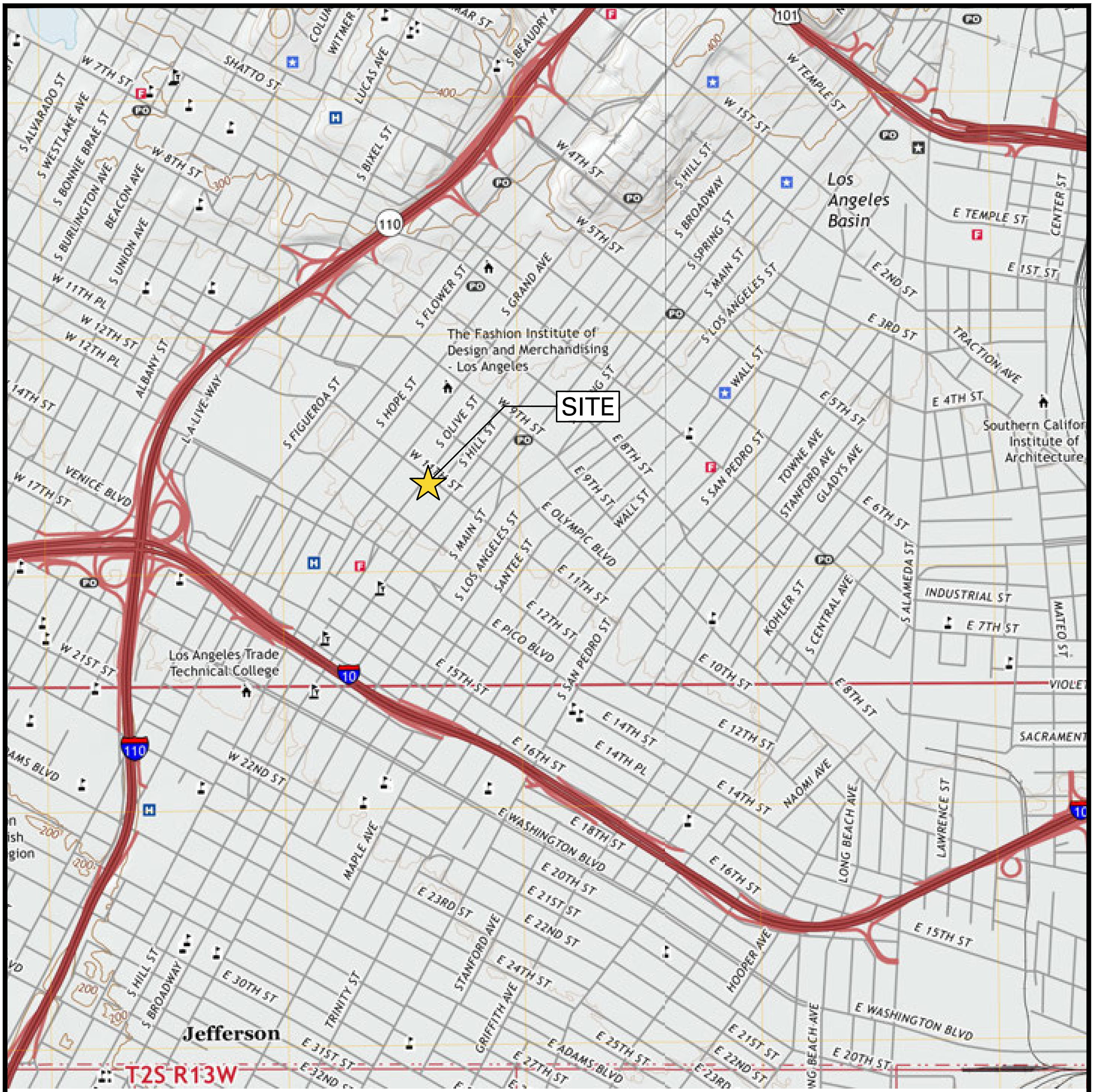
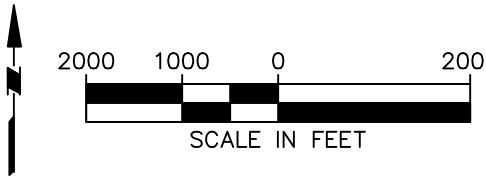


**LEGEND:**

★ SITE LOCATION



REFERENCE: USGS 7.5-MINUTE TOPOGRAPHIC MAP OF THE HOLLYWOOD AND LOS ANGELES, CA QUADRANGLES (2018).

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DUBAI ISTANBUL LONDON PANAMA
Langan Engineering & Environmental Services, Inc.

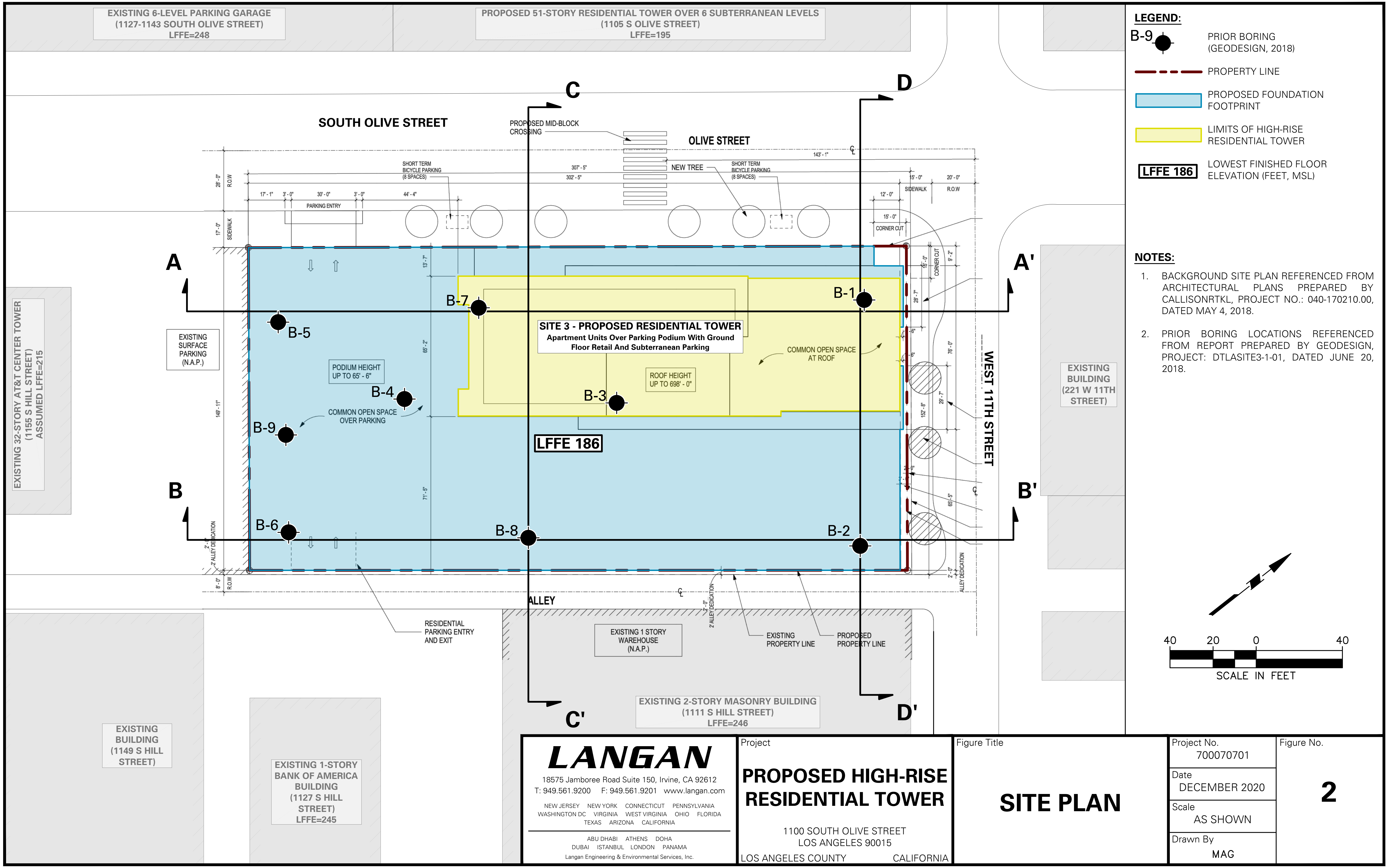
Project
**PROPOSED HIGH-RISE RESIDENTIAL TOWER**

1100 SOUTH OLIVE STREET
LOS ANGELES 90015
LOS ANGELES COUNTY CALIFORNIA

Figure Title
**SITE VICINITY MAP**

Project No.
700070701
Date
DECEMBER 2020
Scale
AS SHOWN
Drawn By
MAG

Figure No.
**1**



**LEGEND:**

| | |
|---|---|
| B-9 ● | PRIOR BORING (GEODESIGN, 2018) |
| ▬ ▬ ▬ | PROPERTY LINE |
| ▭ | PROPOSED FOUNDATION FOOTPRINT |
| ▭ | LIMITS OF HIGH-RISE RESIDENTIAL TOWER |
| LFFE 186 | LOWEST FINISHED FLOOR ELEVATION (FEET, MSL) |

**NOTES:**

1. BACKGROUND SITE PLAN REFERENCED FROM ARCHITECTURAL PLANS PREPARED BY CALLISONRTKL, PROJECT NO.: 040-170210.00, DATED MAY 4, 2018.
2. PRIOR BORING LOCATIONS REFERENCED FROM REPORT PREPARED BY GEODESIGN, PROJECT: DTLASITE3-1-01, DATED JUNE 20, 2018.

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Project

**PROPOSED HIGH-RISE RESIDENTIAL TOWER**

1100 SOUTH OLIVE STREET
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LOS ANGELES COUNTY CALIFORNIA

Figure Title

**SITE PLAN**

| | |
|---|---|
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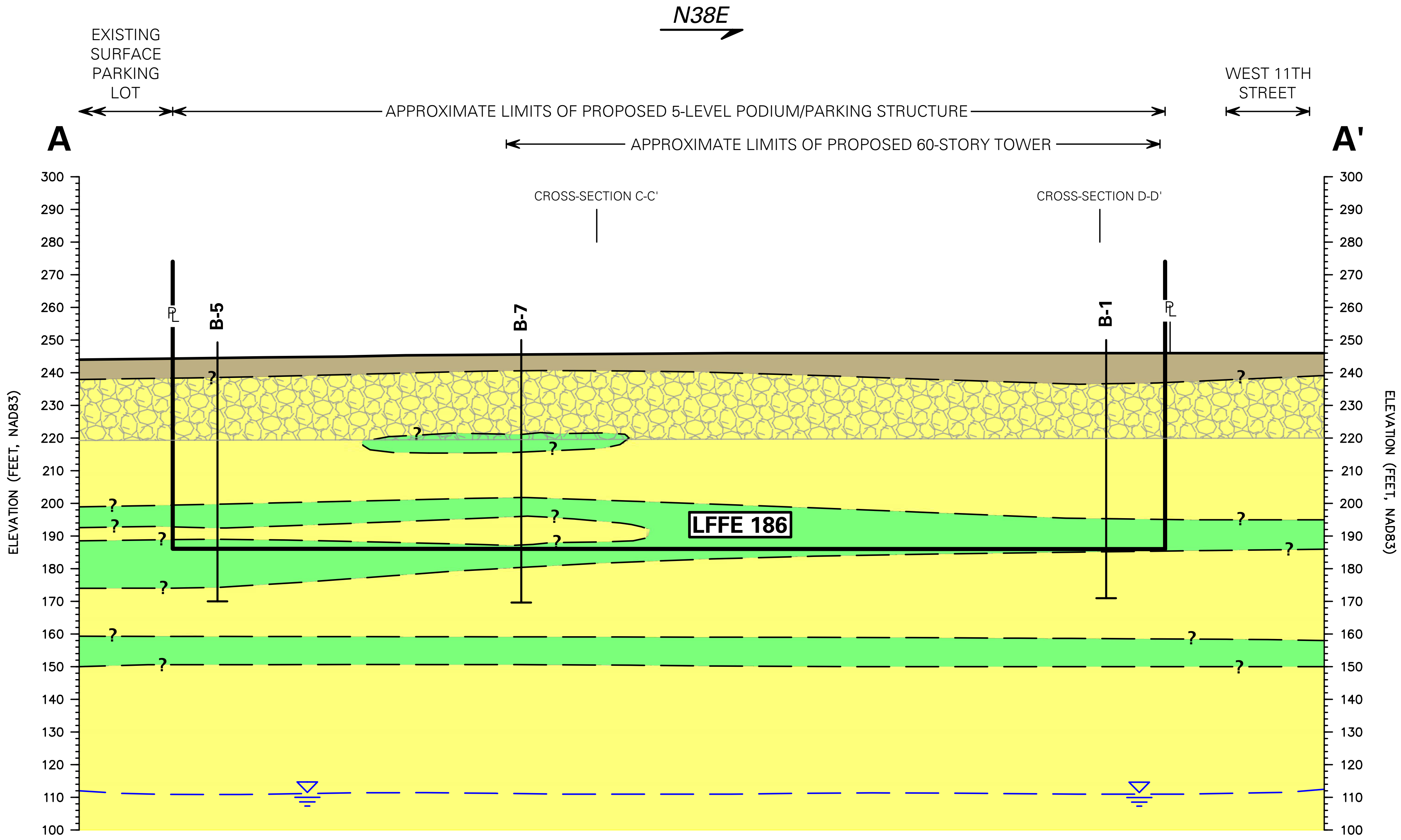
| Date | DECEMBER 2020 |
| Scale | AS SHOWN |
| Drawn By | MAG |

Figure No.

**2**

Filename: \\Langan.com\data\IRV\data7\700070701\Project Data\CAD\01\2D-DesignFiles\GEOTECHNICAL FIGURES.dwg Date: 12/16/2020 Time: 11:16 User: mgalvan Style Table: Langan.stb Layout: 2 - SP



**LEGEND:**

| | |
|---|---|
| 🟫 | UPPER SILT OR FILL |
| 🟩 | STIFF TO HARD SILT AND CLAY |
| 🟨 | DENSE TO VERY DENSE SAND |
| ▨ | COBBLES |
| LFFE 186 | PROPOSED LOWEST FINISHED FLOOR ELEVATION (FEET, MSL) |
| — ? — | INFERRED GEOLOGIC CONTACT |
| — ▽ — | APPROXIMATE GROUNDWATER TABLE |

**NOTES:**

1. FIGURE DISPLAYS GENERALIZED SUBSURFACE CONDITIONS FOR A DETAILED DESCRIPTION OF CONDITIONS ENCOUNTERED, REFER TO BORING LOGS.
2. REFER TO SITE PLAN FOR LOCATION OF CROSS-SECTION.

40 20 0 40
SCALE IN FEET (1V:1H)

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Figure Title
## CROSS SECTION A-A'

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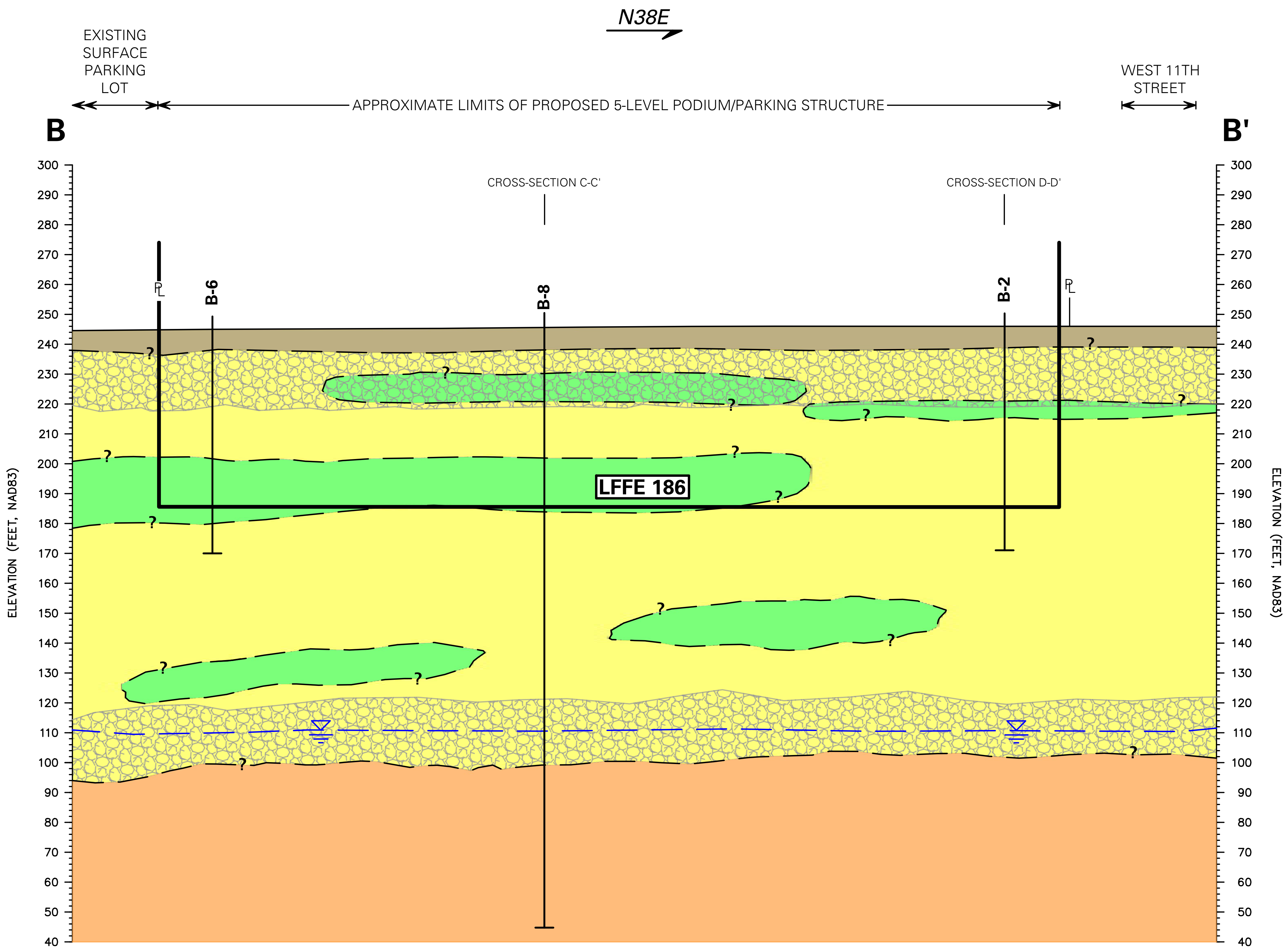
Drawn By CDC
Figure No. 3

Filename: \\Langan.com\data\IRV\data7\700070701\Project Data\CAD\01\2D-DesignFiles\GEOTECHNICAL FIGURES.dwg Date: 12/16/2020 Time: 11:22 User: mgalvan Style Table: Langan.stb Layout: XS-A



**LEGEND:**

| | |
|---|---|
| 🟫 | UPPER SILT OR FILL |
| 🟩 | STIFF TO HARD SILT AND CLAY |
| 🟨 | DENSE TO VERY DENSE SAND |
| 🟧 | BEDROCK |
| ▨ | COBBLES |
| LFFE 186 | PROPOSED LOWEST FINISHED FLOOR ELEVATION (FEET, MSL) |
| — ? — | INFERRED GEOLOGIC CONTACT |
| — ▽ — | APPROXIMATE GROUNDWATER TABLE |

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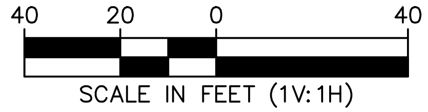

SCALE IN FEET (1V:1H)

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Figure Title

**CROSS SECTION B-B'**

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CDC

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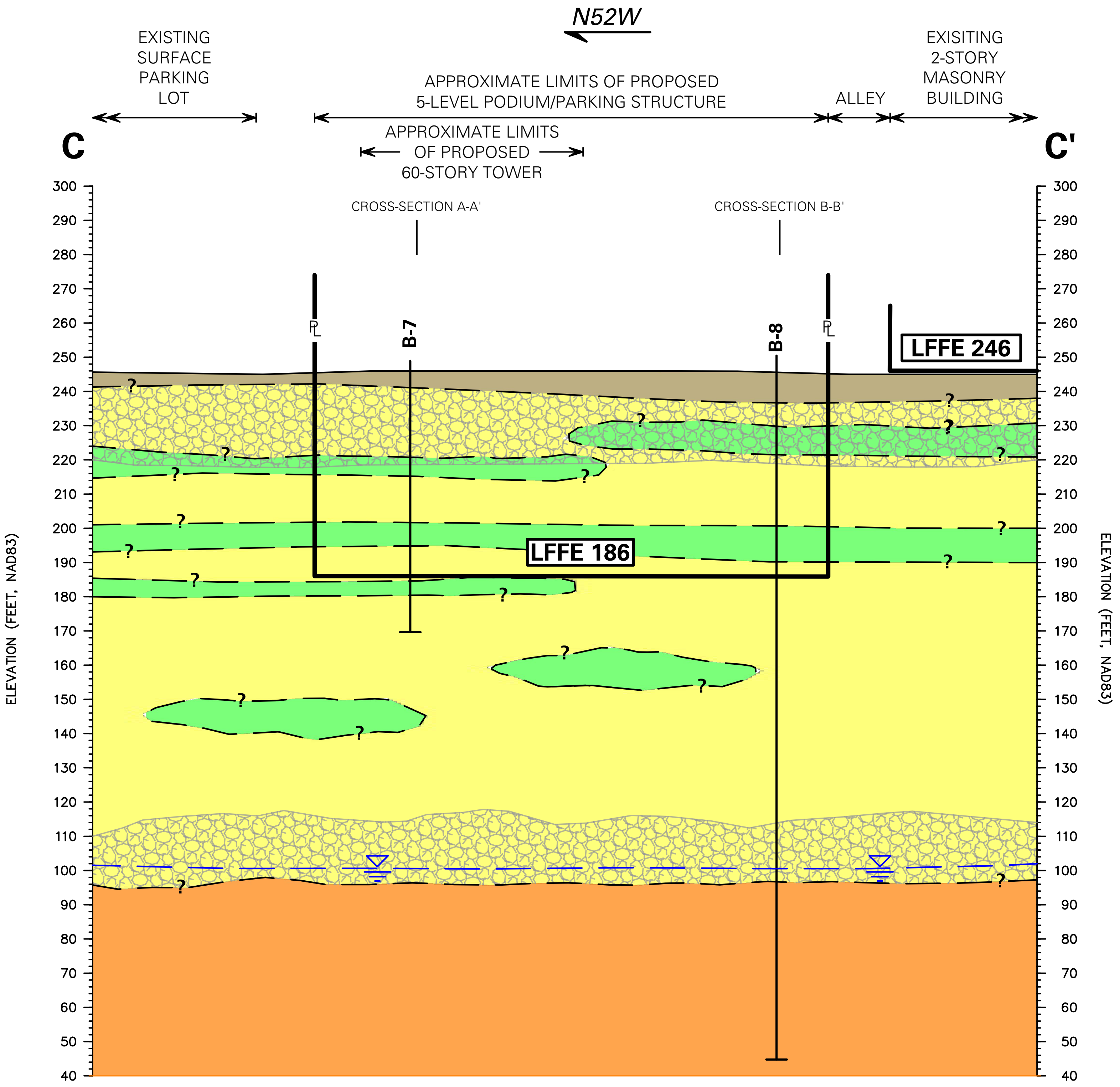
**4**

Filename: \\Langan.com\data\IRV\data7\700070701\Project Data\CAD\01\2D-DesignFiles\GEOTECHNICAL FIGURES.dwg Date: 12/16/2020 Time: 11:25 User: mgalvan Style Table: Langan.stb Layout: XS-B



**LEGEND:**

| | |
|---|---|
| 🟫 | UPPER SILT OR FILL |
| 🟩 | STIFF TO HARD SILT AND CLAY |
| 🟨 | DENSE TO VERY DENSE SAND |
| 🟧 | BEDROCK |
| ▨ | COBBLES |
| LFFE 186 | PROPOSED LOWEST FINISHED FLOOR ELEVATION (FEET, MSL) |
| — ? — | INFERRED GEOLOGIC CONTACT |
| — ▽ — | APPROXIMATE GROUNDWATER TABLE |

**NOTES:**

1. FIGURE DISPLAYS GENERALIZED SUBSURFACE CONDITIONS FOR A DETAILED DESCRIPTION OF CONDITIONS ENCOUNTERED, REFER TO BORING LOGS.
2. REFER TO SITE PLAN FOR LOCATION OF CROSS-SECTION.

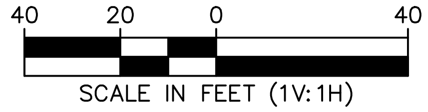


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Figure Title

**CROSS SECTION C-C'**

Project No.
700070701

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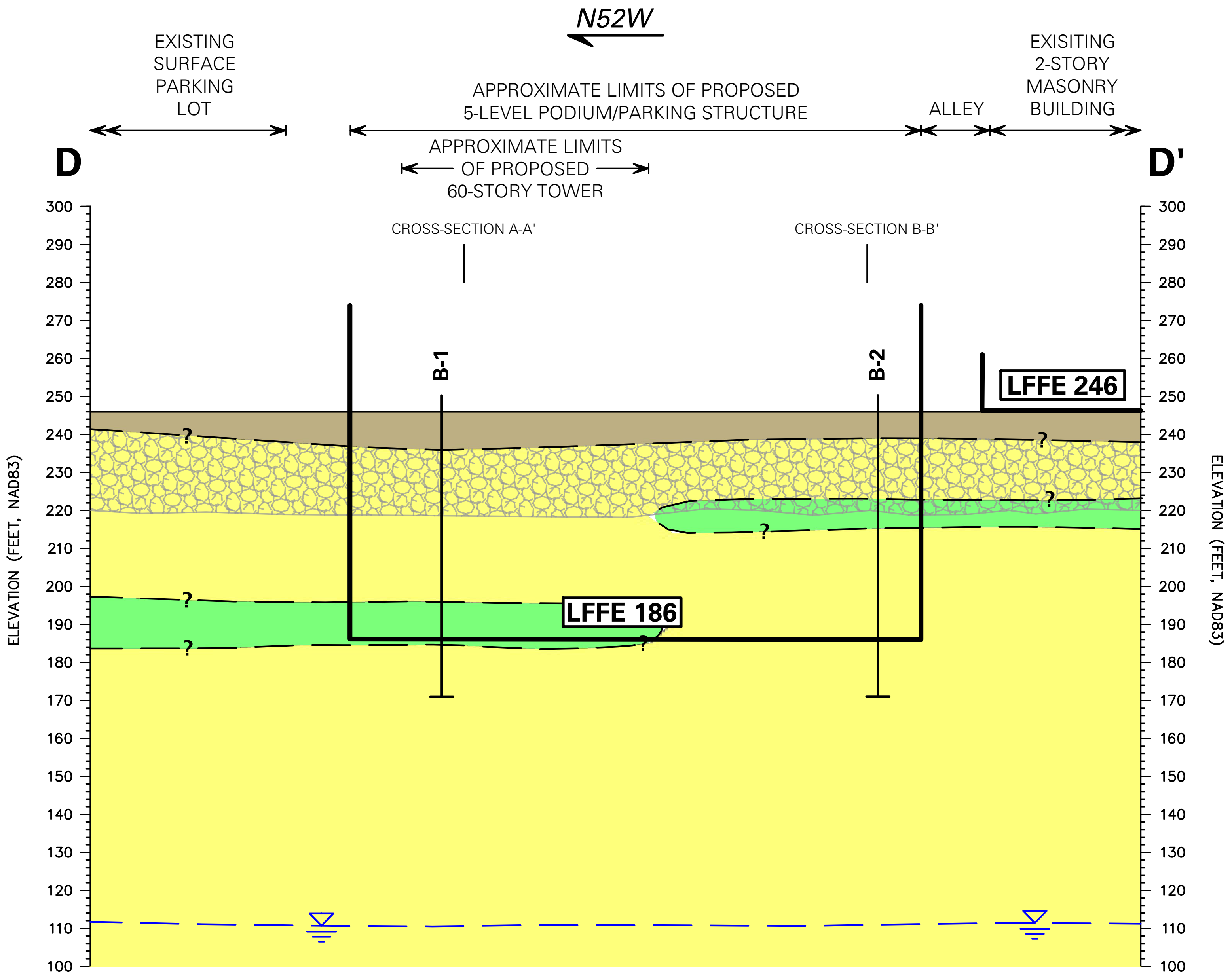
Figure No.
**5**

Filename: \\Langan.com\data\IRV\data7\700070701\Project Data\CAD\01\2D-DesignFiles\GEOTECHNICAL FIGURES.dwg Date: 12/16/2020 Time: 11:27 User: mgalvan Style Table: Langan.stb Layout: XS-C



**LEGEND:**

| | |
|---|---|
| 🟫 | UPPER SILT OR FILL |
| 🟩 | STIFF TO HARD SILT AND CLAY |
| 🟨 | DENSE TO VERY DENSE SAND |
| ▨ | COBBLES |
| LFFE 186 | PROPOSED LOWEST FINISHED FLOOR ELEVATION (FEET, MSL) |
| — ? — | INFERRED GEOLOGIC CONTACT |
| — ▽ — | APPROXIMATE GROUNDWATER TABLE |

**NOTES:**

1. FIGURE DISPLAYS GENERALIZED SUBSURFACE CONDITIONS FOR A DETAILED DESCRIPTION OF CONDITIONS ENCOUNTERED, REFER TO BORING LOGS.
2. REFER TO SITE PLAN FOR LOCATION OF CROSS-SECTION.

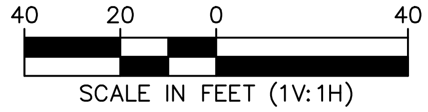


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Figure Title
**CROSS SECTION D-D'**

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Figure No.
**6**

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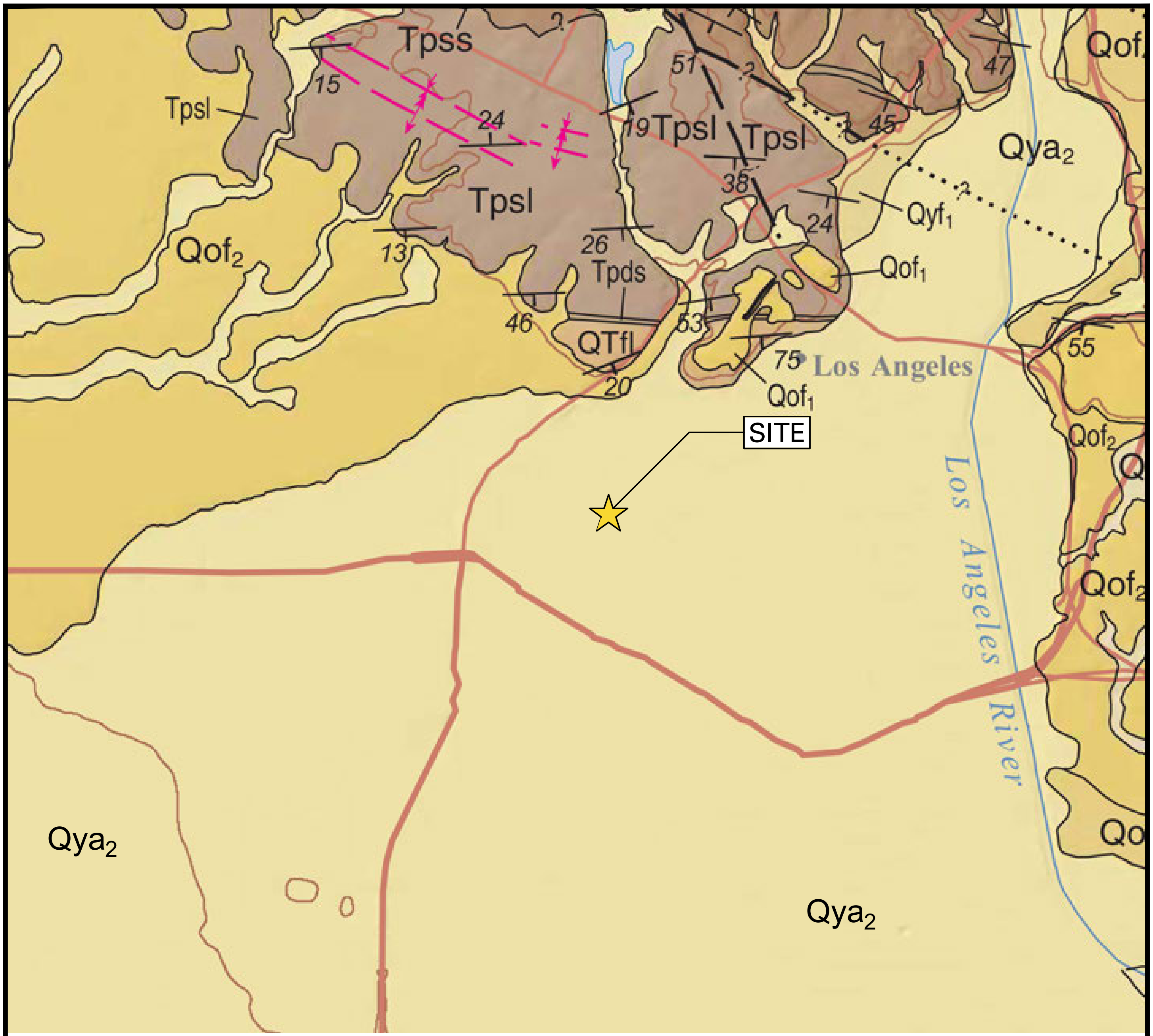
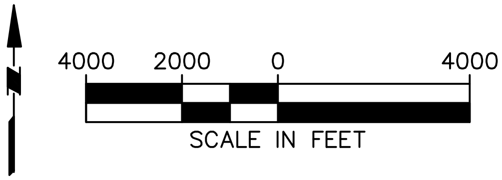


**LEGEND:**

| | |
|---|---|
| Qya | YOUNG ALLUVIUM |
| Qyf | YOUNG ALLUVIAL FAN DEPOSITS |
| Qof | OLD ALLUVIAL FAN DEPOSITS |
| QTfl | LOS ANGELES BASIN FERNANDO FORMATION |
| Tpsl | PUENTE FORMATION - SILTSTONE |
| Tpds | PUENTE FORMATION - DIATOMACEOUS SHALE |
| Tpss | PUENTE FORMATION - SANDSTONE |



REFERENCE: CGS PRELIMINARY GEOLOGIC MAP OF THE LOS ANGELES 30' x 60' QUADRANGLE, CALIFORNIA (CAMPBELL ET AL., 2014).

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Figure Title

**REGIONAL GEOLOGIC MAP**

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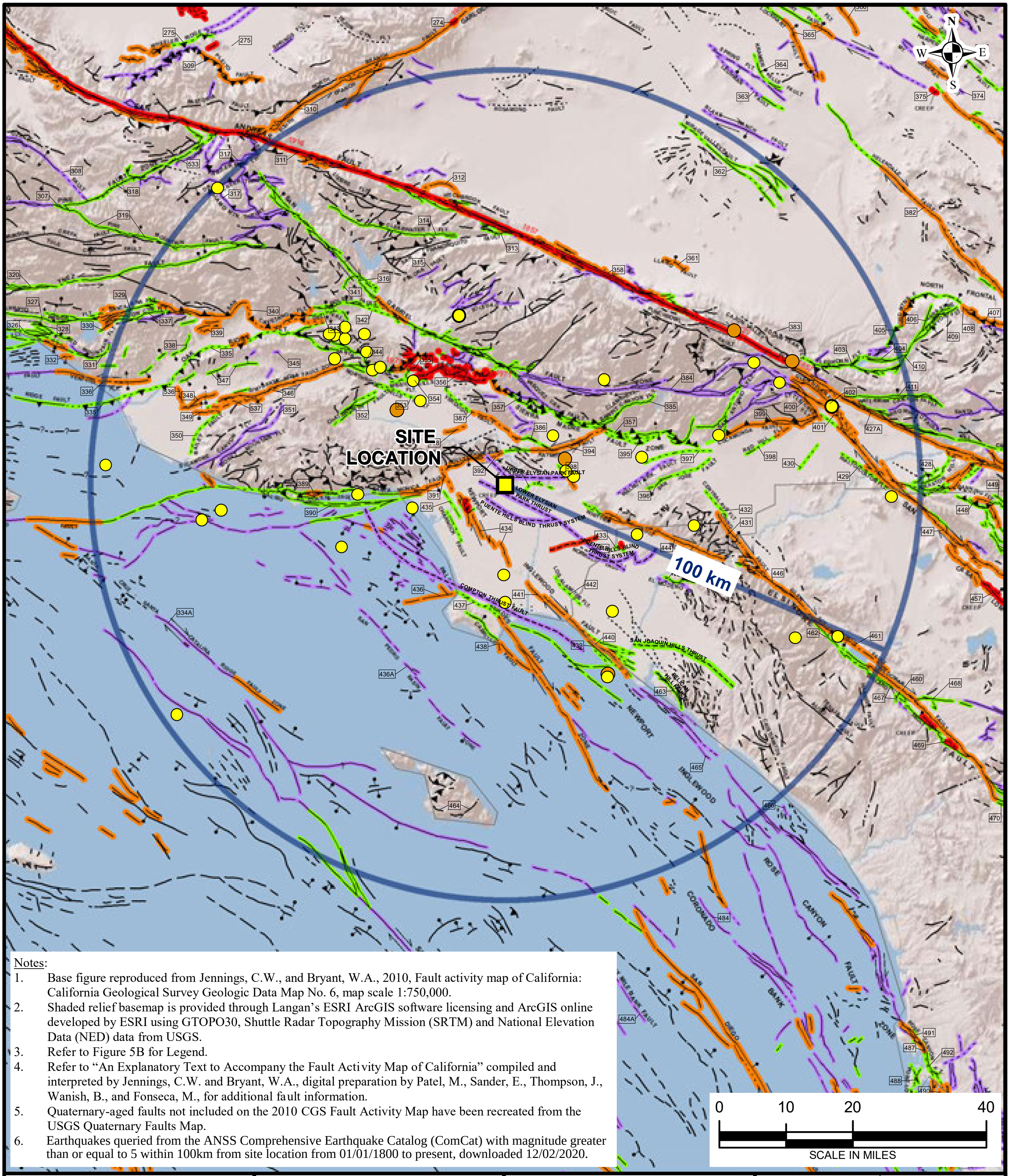
Figure No.
**7**



Notes:
1. Base figure reproduced from Jennings, C.W., and Bryant, W.A., 2010, Fault activity map of California: California Geological Survey Geologic Data Map No. 6, map scale 1:750,000.
2. Shaded relief basemap is provided through Langan's ESRI ArcGIS software licensing and ArcGIS online developed by ESRI using GTOPO30, Shuttle Radar Topography Mission (SRTM) and National Elevation Data (NED) data from USGS.
3. Refer to Figure 5B for Legend.
4. Refer to "An Explanatory Text to Accompany the Fault Activity Map of California" compiled and interpreted by Jennings, C.W. and Bryant, W.A., digital preparation by Patel, M., Sander, E., Thompson, J., Wanish, B., and Fonseca, M., for additional fault information.
5. Quaternary-aged faults not included on the 2010 CGS Fault Activity Map have been recreated from the USGS Quaternary Faults Map.
6. Earthquakes queried from the ANSS Comprehensive Earthquake Catalog (ComCat) with magnitude greater than or equal to 5 within 100km from site location from 01/01/1800 to present, downloaded 12/02/2020.

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LOS ANGELES COUNTY CALIFORNIA

Figure Title

QUATERNARY FAULT AND EARTHQUAKE EPICENTER MAP

Project No.
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Date
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Scale
1 inch = 20 miles

Drawn By
NB

Figure
8A

Path: \\Langan.com\data\IRV\data7\700070701\Project Data\ArcGIS\MXD\Geotech_Figures\2010_Fault Map IRV.mxd Date: 12/2/2020 User: NBurke Time: 7:00:03 PM

**LEGEND:**

🟨 Site Location

## Fault Age

**The age classifications are based on geologic evidence to determine the youngest faulted unit and the oldest unfaulted unit along each fault of fault seciton**

- 🟥 Historic
- 🟧 Holocene
- 🟩 Late Quaternary
- 🟪 Quaternary
- ▭ 100 km

## Earthquake Epicenter

- ● Magnitude 5 to 5.9
- ● Magnitude 6 to 6.9
- ● Magnitude 7 to 7.4
- ● Magnitude 7.5 to 8

## Pre Quaternary Faults

- —— fault, certain
- – — – fault, approx. located
- ········ fault, concealed
- —▲— thrust fault, certain
- - ▲ - thrust fault, approx. located
- ···▲··· thrust fault, approx. located, queried
- —⊥— fault, certain, barball
- ···⊥·· fault, concealed, barball
- - ⊥ - fault, approx. located, barball

## Quaternary Faults

- —— fault, certain
- — — fault, approx. located
- —?— fault, approx. located, queried
- - ? - fault, inferred, queried
- ······ fault, concealed
- - -?- - fault, concealed, queried
- —▼— thrust fault, certain
- - ▼ - thrust fault, approx. located
- ···▼··· thrust fault, concealed
- —— dextral fault, certain
- – — – dextral fault, approx. located
- ······· dextral fault, concealed
- —— sinistral fault, certain
- – — – sinistral fault, approx. located
- ······· sinistral fault, concealed
- —— thrust fault, certain (2)
- — — thrust fault, approx. located (2)
- ······ thrust fault, concealed (2)
- —⊥— fault, solid, barball
- - ⊥ - fault, dashed, barball
- ···⊥··· fault, dotted, barball
- —⊥— dextral fault, solid, barball
- - -?- - fault, dotted, queried, ballbar
- - -?- - fault, dotted, queried, ballbar (2)
- —— fault, solid, dip
- — — fault, dashed, dip
- ······ fault, dotted, dip
- —⊥— reverse fault, solid
- - ⊥ - reverse fault, dashed
- ···⊥··· reverse fault, dotted

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Figure Title
QUATERNARY FAULT AND EARTHQUAKE EPICENTER MAP

Project No. 700070701
Date DECEMBER 2020
Scale NOT TO SCALE
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Figure
8B

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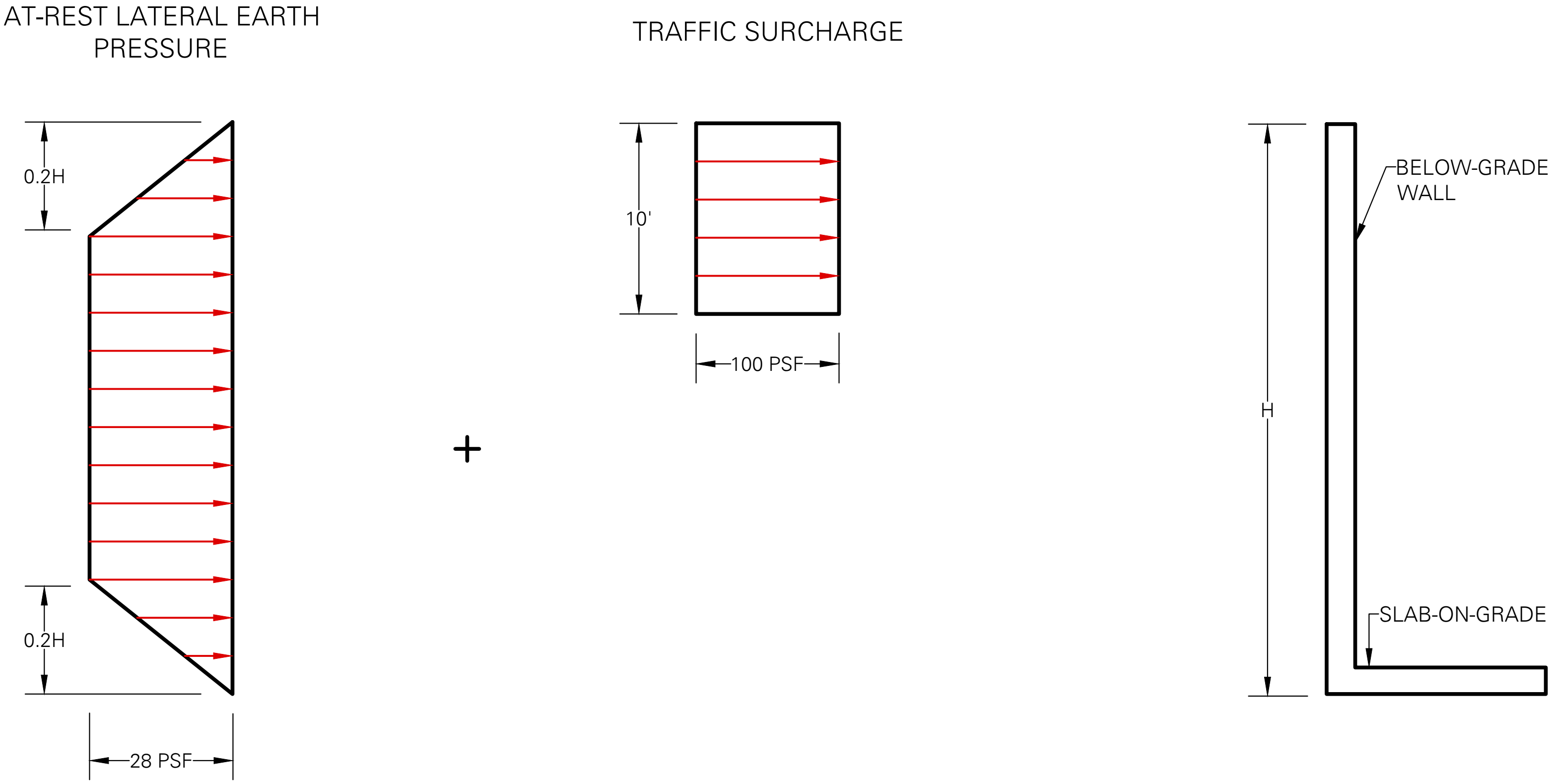

**NOT TO SCALE**

**NOTE:** IN ADDITION TO THESE LATERAL EARTH PRESSURES, SURCHARGE PRESSURES PRESENTED IN SECTION 6.2 SHOULD BE INCLUDED WHERE APPLICABLE.

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Figure Title
**AT-REST LATERAL EARTH PRESSURE**

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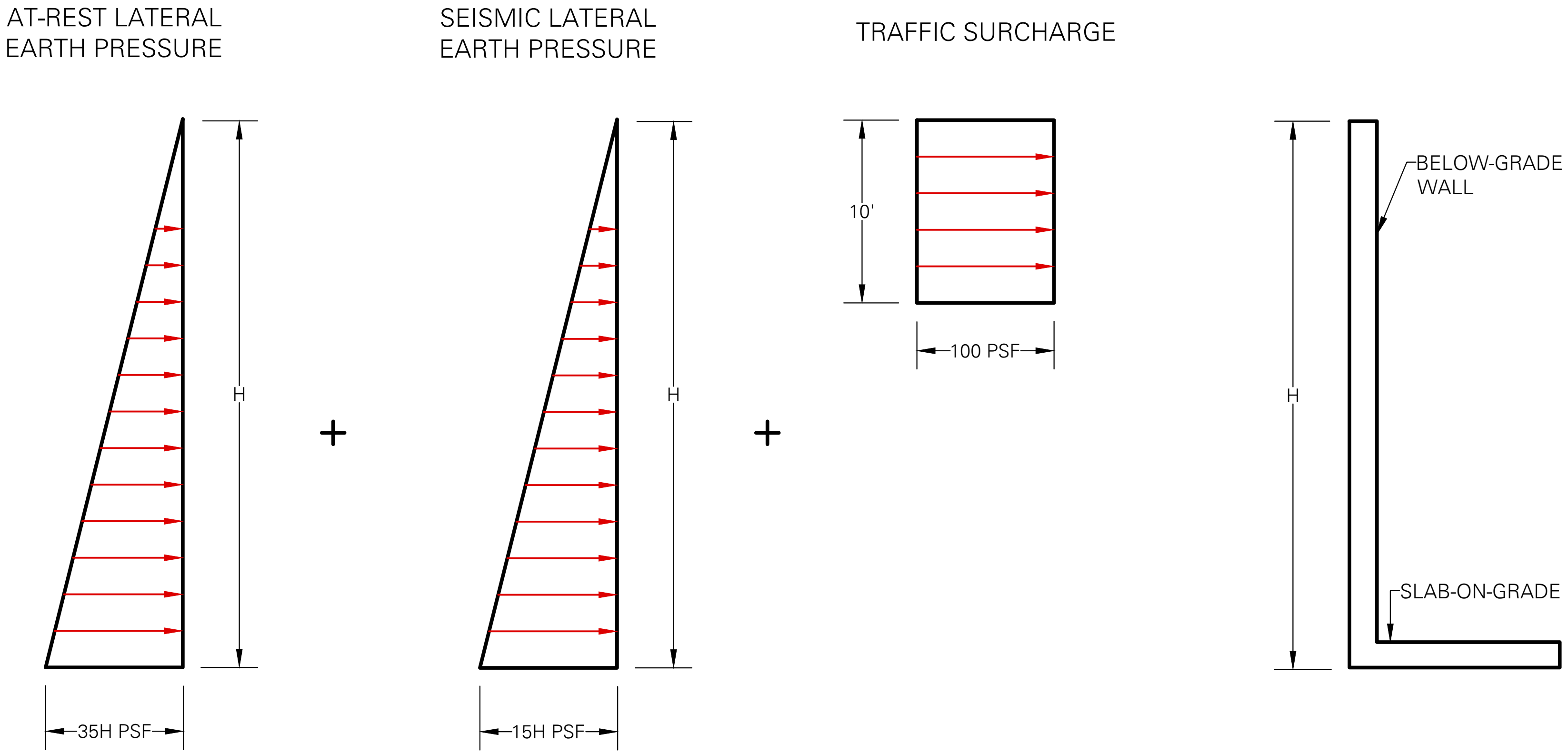
Figure No.
**9**



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**NOTE:** IN ADDITION TO THESE LATERAL EARTH PRESSURES, SURCHARGE PRESSURES PRESENTED IN SECTION 6.2 SHOULD BE INCLUDED WHERE APPLICABLE.

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Figure Title

**SEISMIC LATERAL EARTH PRESSURE**

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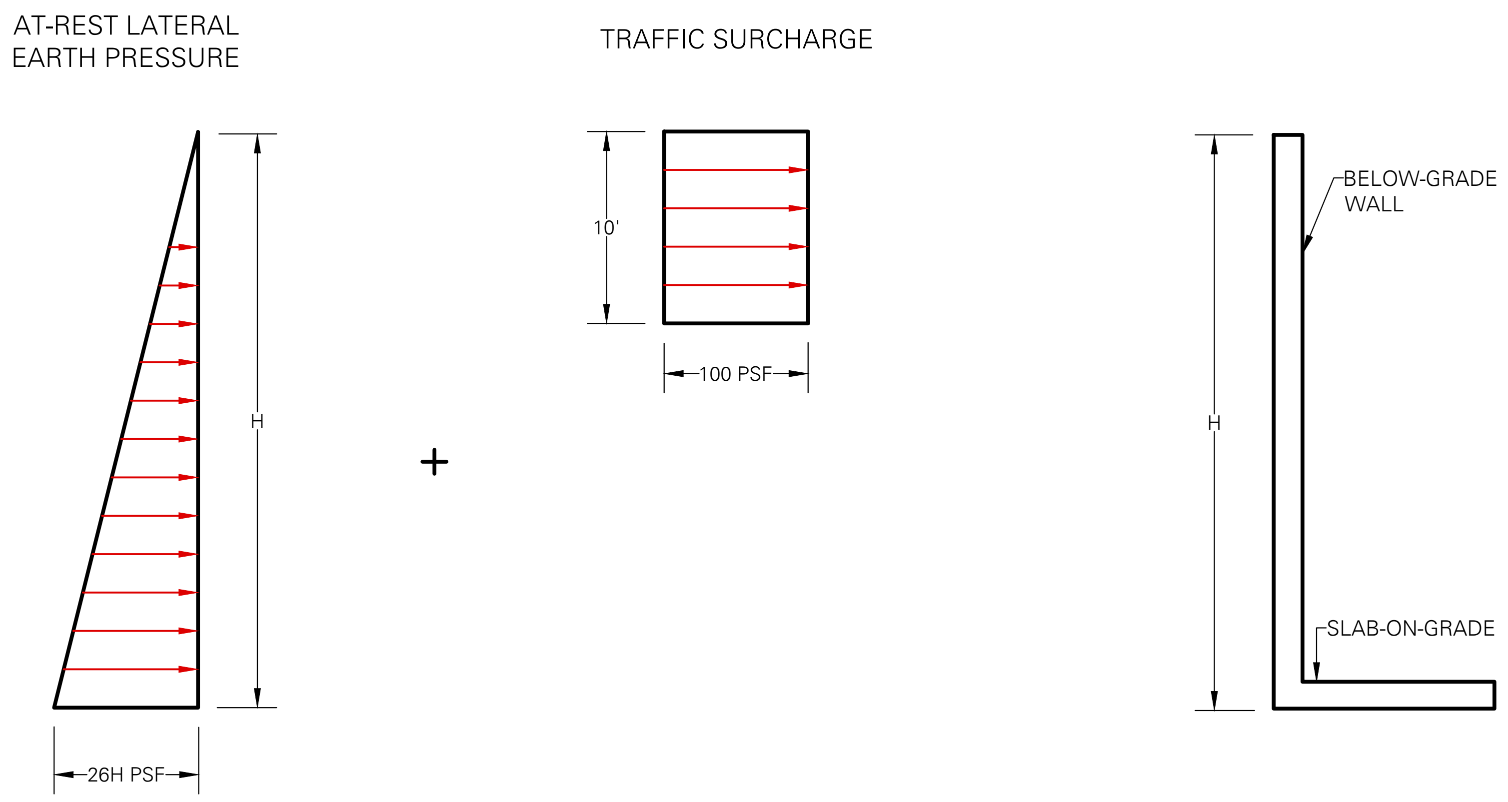
Figure No.
**10**



**NOT TO SCALE**

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**PROPOSED HIGH-RISE RESIDENTIAL TOWER**

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LOS ANGELES COUNTY CALIFORNIA

Figure Title

**LATERAL EARTH PRESSURE - CANTILEVER SHORING**

Project No.
700070701

Date
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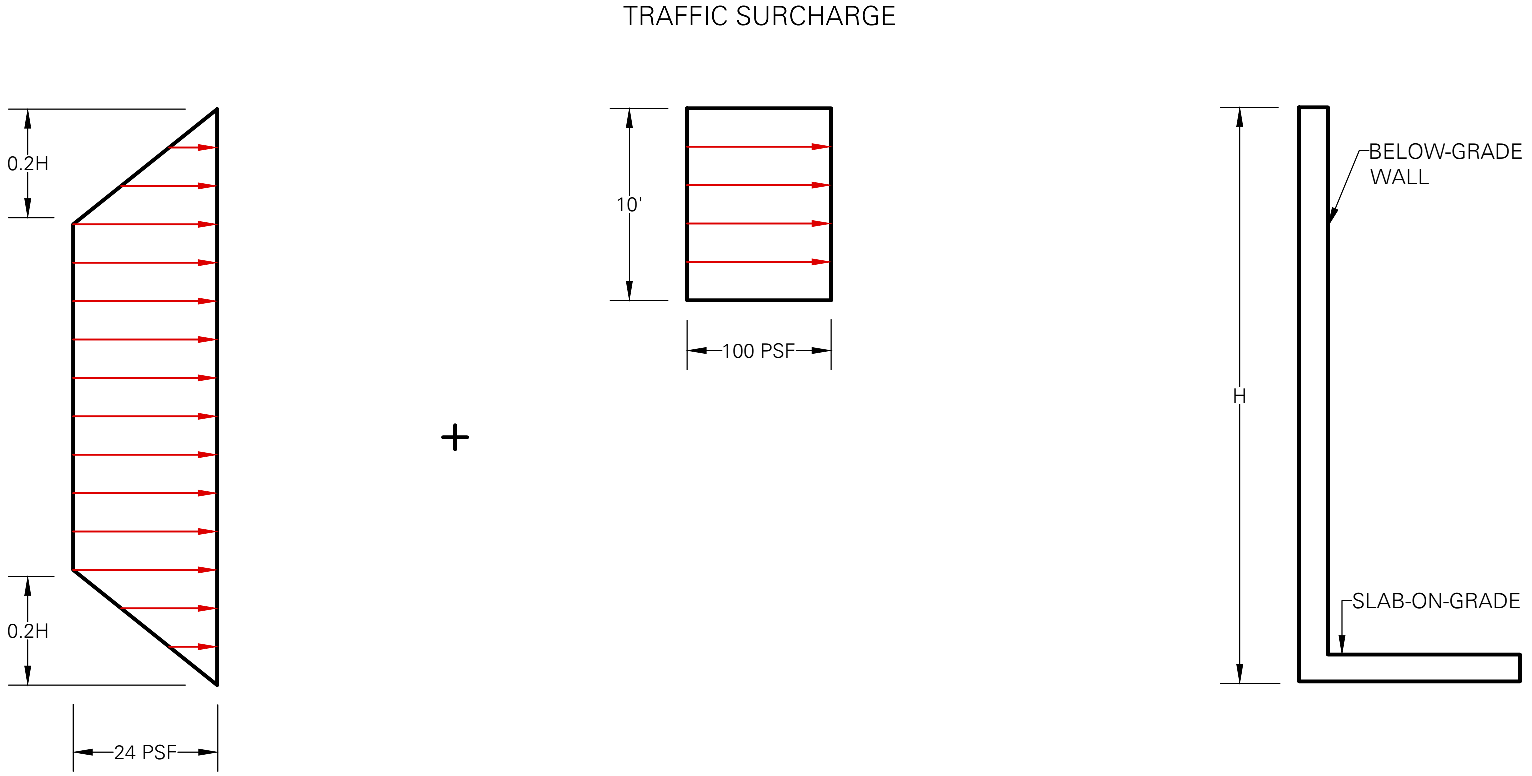
CDC

Figure No.

**11**



**NOT TO SCALE**

**NOTE:** IN ADDITION TO THESE LATERAL EARTH PRESSURES, SURCHARGE PRESSURES PRESENTED IN SECTION 6.2 SHOULD BE INCLUDED WHERE APPLICABLE.

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Figure Title

**LATERAL EARTH PRESSURE - BRACED SHORING**

Project No.
700070701

Date
DECEMBER 2020

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Figure No.

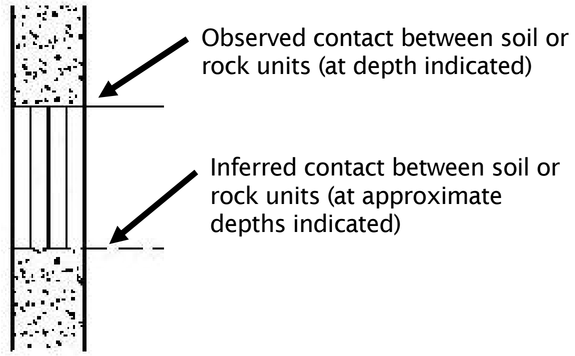
**12**

# APPENDIX A

## Prior Field Explorations and Laboratory Testing

| 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



Observed contact between soil or rock units (at depth indicated)

Inferred contact between soil or rock units (at approximate depths indicated)

## 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<br>(more than 50% retained on No. 200 sieve) | GRAVEL<br>(more than 50% of coarse fraction retained on No. 4 sieve) | CLEAN GRAVEL<br>(< 5% fines) | GW or GP | GRAVEL |
| | | GRAVEL WITH FINES<br>(≥ 5% and ≤ 12% fines) | GW-GM or GP-GM | GRAVEL with silt |
| | | | GW-GC or GP-GC | GRAVEL with clay |
| | | GRAVEL WITH FINES<br>(> 12% fines) | GM | silty GRAVEL |
| | | | GC | clayey GRAVEL |
| | | | GC-GM | silty, clayey GRAVEL |
| | SAND<br>(50% or more of coarse fraction passing No. 4 sieve) | CLEAN SAND<br>(<5% fines) | SW or SP | SAND |
| | | SAND WITH FINES<br>(≥ 5% and ≤ 12% fines) | SW-SM or SP-SM | SAND with silt |
| | | | SW-SC or SP-SC | SAND with clay |
| | | SAND WITH FINES<br>(> 12% fines) | SM | silty SAND |
| | | | SC | clayey SAND |
| | | | SC-SM | silty, clayey SAND |
| FINE-GRAINED SOIL<br>(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:** | | **Percent** | **Sand and Gravel In:** | |
| | **Fine-Grained Soil** | **Coarse-Grained Soil** | | **Fine-Grained Soil** | **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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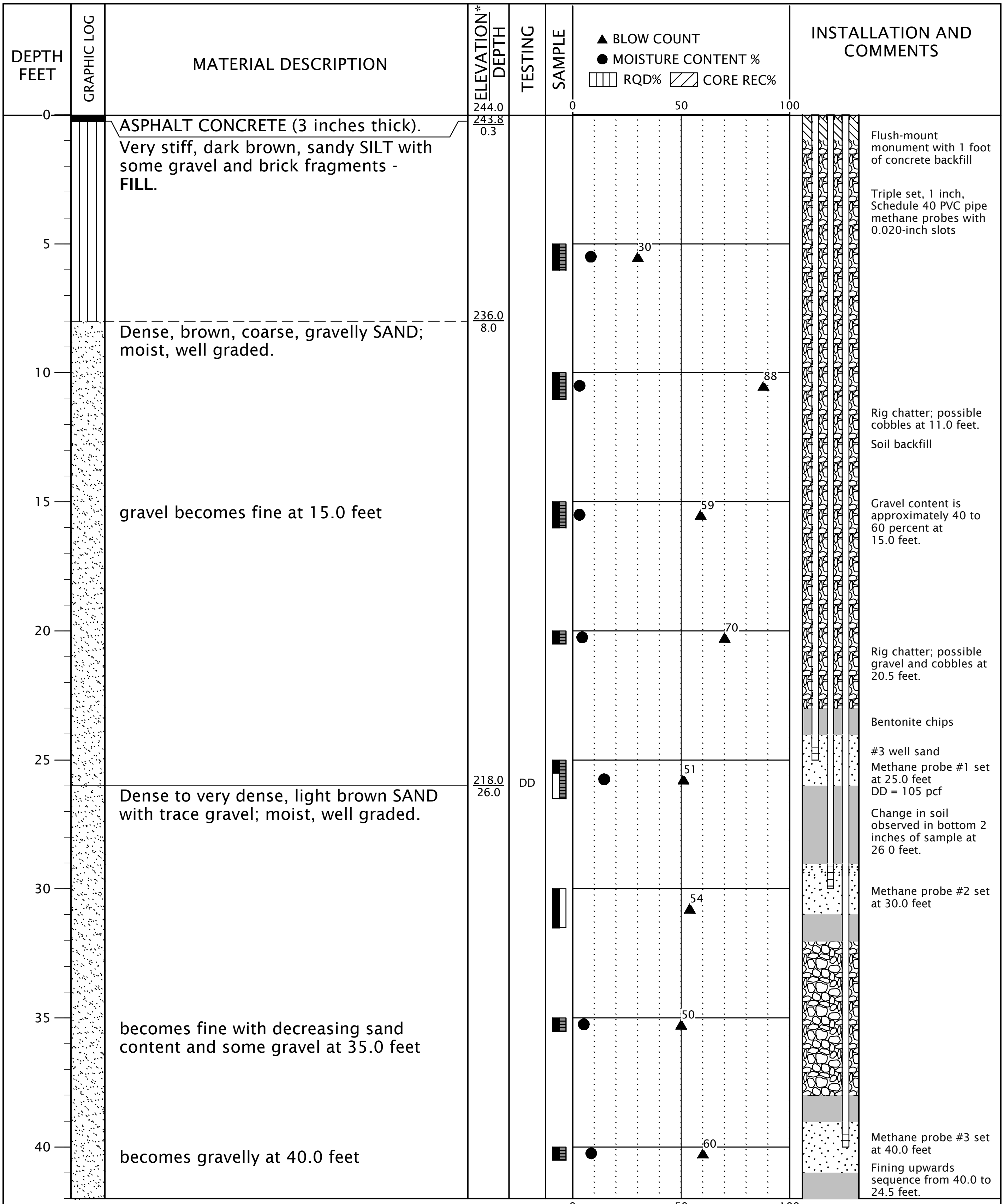
**SOIL CLASSIFICATION SYSTEM**

**TABLE A-2**



| DEPTH FEET | MATERIAL DESCRIPTION | ELEVATION* / DEPTH |
|---|---|---|
| 0 | ASPHALT CONCRETE (3 inches thick). | 244.0 |
|  | Very stiff, dark brown, sandy SILT with some gravel and brick fragments - **FILL**. | 243.8 / 0.3 |
|  | Dense, brown, coarse, gravelly SAND; moist, well graded. | 236.0 / 8.0 |
| 15 | gravel becomes fine at 15.0 feet |  |
|  | Dense to very dense, light brown SAND with trace gravel; moist, well graded. | 218.0 / 26.0 |
| 35 | becomes fine with decreasing sand content and some gravel at 35.0 feet |  |
| 40 | becomes gravelly at 40.0 feet |  |

INSTALLATION AND COMMENTS

Flush-mount monument with 1 foot of concrete backfill

Triple set, 1 inch, Schedule 40 PVC pipe methane probes with 0.020-inch slots

Rig chatter; possible cobbles at 11.0 feet.

Soil backfill

Gravel content is approximately 40 to 60 percent at 15.0 feet.

Rig chatter; possible gravel and cobbles at 20.5 feet.

Bentonite chips

#3 well sand

Methane probe #1 set at 25.0 feet
DD = 105 pcf

Change in soil observed in bottom 2 inches of sample at 26 0 feet.

Methane probe #2 set at 30.0 feet

Methane probe #3 set at 40.0 feet

Fining upwards sequence from 40.0 to 24.5 feet.

*Elevation is approximate and based on USGS datum

DRILLED BY: JDK Drilling, Inc.

LOGGED BY: DTW/MBP

COMPLETED: 04/29/07

BORING METHOD: hollow-stem auger (see report text)

BORING BIT DIAMETER: 8.0 in

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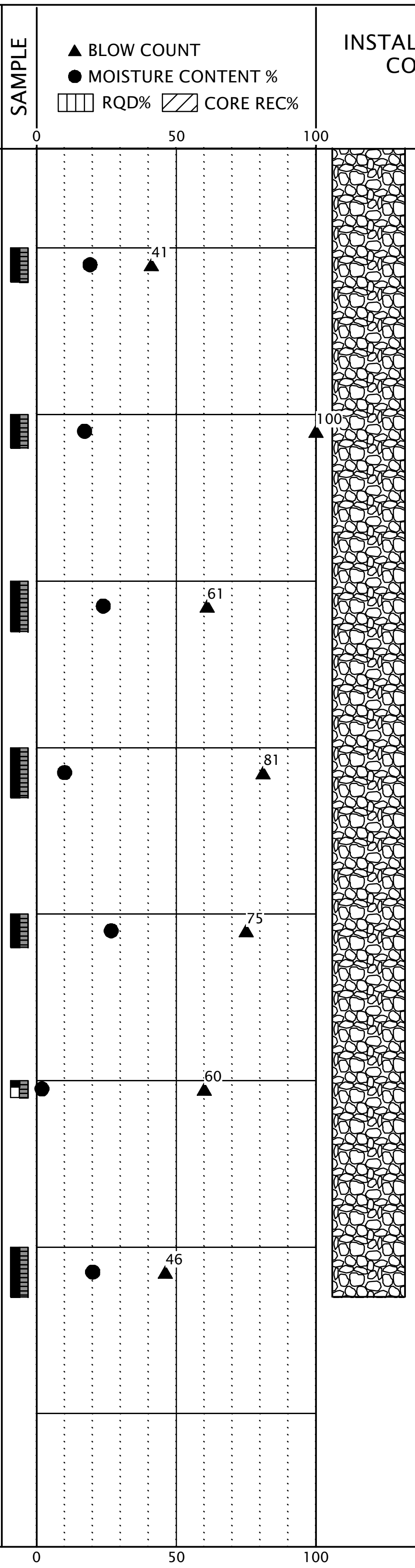
**BORING B-1**

PROPOSED 1100 SOUTH OLIVE STREET TOWER
LOS ANGELES, CA

**FIGURE A-1**

BORING - ELEVATION MERUELO-13-01.GPJ GEODESIGN.GDT PRINT DATE: 6/4/07:KYK

| DEPTH FEET | MATERIAL DESCRIPTION | ELEVATION* DEPTH | TESTING | ▲ BLOW COUNT | INSTALLATION AND COMMENTS |
|---|---|---|---|---|---|
| | (continued from previous page) | | | | |
| | Medium dense, gray-brown, silty SAND with orange mottles; moist; with some lenses of sandy silt. | 200.0<br>44.0 | | | |
| 45 | | | DD<br>DS | 41 | DD = 98 pcf |
| | Hard, gray-brown, sandy SILT with orange mottles; moist to wet; porous. | 195.0<br>49.0 | | | |
| 50 | | | DD<br>CON | 100 | DD = 109 pcf |
| 55 | with lenses of clay with some silt at 55.0 feet | | DD | 61 | DD = 101 pcf |
| | Dense, brown, silty SAND with some clay; moist. | 186.0<br>58.0 | | | |
| 60 | | | DD<br>DS | 81 | DD = 119 pcf |
| | Dense, yellow-brown, fine SAND; moist, poorly graded, homogenous. | 181.5<br>62.5 | | | |
| 65 | | | DD | 75 | DD = 95 pcf<br>Rig chatter from 65.5 to 68.0 feet. |
| 70 | | | | 60 | Disturbed sample; possible trace gravel at 70.0 feet. |
| 75 | with some silt and trace clay at 75.0 feet | | DD<br>DS | 46 | DD = 109 pcf |
| | | 167.5<br>76.5 | | | All samplers driven with 140-lb. hammer falling 30 inches. |
| 80 | | | | | |



**Exploration completed at 76.5 feet.**

**Groundwater not encountered.**

**Methane probes installed at depths of 25.0, 30.0, and 40.0 feet BGS upon completion of drilling.**

**Borehole backfilled with cuttings.**

*Elevation is approximate and based on USGS datum

DRILLED BY: JDK Drilling, Inc.    LOGGED BY: DTW/MBP    COMPLETED: 04/29/07

BORING METHOD: hollow-stem auger (see report text)    BORING BIT DIAMETER: 8.0 in

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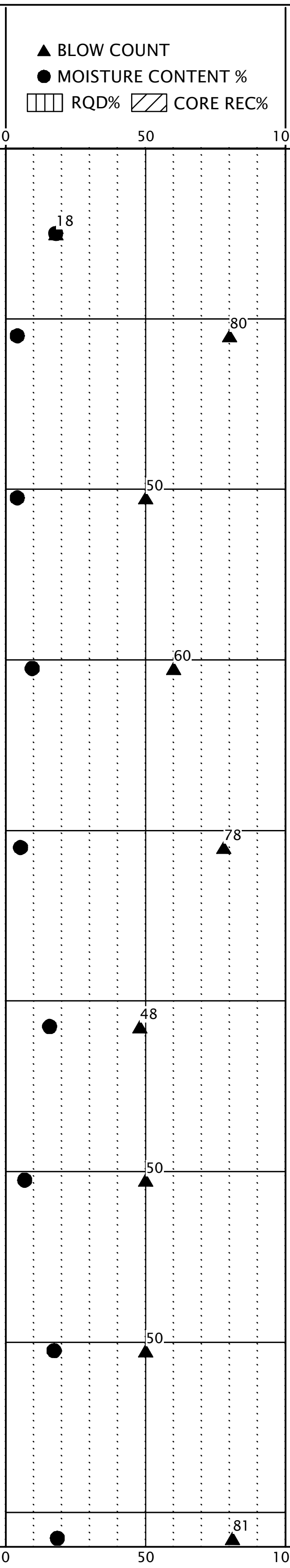
**BORING B-1**
(continued)

PROPOSED 1100 SOUTH OLIVE STREET TOWER
LOS ANGELES, CA

**FIGURE A-1**

BORING - ELEVATION MERUELO-13-01.GPJ GEODESIGN.GDT PRINT DATE: 6/4/07:KYK

# BORING B-2



| DEPTH FEET | MATERIAL DESCRIPTION | ELEVATION* DEPTH | TESTING | BLOW COUNT | INSTALLATION AND COMMENTS |
|---|---|---|---|---|---|
| 0 | ASPHALT CONCRETE (3 inches thick). | 244.0<br>243.8<br>0.3 | | | All samplers driven with 140-lb. hammer falling 30 inches. |
| | Stiff, dark brown, sandy SILT with trace clay; moist - **FILL**. | | DD | 18 | DD = 109 pcf |
| | Dense, light brown, silty SAND with some gravel; moist. | 240.0<br>4.0 | | | |
| 5 | | | | 80 | |
| | Dense, brown, coarse, gravelly SAND; moist, well graded. | 235.0<br>9.0 | | | |
| 10 | | | | 50 | Rig chatter; possible cobbles from 9.0 to 20.0 feet. |
| 15 | | | DD | 60 | DD = 105 pcf |
| | Dense, brown, medium SAND; moist, poorly graded. | 226.0<br>18.0 | | | |
| 20 | | | DD | 78 | DD = 106 pcf |
| | Dense, brown, medium to coarse, gravelly SAND; moist, well graded. | 223.5<br>20.5 | | | |
| | Very stiff, brown, sandy SILT with trace gravel; moist. | 222.0<br>22.0 | | | |
| 25 | | | DD<br>CON<br>DS | 48 | DD = 109 pcf |
| | Dense to very dense, light brown, coarse SAND with some gravel; moist. | 216.0<br>28.0 | | | |
| 30 | | | DD | 50 | DD = 113 pcf |
| 35 | with increasing gravel content at 35.0 feet | | DD | 50 | DD = 111 pcf |
| | Dense, brown, fine, silty SAND; moist. | 205.0<br>39.0 | | | |
| 40 | | | DD | 81 | DD = 108 pcf |

*Elevation is approximate and based on USGS datum

DRILLED BY: JDK Drilling, Inc.

LOGGED BY: JAA/MBP

COMPLETED: 04/24/07

BORING METHOD: hollow-stem auger (see report text)

BORING BIT DIAMETER: 8.0 in

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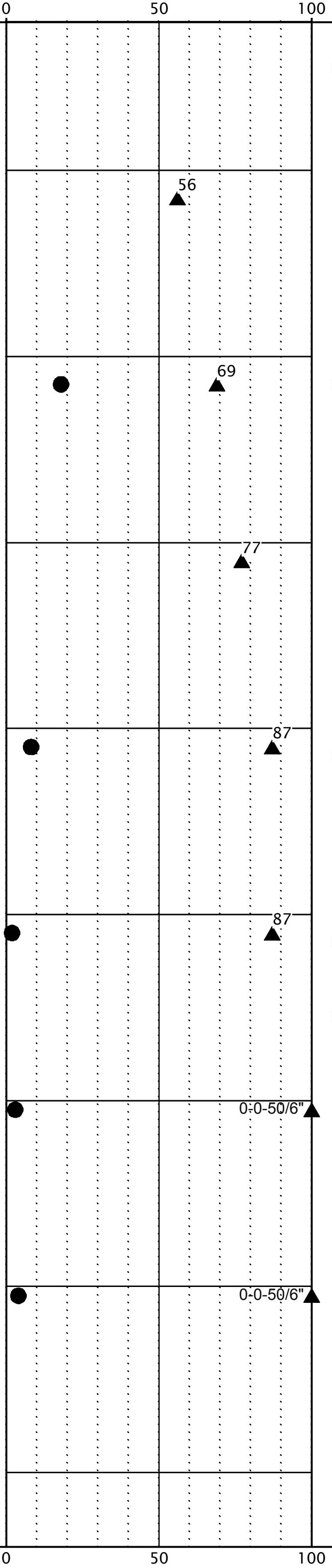
BORING B-2

PROPOSED 1100 SOUTH OLIVE STREET TOWER
LOS ANGELES, CA

FIGURE A-2

BORING - ELEVATION MERUELO-13-01.GPJ GEODESIGN.GDT PRINT DATE: 6/4/07:KYK

DEPTH FEET | GRAPHIC LOG | MATERIAL DESCRIPTION | ELEVATION* / DEPTH | TESTING | SAMPLE | ▲ BLOW COUNT ● MOISTURE CONTENT % ▥ RQD% ▨ CORE REC% | INSTALLATION AND COMMENTS



| DEPTH FEET | MATERIAL DESCRIPTION | ELEVATION* / DEPTH | TESTING | BLOW COUNT | INSTALLATION AND COMMENTS |
|---|---|---|---|---|---|
| | (continued from previous page) | | DS | | Rig chatter at 41.5 feet. |
| 45 | | | | 56 | |
| 50 | | | DD<br>DS | 69 | DD = 105 pcf |
| 55 | | | | 77 | |
| | Dense to very dense, brown, fine SAND; moist, poorly graded. | 186.0<br>58.0 | | | |
| 60 | | | DD<br>DS | 87 | DD = 94 pcf |
| | Very dense, light brown, SAND with trace to some gravel; moist, poorly graded. | 181.0<br>63.0 | | | |
| 65 | | | DD | 87 | DD = 96 pcf |
| | | | | | Rig chatter at 67.0 feet. |
| 70 | | | | 0-0-50/6" | |
| | grades to light gray with trace gravel at 70.5 feet | | | | |
| | with some gravel; well graded at 73.5 feet | | | | |
| 75 | | | | 0-0-50/6" | |
| | **Exploration completed at 75.5 feet.**<br>**Groundwater not encountered.**<br>**Borehole backfilled with cuttings and patched with asphalt concrete.** | 168.5<br>75.5 | | | |
| 80 | | | | | |

*Elevation is approximate and based on USGS datum

DRILLED BY: JDK Drilling, Inc.  LOGGED BY: JAA/MBP  COMPLETED: 04/24/07

BORING METHOD: hollow-stem auger (see report text)  BORING BIT DIAMETER: 8.0 in

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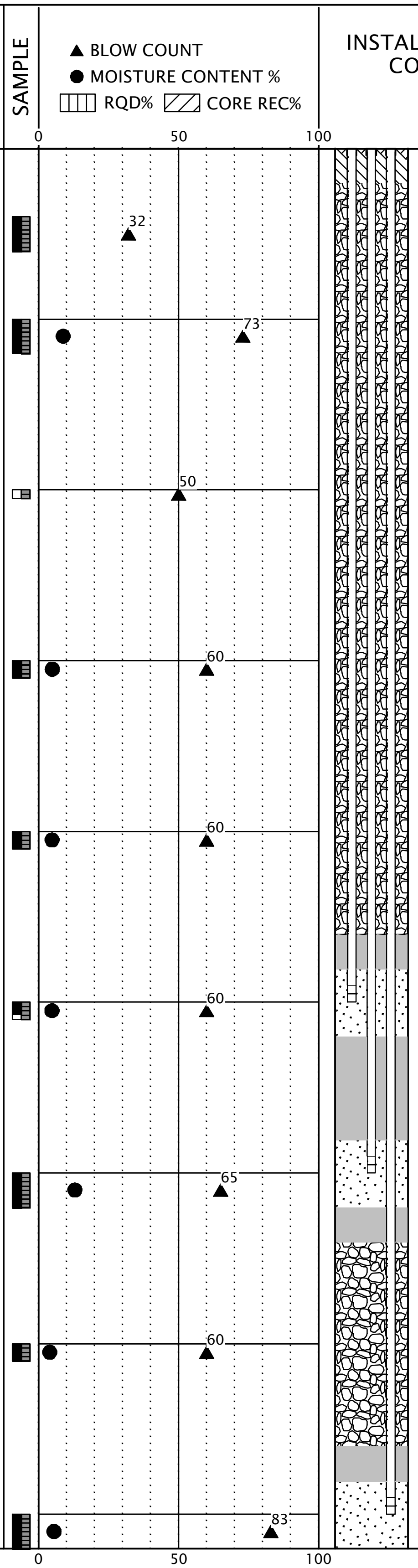
**BORING B-2**
(continued)

PROPOSED 1100 SOUTH OLIVE STREET TOWER
LOS ANGELES, CA

**FIGURE A-2**

| DEPTH FEET | ELEVATION* / DEPTH | MATERIAL DESCRIPTION | TESTING | BLOW COUNT | INSTALLATION AND COMMENTS |
|---|---|---|---|---|---|
| 0 | 244.0 | | | | |
| | 243.8<br>0.3 | ASPHALT CONCRETE (3 inches thick). | | | |
| | | Medium dense, brown, silty SAND with some gravel and brick fragments; moist - **FILL**. | | 32 | Flush-mount monument with 1 foot of concrete backfill<br>Triple set, 1 inch, Schedule 40 PVC pipe methane probes with 0.020-inch slots |
| | 240.0<br>4.0 | Dense, brown, silty SAND with trace to some gravel; moist. | | | |
| 5 | | | DD | 73 | DD = 118 pcf |
| | | possible cobbles from 8.0 to 25.0 feet | | | |
| | 235.0<br>9.0 | Very dense, brown, coarse, gravelly SAND; moist, well graded. | | | |
| 10 | | | | 50 | No recovery at 10.0 feet.<br>Rig bouncing at 11.5 feet. |
| 15 | | | DD | 60 | Soil backfill<br>DD = 108 pcf<br>Rig bouncing at 16.5 feet. |
| 20 | | with increasing gravel content at 20.0 feet | | 60 | |
| | | | | | Bentonite chips<br>#3 well sand |
| 25 | | | | 60 | Methane probe #1 set at 25.0 feet |
| | 216.0<br>28.0 | Medium dense to dense, brown, fine to medium SAND; moist, poorly graded, homogenous. | | | |
| 30 | | | DD<br>CON<br>DS | 65 | Methane probe #2 set at 30.0 feet<br>DD = 98 pcf |
| | 211.0<br>33.0 | Dense to very dense, brown, gravelly SAND; moist, well graded. | | | |
| 35 | | | | 60 | |
| 40 | | becomes dense with trace fine gravel at | DD | 83 | Methane probe #3 set at 40.0 feet |



*Elevation is approximate and based on USGS datum

DRILLED BY: JDK Drilling, Inc.

LOGGED BY: DTW/MBP

COMPLETED: 04/24/07

BORING METHOD: hollow-stem auger (see report text)

BORING BIT DIAMETER: 8.0 in

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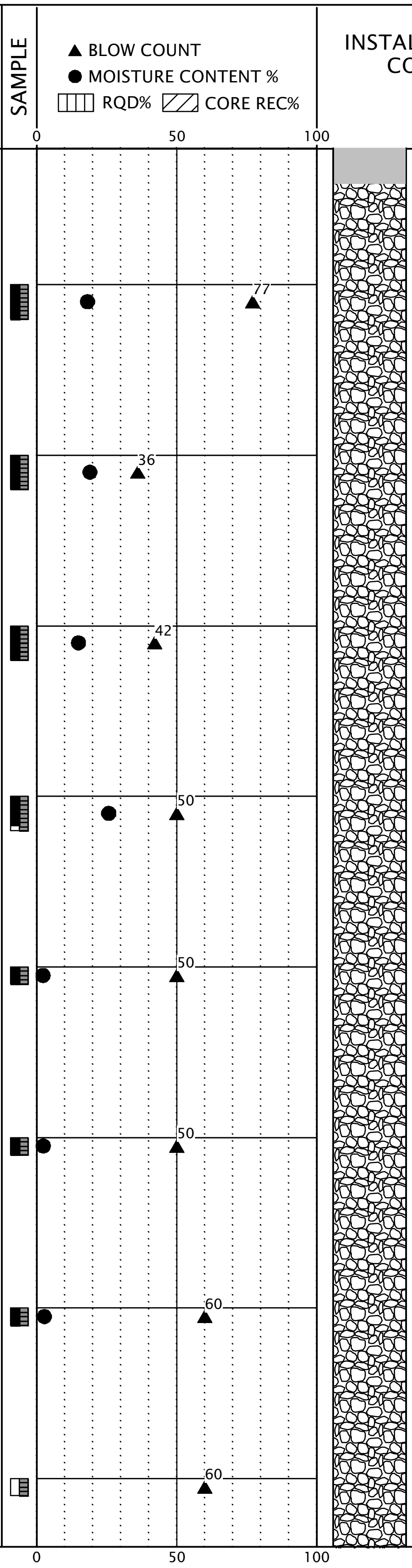
**BORING B-3**

PROPOSED 1100 SOUTH OLIVE STREET TOWER
LOS ANGELES, CA

**FIGURE A-3**

| DEPTH FEET | MATERIAL DESCRIPTION | ELEVATION* / DEPTH | TESTING | INSTALLATION AND COMMENTS |
|---|---|---|---|---|
|  | 40.0 feet |  |  | DD = 108 pcf |
|  | Hard, dark brown, clayey SILT with trace sand; moist. | 200.5 / 43.5 |  |  |
| 45 |  |  | DD<br>CON | DD = 110 pcf |
| 50 | becomes very stiff with gray mottles at 50.0 feet |  | DD<br>DS | DD = 101 pcf |
| 55 | with lenses of silty sand at 55.0 feet |  | DD<br>DS | DD = 107 pcf |
|  | Medium dense to dense, brown, sandy SILT; moist; with lenses of clayey silt. | 185.5 / 58.5 |  |  |
| 60 |  |  | DD | DD = 99 pcf |
|  | Dense to very dense, light brown SAND with trace gravel; moist, poorly graded. | 182.0 / 62.0 |  |  |
| 65 |  |  |  |  |
|  | Dense to very dense, light brown, gravelly SAND, moist, well graded. | 175.0 / 69.0 |  |  |
| 70 |  |  |  |  |
|  |  |  |  | Water added to facilitate drilling at 72.0 feet. |
| 75 | with increasing gravel content at 75.0 feet |  |  |  |
| 80 |  |  |  | Rig chatter at 79.0 feet.<br>No recovery at 80.0 feet. |



*Elevation is approximate and based on USGS datum

DRILLED BY: JDK Drilling, Inc.

LOGGED BY: DTW/MBP

COMPLETED: 04/24/07

BORING METHOD: hollow-stem auger (see report text)

BORING BIT DIAMETER: 8.0 in

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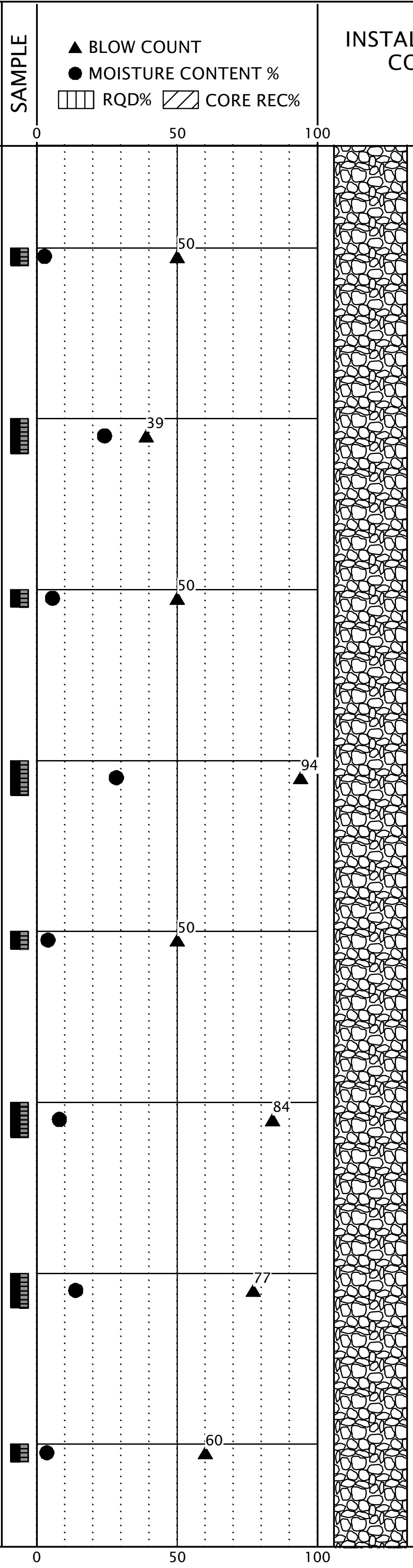
**BORING B-3**
(continued)

PROPOSED 1100 SOUTH OLIVE STREET TOWER
LOS ANGELES, CA

**FIGURE A-3**

| DEPTH FEET | MATERIAL DESCRIPTION | ELEVATION* / DEPTH | TESTING | ▲ BLOW COUNT | INSTALLATION AND COMMENTS |
|---|---|---|---|---|---|
| | (continued from previous page) | | | | Rig chatter at 82.0 feet. |
| 85 | | | | 50 | Water added to facilitate drilling at 84.5 feet. |
| | with decreasing gravel content at 84.5 feet | 158.0 / 86.0 | | | |
| | Very stiff, brown, sandy SILT; moist. | | | | |
| 90 | | | DD<br>DS | 39 | DD = 92 pcf |
| | Very dense, orange and brown, fine to medium SAND with gray mottles; moist, poorly graded. | 151.0 / 93.0 | | | |
| 95 | | | DD | 50 | DD = 91 pcf |
| 100 | with 6-inch layer of brown silty sand at 100.0 feet | | DD<br>DS | 94 | DD = 89 pcf<br>Water added to facilitate drilling at 101.0 feet.<br>More water added to facilitate drilling at 102.0 feet. |
| 105 | grades to gray and light brown at 105.0 feet | | | 50 | |
| 110 | becomes dense at 110.0 feet | | DD | 84 | DD = 90 pcf |
| 115 | grades to orange and brown at 115.0 feet | | DD | 77 | DD = 92 pcf |
| 120 | | | | 60 | |



*Elevation is approximate and based on USGS datum

DRILLED BY: JDK Drilling, Inc. LOGGED BY: DTW/MBP COMPLETED: 04/24/07

BORING METHOD: hollow-stem auger (see report text) BORING BIT DIAMETER: 8.0 in

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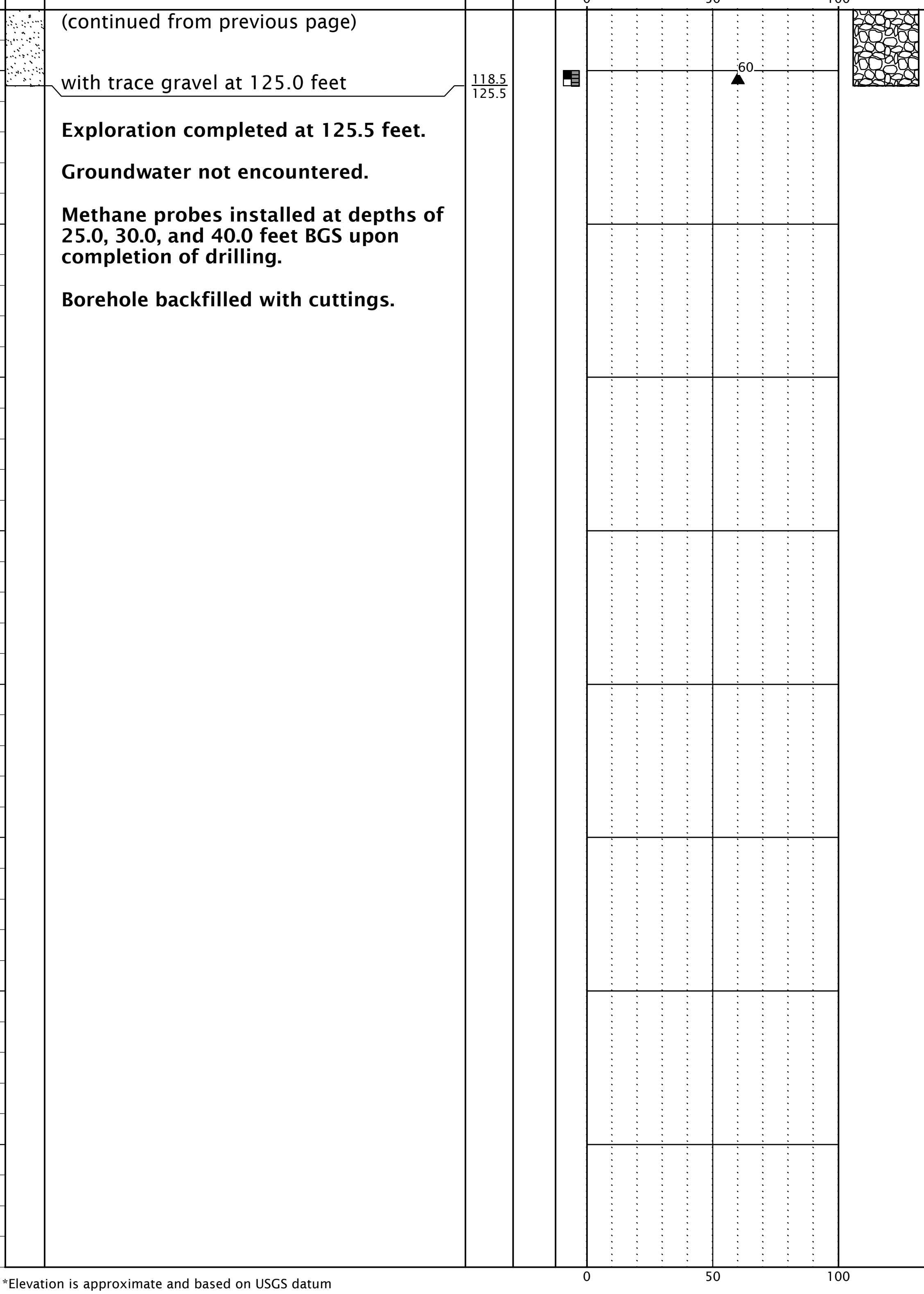
**BORING B-3**
(continued)

PROPOSED 1100 SOUTH OLIVE STREET TOWER
LOS ANGELES, CA

**FIGURE A-3**

BORING - ELEVATION MERUELO-13-01.GPJ GEODESIGN.GDT PRINT DATE: 6/4/07:KYK

| DEPTH FEET | GRAPHIC LOG | MATERIAL DESCRIPTION | ELEVATION* / DEPTH | TESTING | SAMPLE | ▲ BLOW COUNT<br>● MOISTURE CONTENT %<br>▥ RQD% ▨ CORE REC% | INSTALLATION AND COMMENTS |
|---|---|---|---|---|---|---|---|
| | | (continued from previous page) | | | | | Water added to facilitate drilling at 123.0 feet. |
| 125 | | with trace gravel at 125.0 feet | 118.5<br>125.5 | | | 60 | All samplers driven with 140-lb. hammer falling 30 inches. |
| | | **Exploration completed at 125.5 feet.** | | | | | |
| | | **Groundwater not encountered.** | | | | | |
| 130 | | **Methane probes installed at depths of 25.0, 30.0, and 40.0 feet BGS upon completion of drilling.** | | | | | |
| | | **Borehole backfilled with cuttings.** | | | | | |
| 135 | | | | | | | |
| 140 | | | | | | | |
| 145 | | | | | | | |
| 150 | | | | | | | |
| 155 | | | | | | | |
| 160 | | | | | | | |



*Elevation is approximate and based on USGS datum

DRILLED BY: JDK Drilling, Inc.
LOGGED BY: DTW/MBP
COMPLETED: 04/24/07

BORING METHOD: hollow-stem auger (see report text)
BORING BIT DIAMETER: 8.0 in

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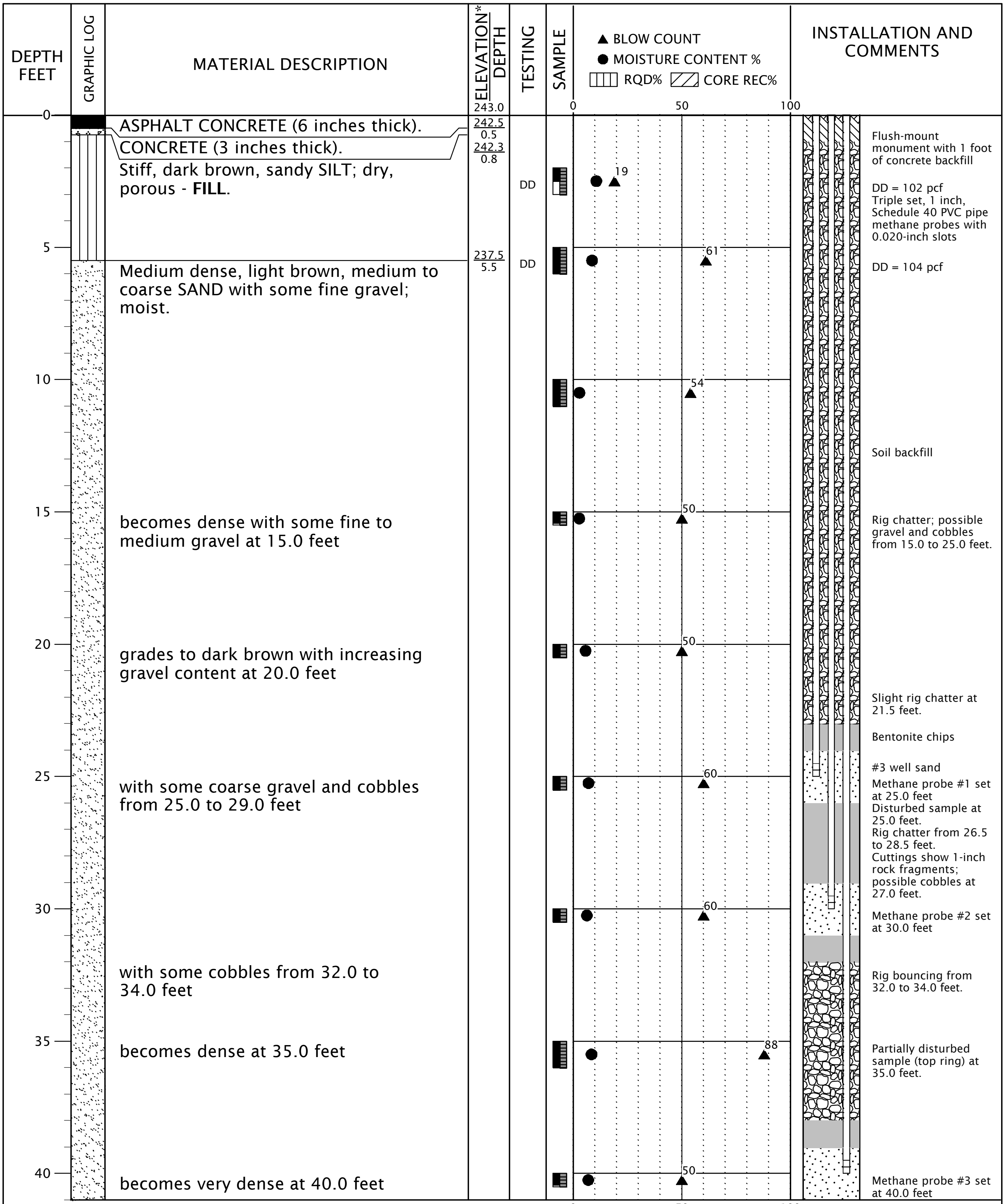
**BORING B-3**
(continued)

PROPOSED 1100 SOUTH OLIVE STREET TOWER
LOS ANGELES, CA

**FIGURE A-3**



| DEPTH FEET | MATERIAL DESCRIPTION | ELEVATION* DEPTH | TESTING | BLOW COUNT | INSTALLATION AND COMMENTS |
|---|---|---|---|---|---|
| 0 | ASPHALT CONCRETE (6 inches thick). | 243.0<br>242.5<br>0.5 | | | Flush-mount monument with 1 foot of concrete backfill |
| | CONCRETE (3 inches thick). | 242.3<br>0.8 | | | |
| | Stiff, dark brown, sandy SILT; dry, porous - **FILL**. | | DD | 19 | DD = 102 pcf<br>Triple set, 1 inch, Schedule 40 PVC pipe methane probes with 0.020-inch slots |
| 5 | Medium dense, light brown, medium to coarse SAND with some fine gravel; moist. | 237.5<br>5.5 | DD | 61 | DD = 104 pcf |
| 10 | | | | 54 | Soil backfill |
| 15 | becomes dense with some fine to medium gravel at 15.0 feet | | | 50 | Rig chatter; possible gravel and cobbles from 15.0 to 25.0 feet. |
| 20 | grades to dark brown with increasing gravel content at 20.0 feet | | | 50 | Slight rig chatter at 21.5 feet.<br>Bentonite chips |
| 25 | with some coarse gravel and cobbles from 25.0 to 29.0 feet | | | 60 | #3 well sand<br>Methane probe #1 set at 25.0 feet<br>Disturbed sample at 25.0 feet.<br>Rig chatter from 26.5 to 28.5 feet.<br>Cuttings show 1-inch rock fragments; possible cobbles at 27.0 feet. |
| 30 | | | | 60 | Methane probe #2 set at 30.0 feet |
| | with some cobbles from 32.0 to 34.0 feet | | | | Rig bouncing from 32.0 to 34.0 feet. |
| 35 | becomes dense at 35.0 feet | | | 88 | Partially disturbed sample (top ring) at 35.0 feet. |
| 40 | becomes very dense at 40.0 feet | | | 50 | Methane probe #3 set at 40.0 feet |

*Elevation is approximate and based on USGS datum

DRILLED BY: JDK Drilling, Inc.
LOGGED BY: DTW/MBP
COMPLETED: 04/26/07

BORING METHOD: hollow-stem auger (see report text)
BORING BIT DIAMETER: 8.0 in

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**BORING B-4**

PROPOSED 1100 SOUTH OLIVE STREET TOWER
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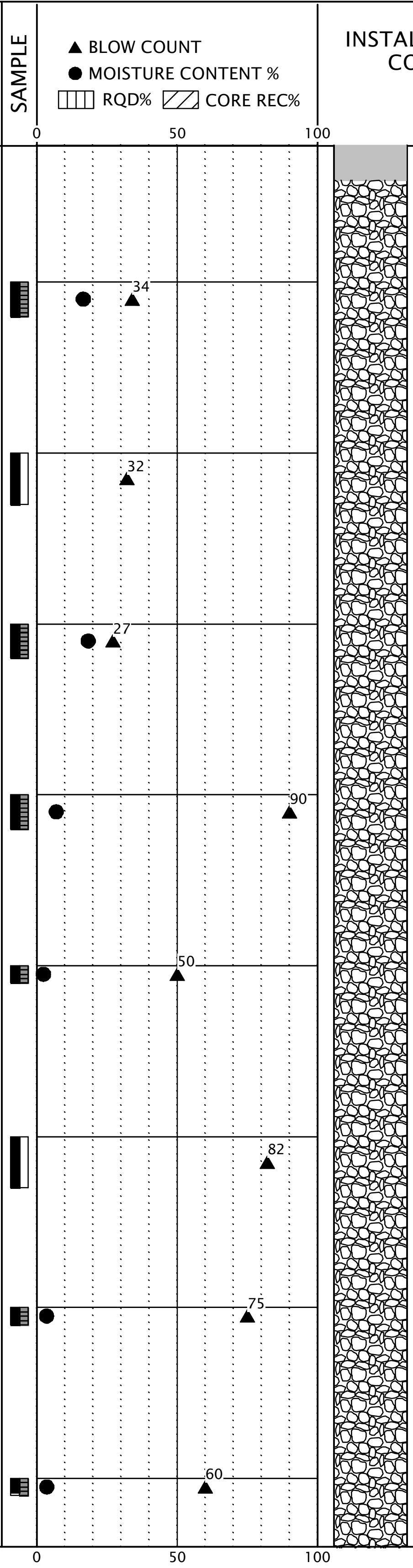
**FIGURE A-4**

BORING - ELEVATION MERUELO-13-01.GPJ GEODESIGN.GDT PRINT DATE: 6/4/07:KYK

| DEPTH FEET | MATERIAL DESCRIPTION | ELEVATION* / DEPTH | TESTING | ▲ BLOW COUNT | INSTALLATION AND COMMENTS |
|---|---|---|---|---|---|
|  | (continued from previous page) |  |  |  | Harder drilling; possible gravel and cobbles at 42.0 feet. |
|  | Very stiff, brown, sandy SILT; moist. | 199.0<br>44.0 |  |  |  |
| 45 |  |  | DD<br>DS | 34 | DD = 104 pcf |
| 50 | with trace clay at 50.0 feet |  |  | 32 |  |
|  | Medium dense, brown, silty SAND; moist. | 189.5<br>53.5 |  |  |  |
| 55 |  |  | DD<br>DS | 27 | DD = 105 pcf |
|  | Dense, red-brown, fine SAND; moist, poorly graded, homogenous. | 185.0<br>58.0 |  |  |  |
| 60 |  |  | DD | 90 | DD = 94 pcf |
| 65 | becomes very dense and medium at 65.0 feet |  | DD | 50 | DD = 96 pcf<br>Water added to facilitate drilling at 66.5 feet. |
| 70 |  |  |  | 82 |  |
|  | Very dense, brown SAND; moist, well graded. | 170.0<br>73.0 |  |  |  |
| 75 |  |  |  | 75 | Rig chatter; water added to facilitate drilling at 76.0 feet. |
| 80 |  |  | DD | 60 | Water added to facilitate drilling at 79.0 feet.<br>DD = 100 pcf |



*Elevation is approximate and based on USGS datum

DRILLED BY: JDK Drilling, Inc.
LOGGED BY: DTW/MBP
COMPLETED: 04/26/07

BORING METHOD: hollow-stem auger (see report text)
BORING BIT DIAMETER: 8.0 in

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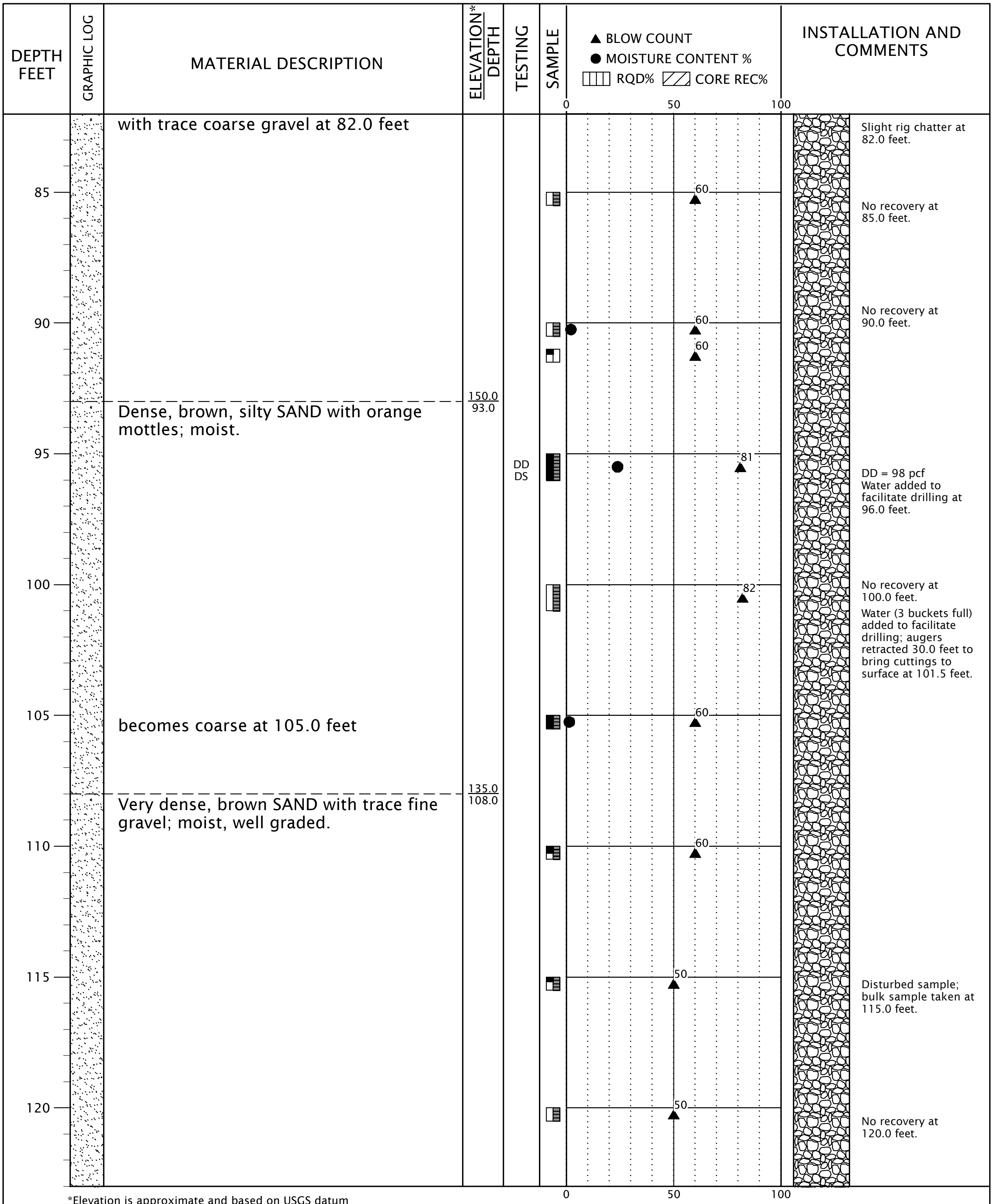
**BORING B-4**
(continued)

PROPOSED 1100 SOUTH OLIVE STREET TOWER
LOS ANGELES, CA

**FIGURE A-4**



with trace coarse gravel at 82.0 feet

Dense, brown, silty SAND with orange mottles; moist.

becomes coarse at 105.0 feet

Very dense, brown SAND with trace fine gravel; moist, well graded.

Slight rig chatter at 82.0 feet.

No recovery at 85.0 feet.

No recovery at 90.0 feet.

DD = 98 pcf
Water added to facilitate drilling at 96.0 feet.

No recovery at 100.0 feet.

Water (3 buckets full) added to facilitate drilling; augers retracted 30.0 feet to bring cuttings to surface at 101.5 feet.

Disturbed sample; bulk sample taken at 115.0 feet.

No recovery at 120.0 feet.

*Elevation is approximate and based on USGS datum

DRILLED BY: JDK Drilling, Inc. LOGGED BY: DTW/MBP COMPLETED: 04/26/07

BORING METHOD: hollow-stem auger (see report text) BORING BIT DIAMETER: 8.0 in

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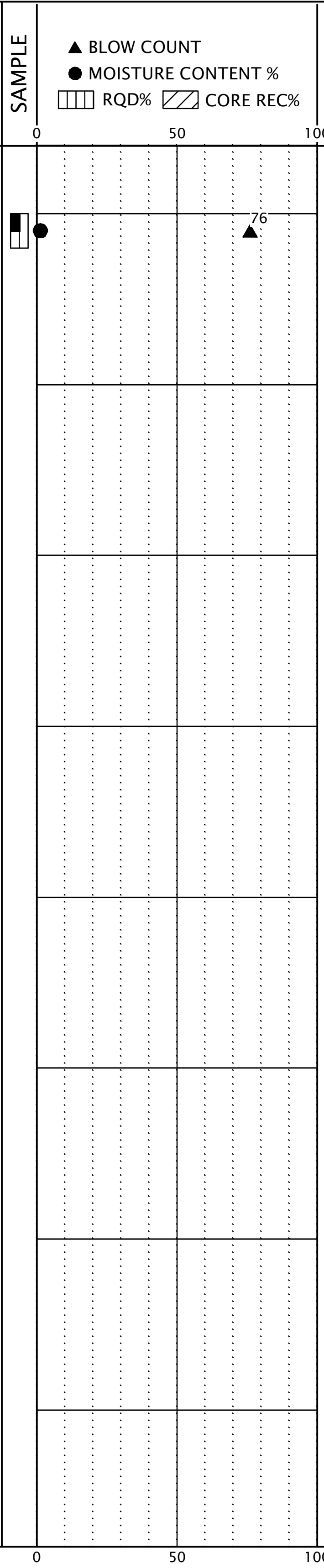
**BORING B-4**
(continued)

PROPOSED 1100 SOUTH OLIVE STREET TOWER
LOS ANGELES, CA

**FIGURE A-4**

BORING - ELEVATION MERUELO-13-01.GPJ GEODESIGN.GDT PRINT DATE: 6/4/07:KYK

| DEPTH FEET | GRAPHIC LOG | MATERIAL DESCRIPTION | ELEVATION* / DEPTH | TESTING | SAMPLE |
|---|---|---|---|---|---|
|  |  | (continued from previous page) |  |  |  |
| 125 |  |  | 117.0 / 126.0 |  |  |
|  |  | **Exploration completed at 126.0 feet.**<br>**Groundwater not encountered.**<br>**Methane probes installed at depths of 25.0, 30.0, and 40.0 feet BGS upon completion of drilling.**<br>**Borehole backfilled with cuttings.** |  |  |  |
| 130 |  |  |  |  |  |
| 135 |  |  |  |  |  |
| 140 |  |  |  |  |  |
| 145 |  |  |  |  |  |
| 150 |  |  |  |  |  |
| 155 |  |  |  |  |  |
| 160 |  |  |  |  |  |



INSTALLATION AND COMMENTS

All samplers driven with 140-lb. hammer falling 30 inches.

*Elevation is approximate and based on USGS datum

DRILLED BY: JDK Drilling, Inc. LOGGED BY: DTW/MBP COMPLETED: 04/26/07

BORING METHOD: hollow-stem auger (see report text) BORING BIT DIAMETER: 8.0 in

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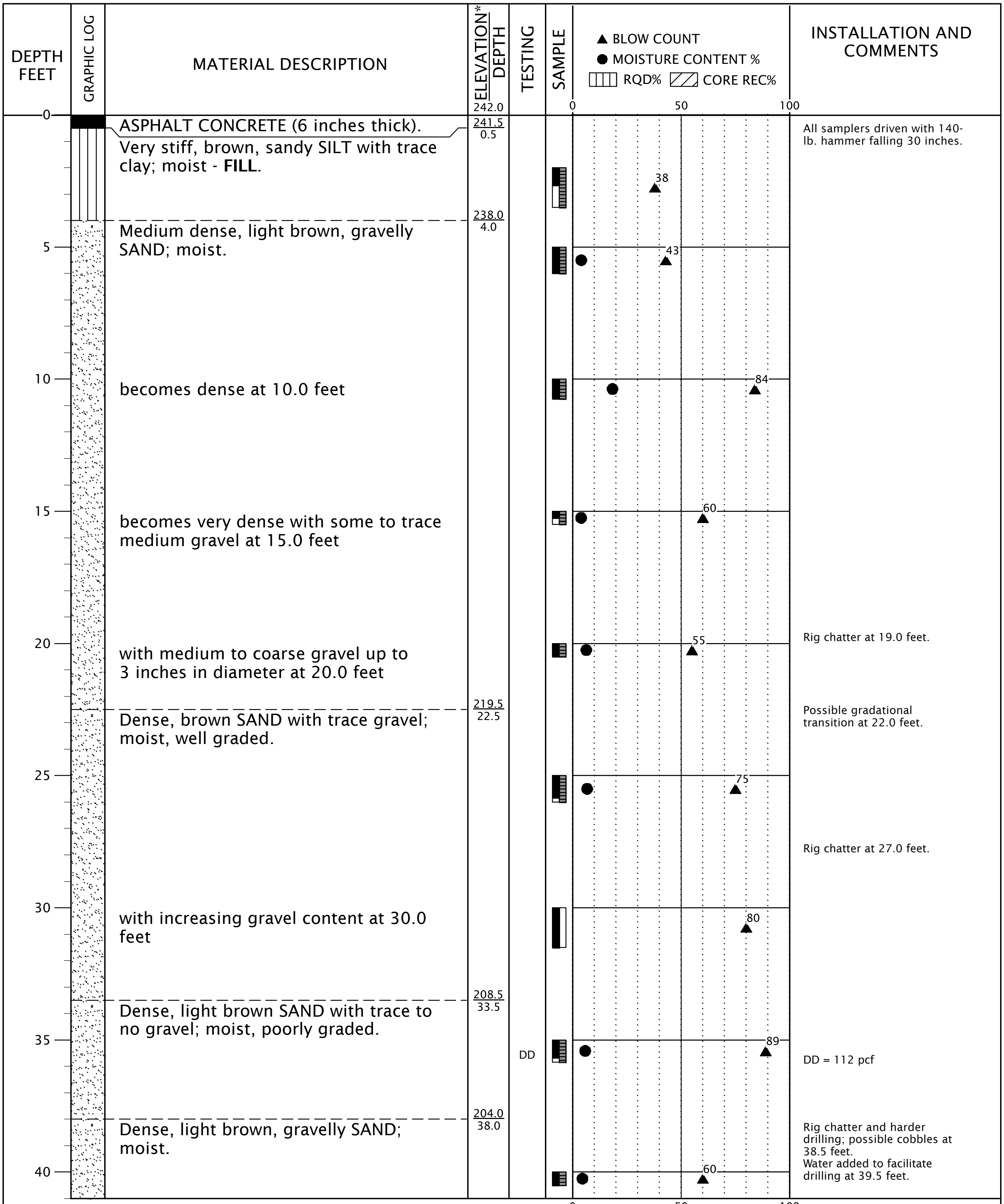
JUNE 2007

**BORING B-4**
(continued)

PROPOSED 1100 SOUTH OLIVE STREET TOWER
LOS ANGELES, CA

**FIGURE A-4**



| DEPTH FEET | MATERIAL DESCRIPTION | ELEVATION* DEPTH | TESTING | BLOW COUNT | INSTALLATION AND COMMENTS |
|---|---|---|---|---|---|
| 0 | ASPHALT CONCRETE (6 inches thick). | 242.0 | | | All samplers driven with 140-lb. hammer falling 30 inches. |
| | Very stiff, brown, sandy SILT with trace clay; moist - **FILL**. | 241.5<br>0.5 | | 38 | |
| | Medium dense, light brown, gravelly SAND; moist. | 238.0<br>4.0 | | | |
| 5 | | | | 43 | |
| 10 | becomes dense at 10.0 feet | | | 84 | |
| 15 | becomes very dense with some to trace medium gravel at 15.0 feet | | | 60 | |
| 20 | with medium to coarse gravel up to 3 inches in diameter at 20.0 feet | | | 55 | Rig chatter at 19.0 feet. |
| | Dense, brown SAND with trace gravel; moist, well graded. | 219.5<br>22.5 | | | Possible gradational transition at 22.0 feet. |
| 25 | | | | 75 | Rig chatter at 27.0 feet. |
| 30 | with increasing gravel content at 30.0 feet | | | 80 | |
| | Dense, light brown SAND with trace to no gravel; moist, poorly graded. | 208.5<br>33.5 | | | |
| 35 | | | DD | 89 | DD = 112 pcf |
| | Dense, light brown, gravelly SAND; moist. | 204.0<br>38.0 | | | Rig chatter and harder drilling; possible cobbles at 38.5 feet.<br>Water added to facilitate drilling at 39.5 feet. |
| 40 | | | | 60 | |

*Elevation is approximate and based on USGS datum

DRILLED BY: JDK Drilling, Inc.
LOGGED BY: DTW/MBP
COMPLETED: 04/25/07

BORING METHOD: hollow-stem auger (see report text)
BORING BIT DIAMETER: 8.0 in

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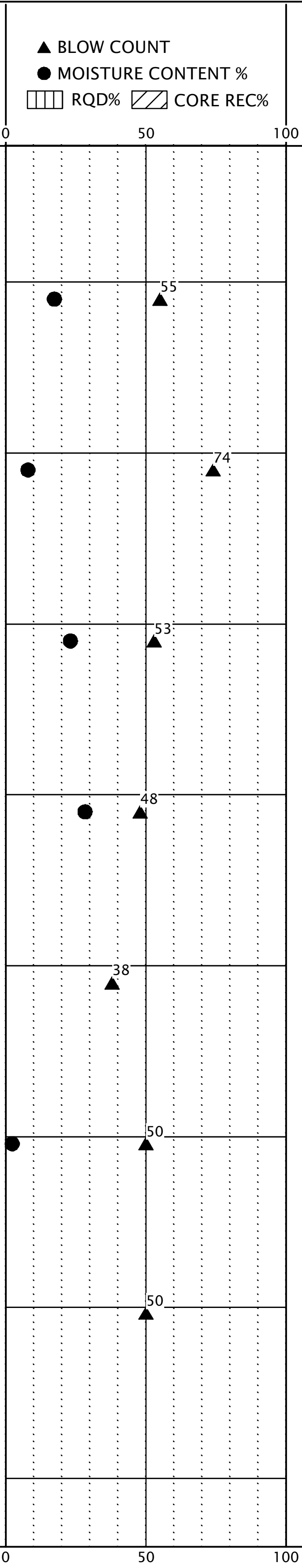
**BORING B-5**

PROPOSED 1100 SOUTH OLIVE STREET TOWER
LOS ANGELES, CA

**FIGURE A-5**

BORING - ELEVATION MERUELO-13-01.GPJ GEODESIGN.GDT PRINT DATE: 6/4/07:KYK

| DEPTH FEET | MATERIAL DESCRIPTION | ELEVATION* DEPTH | TESTING | INSTALLATION AND COMMENTS |
|---|---|---|---|---|
| | (continued from previous page) | | | Rig chatter at 41.5 feet. |
| | Very stiff, brown, sandy SILT with trace clay; moist. | 199.5<br>42.5 | | |
| 45 | | | DD | DD = 108 pcf |
| | | | | Silty cuttings at 47.5 feet. |
| | Dense, brown, silty SAND; moist. | 193.0<br>49.0 | | Water added to facilitate drilling at 49.0 feet. |
| 50 | | | DD<br>DS | DD = 120 pcf |
| | Very stiff, brown, clayey SILT; moist. | 189.0<br>53.0 | | Water added to facilitate drilling at 53.5 feet. |
| 55 | | | DD | DD = 102 pcf |
| | Very stiff, brown, sandy SILT; moist. | 184.0<br>58.0 | | |
| 60 | | | DD<br>DS | DD = 94 pcf |
| | | | | Water added to facilitate drilling at 64.0 feet. |
| 65 | with lenses of silty sand at 65.0 feet | | | |
| | Medium dense, light brown SAND with trace gravel; moist, well graded. | 174.5<br>67.5 | | Rig chatter at 68.0 feet. |
| 70 | | | | |
| | | | | Rig chatter at 71.5 feet. |
| 75 | with increasing gravel content at 75.0 feet | 166.7<br>75.3 | | Disturbed sample at 75.0 feet. |
| | **Exploration completed at 75.3 feet.** | | | |
| | **Groundwater not encountered.** | | | |
| 80 | **Borehole backfilled with cuttings and patched with asphalt concrete.** | | | |



*Elevation is approximate and based on USGS datum

DRILLED BY: JDK Drilling, Inc.

LOGGED BY: DTW/MBP

COMPLETED: 04/25/07

BORING METHOD: hollow-stem auger (see report text)

BORING BIT DIAMETER: 8.0 in

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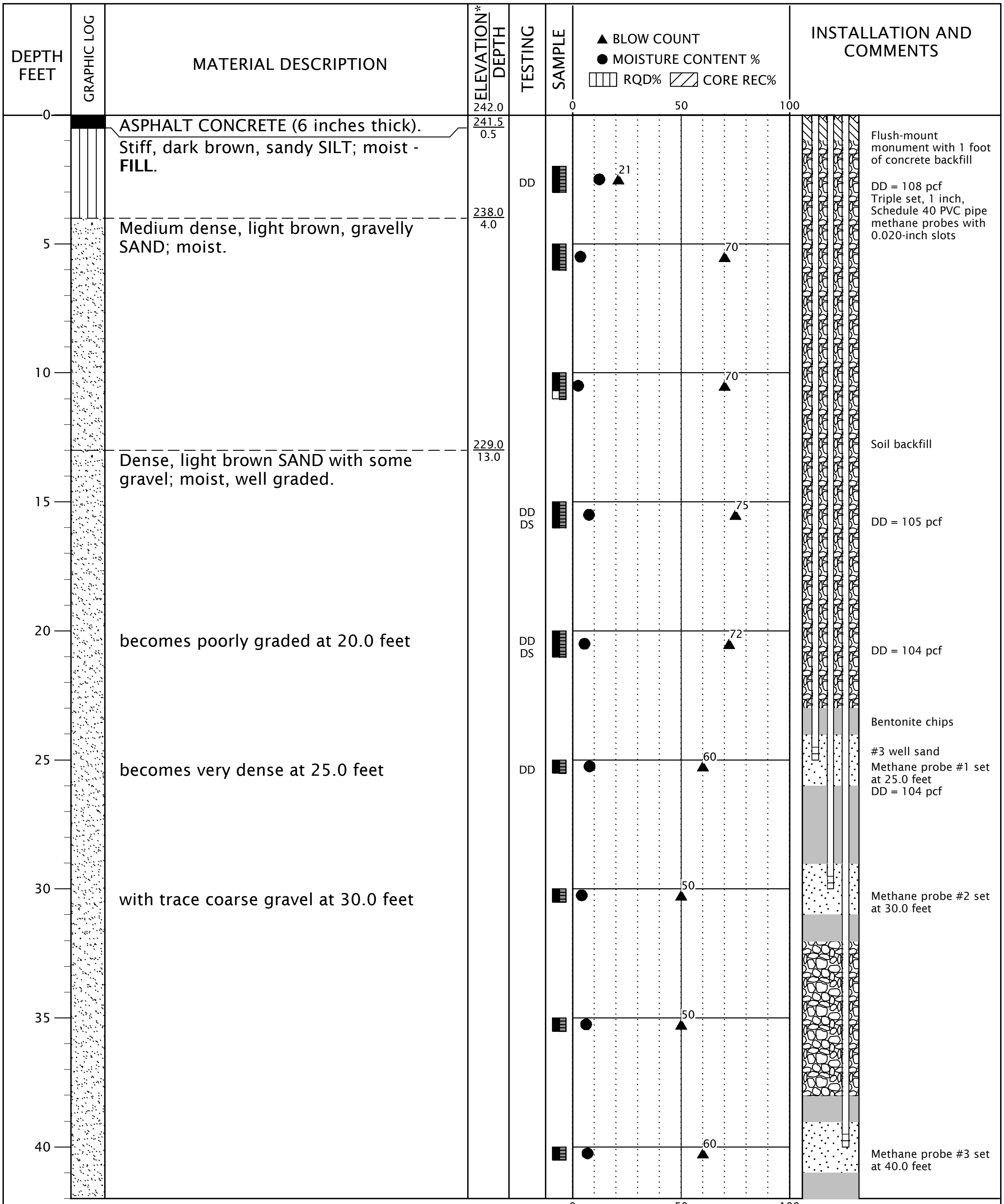
**BORING B-5**
(continued)

PROPOSED 1100 SOUTH OLIVE STREET TOWER
LOS ANGELES, CA

**FIGURE A-5**



ASPHALT CONCRETE (6 inches thick).

Stiff, dark brown, sandy SILT; moist - **FILL**.

Medium dense, light brown, gravelly SAND; moist.

Dense, light brown SAND with some gravel; moist, well graded.

becomes poorly graded at 20.0 feet

becomes very dense at 25.0 feet

with trace coarse gravel at 30.0 feet

Flush-mount monument with 1 foot of concrete backfill

DD = 108 pcf
Triple set, 1 inch, Schedule 40 PVC pipe methane probes with 0.020-inch slots

Soil backfill

DD = 105 pcf

DD = 104 pcf

Bentonite chips

#3 well sand

Methane probe #1 set at 25.0 feet
DD = 104 pcf

Methane probe #2 set at 30.0 feet

Methane probe #3 set at 40.0 feet

*Elevation is approximate and based on USGS datum

DRILLED BY: JDK Drilling, Inc. LOGGED BY: DTW/MBP COMPLETED: 04/25/07

BORING METHOD: hollow-stem auger (see report text) BORING BIT DIAMETER: 8.0 in

| | | |
|---|---|---|
| GEODESIGN INC<br>2121 S Towne Centre Place - Suite 130<br>Anaheim CA 92806<br>Off 714.634.3701 Fax 714.634.3711 | MERUELO-13-01 | **BORING B-6** |
| | JUNE 2007 | PROPOSED 1100 SOUTH OLIVE STREET TOWER<br>LOS ANGELES, CA |
| | | **FIGURE A-6** |

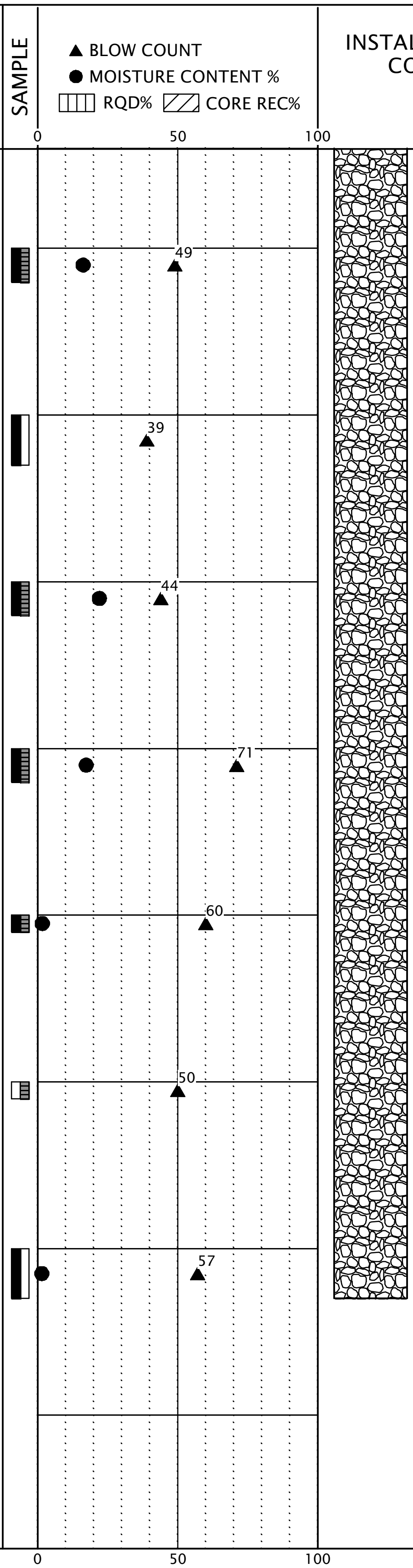
| DEPTH FEET | MATERIAL DESCRIPTION | ELEVATION* / DEPTH | TESTING | INSTALLATION AND COMMENTS |
|---|---|---|---|---|
| | (continued from previous page) | | | |
| | Dense to very dense, brown, silty SAND to sandy SILT; moist. | 199.0 / 43.0 | | |
| 45 | | | DD | DD = 102 pcf |
| 50 | | | | |
| 55 | possible trace clay at 55.0 feet | | DD | DD = 98 pcf |
| 60 | | | DD | DD = 100 pcf |
| | Medium dense, light brown SAND with trace gravel; moist, poorly graded. | 177.5 / 64.5 | | |
| 65 | | | | |
| 70 | | | | No recovery at 70.0 feet. |
| 75 | with 6-inch layer of clayey silt at 75.5 feet | 165.5 / 76.5 | | All samplers driven with 140-lb. hammer falling 30 inches. |
| 80 | | | | |

**Exploration completed at 76.5 feet.**

**Groundwater not encountered.**

**Methane probes installed at depths of 25.0, 30.0, and 40.0 feet BGS upon completion of drilling.**

**Borehole backfilled with cuttings.**



*Elevation is approximate and based on USGS datum

DRILLED BY: JDK Drilling, Inc.
LOGGED BY: DTW/MBP
COMPLETED: 04/25/07

BORING METHOD: hollow-stem auger (see report text)
BORING BIT DIAMETER: 8.0 in

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JUNE 2007

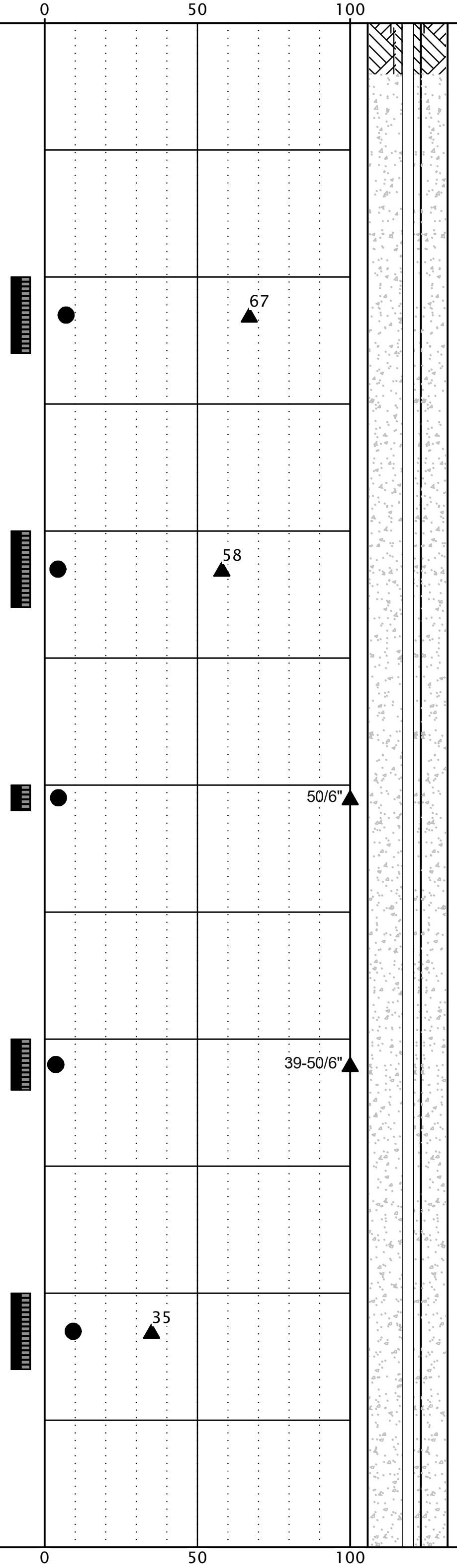
**BORING B-6**
(continued)

PROPOSED 1100 SOUTH OLIVE STREET TOWER
LOS ANGELES, CA

**FIGURE A-6**

| DEPTH FEET | MATERIAL DESCRIPTION | ELEVATION DEPTH |
|---|---|---|
| 0.0 | ASPHALT CONCRETE (3.0 inches). | 245.2 |
|  | Medium dense, dark brown, clayey SAND (SC); moist, fine - **FILL**. | 244.9<br>0.3 |
|  | Dense, dark brown, silty SAND with gravel and cobbles (SM); moist, medium to coarse, gravel is fine to coarse, micaceous. | 242.2<br>3.0 |
| 10.0 | decreased silt; medium to coarse at 10.0 feet |  |
| 15.0 | very dense, increased silt; fine to medium at 15.0 feet |  |
|  | Very dense, light brown SAND with gravel and cobbles (SP); moist, medium to coarse, gravel is fine to coarse, micaceous. | 227.2<br>18.0 |
|  | Very stiff, olive-brown SILT (ML), trace sand; moist, low plasticity, sand is fine, trace pin hole pores, trace iron oxide staining, micaceous. | 222.7<br>22.5 |
|  | Medium dense, red-brown, silty SAND (SM); moist, fine to coarse, micaceous. | 217.2<br>28.0 |
| 30.0 |  |  |

| TESTING | ▲ BLOW COUNT | INSTALLATION AND COMMENTS |
|---|---|---|
|  |  | Flush-mount monument with 1 foot of concrete backfill |
|  |  | Hand augered to 5.0 feet. |
|  |  | Cement-bentonite grout |
| DD | 67 | DD = 126 pcf |
| DD<br>DS | 58 | DD = 119 pcf |
| DD | 50/6" | DD = 110 pcf |
| DD<br>DS | 39-50/6" | DD = 114 pcf |
| DD | 35 | DD = 126 pcf |



DRILLED BY: Martini Drilling

LOGGED BY: AJA

COMPLETED: 12/05/17

BORING METHOD: hollow-stem auger (see document text)

BORING BIT DIAMETER: 8 inches

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DTLASITE3-1-01

JUNE 2018

**BORING B-7**

PROPOSED HIGH-RISE TOWER - SITE 3
LOS ANGELES, CA

**FIGURE A-1**

| DEPTH FEET | MATERIAL DESCRIPTION | ELEVATION DEPTH | TESTING | ▲ BLOW COUNT | INSTALLATION AND COMMENTS |
|---|---|---|---|---|---|
| 30.0 | (continued from previous page) | | | 25 | |
| 32.5 | Medium stiff, olive-brown CLAY (CL), trace sand; moist, sand is fine, high plasticity. | 212.7<br>32.5 | | | |
| 35.0 | Very dense, yellow-brown SAND with gravel (SP); moist, medium to coarse, gravel is fine to coarse, micaceous. | 209.9<br>35.3 | DD | 50/6" | DD = 94 pcf<br>Bentonite chips |
| 37.5 | Medium dense, olive-brown, clayey SAND (SC); moist, fine to medium. | 207.7<br>37.5 | | | |
| 40.0 | Very dense, yellow-brown SAND with gravel (SP); moist, medium to coarse, gravel is fine to coarse. | 204.7<br>40.5 | | 68 | |
| | Hard, dark brown CLAY with sand (CL); moist, medium plasticity, sand is fine. | 203.7<br>41.5 | | | |
| 42.5 | | | | | |
| 45.0 | olive-brown, sandy; micaceous at 45.0 feet | | DD<br>CON<br>DS | 64 | Clean sand<br>DD = 110 pcf<br>Vapor probe set at 45.0 feet<br>Bentonite chips |
| 47.5 | Medium dense, red-brown, clayey SAND (SC); moist, fine to medium, micaceous. | 197.2<br>48.0 | | | Clean sand |
| 50.0 | | | | 25 | Vapor probe set at 50.0 feet |
| 52.5 | Very dense, yellow-brown SAND with silt (SP-SM); moist, fine to medium, micaceous. | 192.2<br>53.0 | | | Bentonite chips |
| 55.0 | | | DD | 18-50/6" | DD = 101 pcf |
| 57.5 | Stiff, olive-brown, sandy CLAY (CL); moist, medium plasticity, sand is fine. | 187.2<br>58.0 | | | Clean sand |
| 60.0 | | | | | |

▲ BLOW COUNT
● MOISTURE CONTENT %
▥ RQD% ▨ CORE REC%
0 50 100

DRILLED BY: Martini Drilling
LOGGED BY: AJA
COMPLETED: 12/05/17

BORING METHOD: hollow-stem auger (see document text)
BORING BIT DIAMETER: 8 inches

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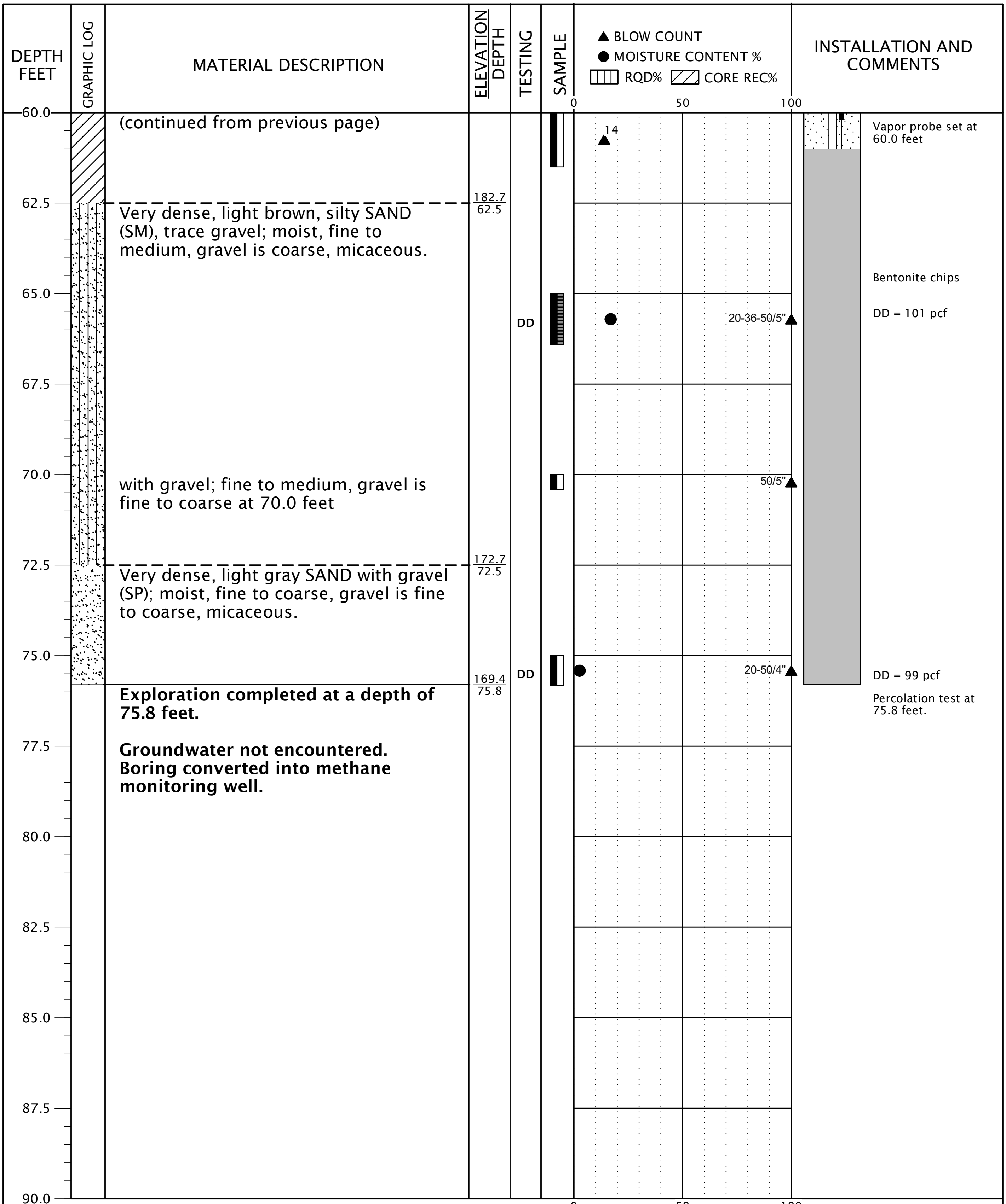
JUNE 2018

**BORING B-7**
(continued)

PROPOSED HIGH-RISE TOWER - SITE 3
LOS ANGELES, CA

**FIGURE A-1**

BORING LOG DTLASITE3-1-01-B7_9.GPJ GEODESIGN.GDT PRINT DATE: 6/19/18:RC:KT



| DEPTH FEET | GRAPHIC LOG | MATERIAL DESCRIPTION | ELEVATION<br>DEPTH | TESTING | SAMPLE | ▲ BLOW COUNT<br>● MOISTURE CONTENT %<br>RQD% CORE REC% | INSTALLATION AND COMMENTS |
|---|---|---|---|---|---|---|---|
| 60.0 | | (continued from previous page) | | | | ▲ 14 | Vapor probe set at 60.0 feet |
| 62.5 | | Very dense, light brown, silty SAND (SM), trace gravel; moist, fine to medium, gravel is coarse, micaceous. | 182.7<br>62.5 | | | | Bentonite chips |
| 65.0 | | | | DD | | ● 20-36-50/5" ▲ | DD = 101 pcf |
| 70.0 | | with gravel; fine to medium, gravel is fine to coarse at 70.0 feet | | | | 50/5" ▲ | |
| 72.5 | | Very dense, light gray SAND with gravel (SP); moist, fine to coarse, gravel is fine to coarse, micaceous. | 172.7<br>72.5 | | | | |
| 75.0 | | | | DD | | ● 20-50/4" ▲ | DD = 99 pcf |
| 75.8 | | **Exploration completed at a depth of 75.8 feet.**<br>**Groundwater not encountered. Boring converted into methane monitoring well.** | 169.4<br>75.8 | | | | Percolation test at 75.8 feet. |

DRILLED BY: Martini Drilling LOGGED BY: AJA COMPLETED: 12/05/17

BORING METHOD: hollow-stem auger (see document text) BORING BIT DIAMETER: 8 inches

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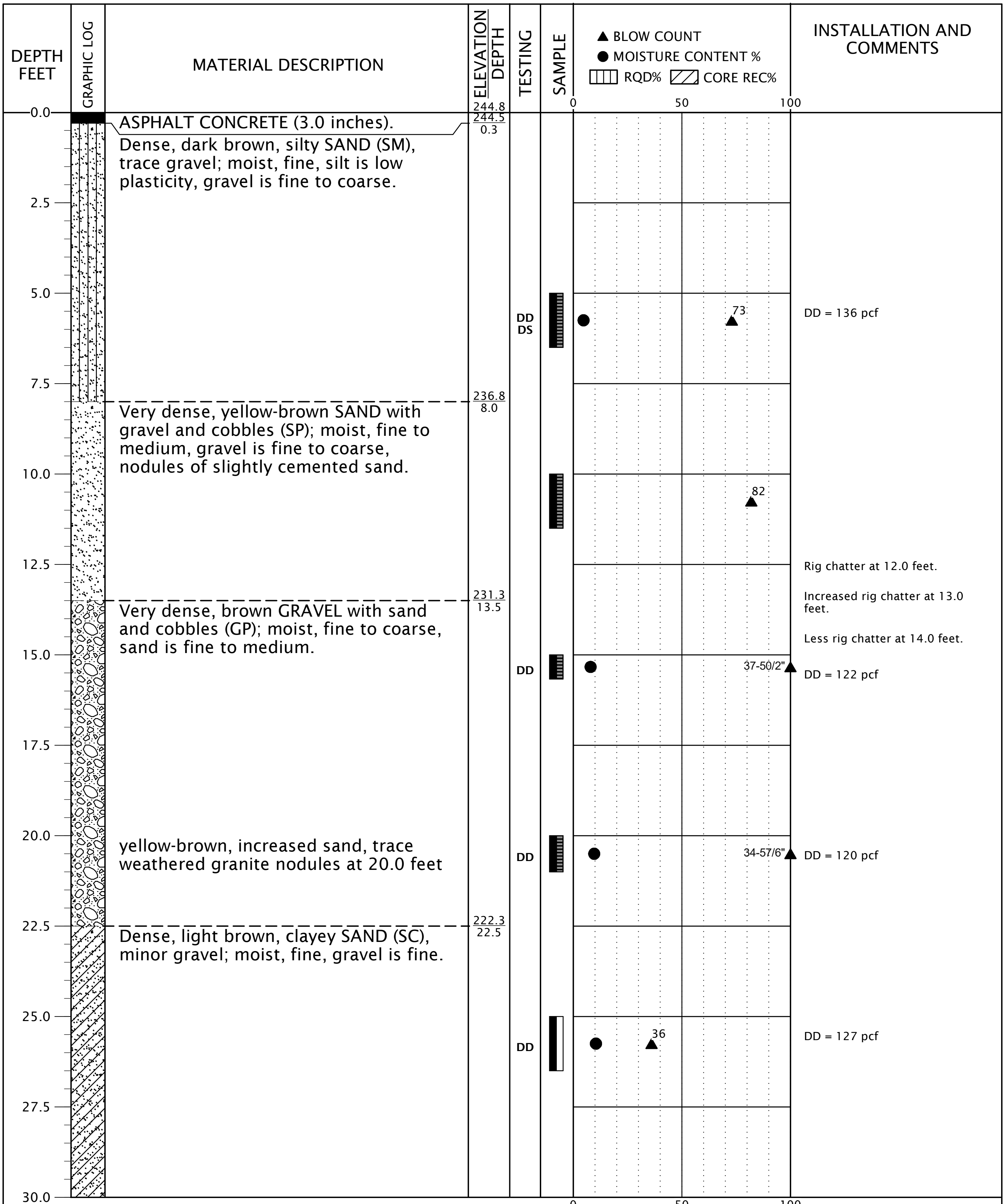
**BORING B-7**
(continued)

PROPOSED HIGH-RISE TOWER - SITE 3
LOS ANGELES, CA

**FIGURE A-1**



| MATERIAL DESCRIPTION | ELEVATION DEPTH | TESTING | ▲ BLOW COUNT | INSTALLATION AND COMMENTS |
|---|---|---|---|---|
| ASPHALT CONCRETE (3.0 inches). | 244.8 | | | |
| Dense, dark brown, silty SAND (SM), trace gravel; moist, fine, silt is low plasticity, gravel is fine to coarse. | 244.5<br>0.3 | | | |
| | | DD<br>DS | 73 | DD = 136 pcf |
| Very dense, yellow-brown SAND with gravel and cobbles (SP); moist, fine to medium, gravel is fine to coarse, nodules of slightly cemented sand. | 236.8<br>8.0 | | | |
| | | | 82 | |
| | | | | Rig chatter at 12.0 feet. |
| | | | | Increased rig chatter at 13.0 feet. |
| Very dense, brown GRAVEL with sand and cobbles (GP); moist, fine to coarse, sand is fine to medium. | 231.3<br>13.5 | | | |
| | | | | Less rig chatter at 14.0 feet. |
| | | DD | 37-50/2" | DD = 122 pcf |
| yellow-brown, increased sand, trace weathered granite nodules at 20.0 feet | | DD | 34-57/6" | DD = 120 pcf |
| Dense, light brown, clayey SAND (SC), minor gravel; moist, fine, gravel is fine. | 222.3<br>22.5 | | | |
| | | DD | 36 | DD = 127 pcf |

DRILLED BY: SoCal Drilling

LOGGED BY: AJA

COMPLETED: 11/30/17

BORING METHOD: mud rotary (see document text)

BORING BIT DIAMETER: 4 inches

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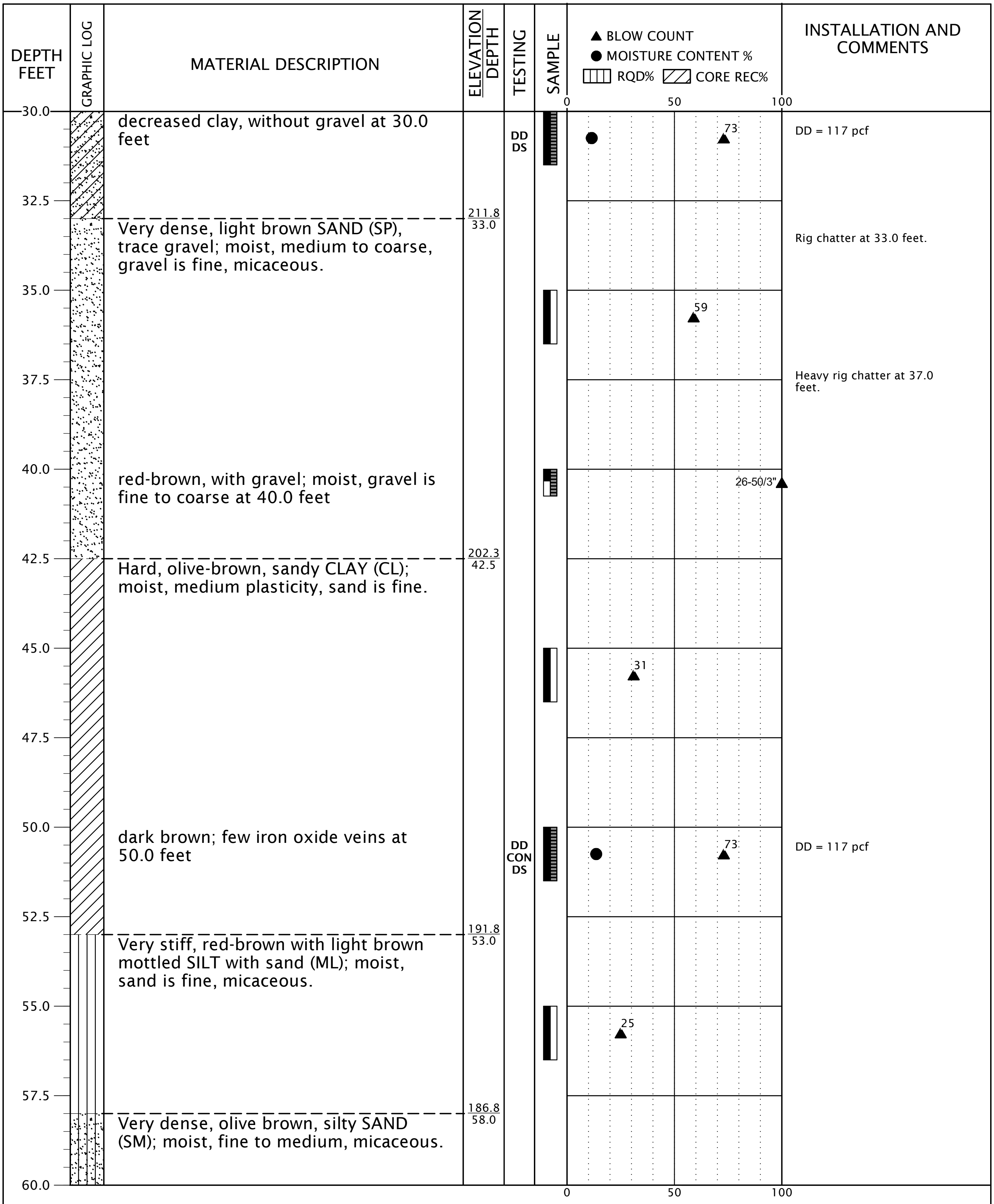
JUNE 2018

**BORING B-8**

PROPOSED HIGH-RISE TOWER - SITE 3
LOS ANGELES, CA

**FIGURE A-2**



| DEPTH FEET | MATERIAL DESCRIPTION | ELEVATION / DEPTH | TESTING | BLOW COUNT | INSTALLATION AND COMMENTS |
|---|---|---|---|---|---|
| 30.0 | decreased clay, without gravel at 30.0 feet | | DD<br>DS | 73 | DD = 117 pcf |
| 33.0 | Very dense, light brown SAND (SP), trace gravel; moist, medium to coarse, gravel is fine, micaceous. | 211.8<br>33.0 | | | Rig chatter at 33.0 feet. |
| 35.0 | | | | 59 | |
| 37.0 | | | | | Heavy rig chatter at 37.0 feet. |
| 40.0 | red-brown, with gravel; moist, gravel is fine to coarse at 40.0 feet | | | 26-50/3" | |
| 42.5 | Hard, olive-brown, sandy CLAY (CL); moist, medium plasticity, sand is fine. | 202.3<br>42.5 | | | |
| 45.0 | | | | 31 | |
| 50.0 | dark brown; few iron oxide veins at 50.0 feet | | DD<br>CON<br>DS | 73 | DD = 117 pcf |
| 53.0 | Very stiff, red-brown with light brown mottled SILT with sand (ML); moist, sand is fine, micaceous. | 191.8<br>53.0 | | | |
| 55.0 | | | | 25 | |
| 58.0 | Very dense, olive brown, silty SAND (SM); moist, fine to medium, micaceous. | 186.8<br>58.0 | | | |

DRILLED BY: SoCal Drilling
LOGGED BY: AJA
COMPLETED: 11/30/17

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

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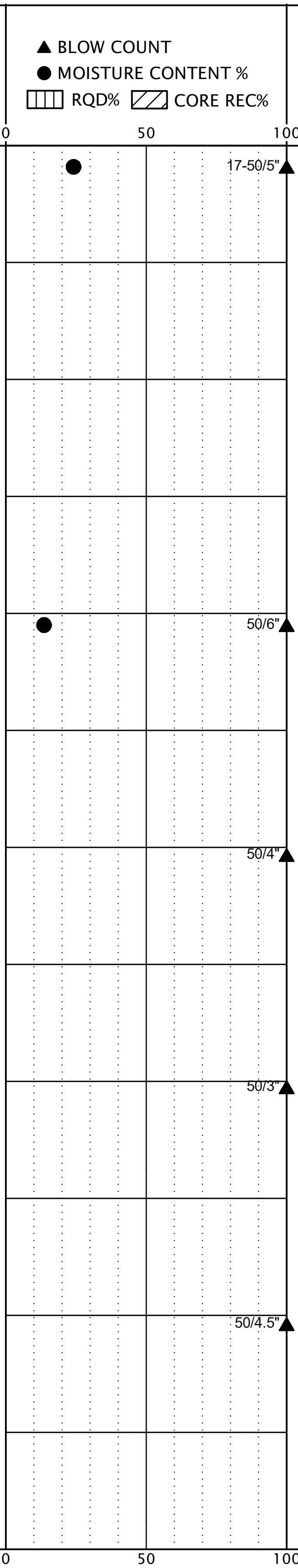
**BORING B-8**
(continued)

PROPOSED HIGH-RISE TOWER - SITE 3
LOS ANGELES, CA

**FIGURE A-2**

BORING LOG DTLASITE3-1-01-B7_9.GPJ GEODESIGN.GDT PRINT DATE: 6/19/18:RC:KT

| DEPTH FEET | MATERIAL DESCRIPTION | ELEVATION DEPTH | TESTING | BLOW COUNT | INSTALLATION AND COMMENTS |
|---|---|---|---|---|---|
| 60.0 | (continued from previous page) |  | DD | 17-50/5" | DD = 102 pcf |
| 67.5 | Very dense, light brown SAND with cobbles (SP), trace gravel; moist, fine to coarse, gravel is fine to coarse, micaceous. | 177.3<br>67.5 |  |  | Rig chatter at 67.5 feet. |
| 70.0 |  |  | DD | 50/6" | DD = 111 pcf |
| 72.5 |  |  |  |  | Large cobble, heavy/rig chatter at 73.0 feet.<br>Drilling slow due to cobbles. |
| 75.0 | red-brown, with gravel and cobbles; medium to coarse at 75.0 feet |  |  | 50/4" |  |
| 80.0 |  |  |  | 50/3" | Sample disturbed.<br>Heavy rig chatter at 80.0 feet. |
| 85.0 | decreased gravel, without cobbles at 85.0 feet |  |  | 50/4.5" |  |
| 89.0 | Very dense, dark olive-brown, silty SAND (SM); moist, fine to medium, | 155.8<br>89.0 |  |  |  |



DRILLED BY: SoCal Drilling
LOGGED BY: AJA
COMPLETED: 11/30/17

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

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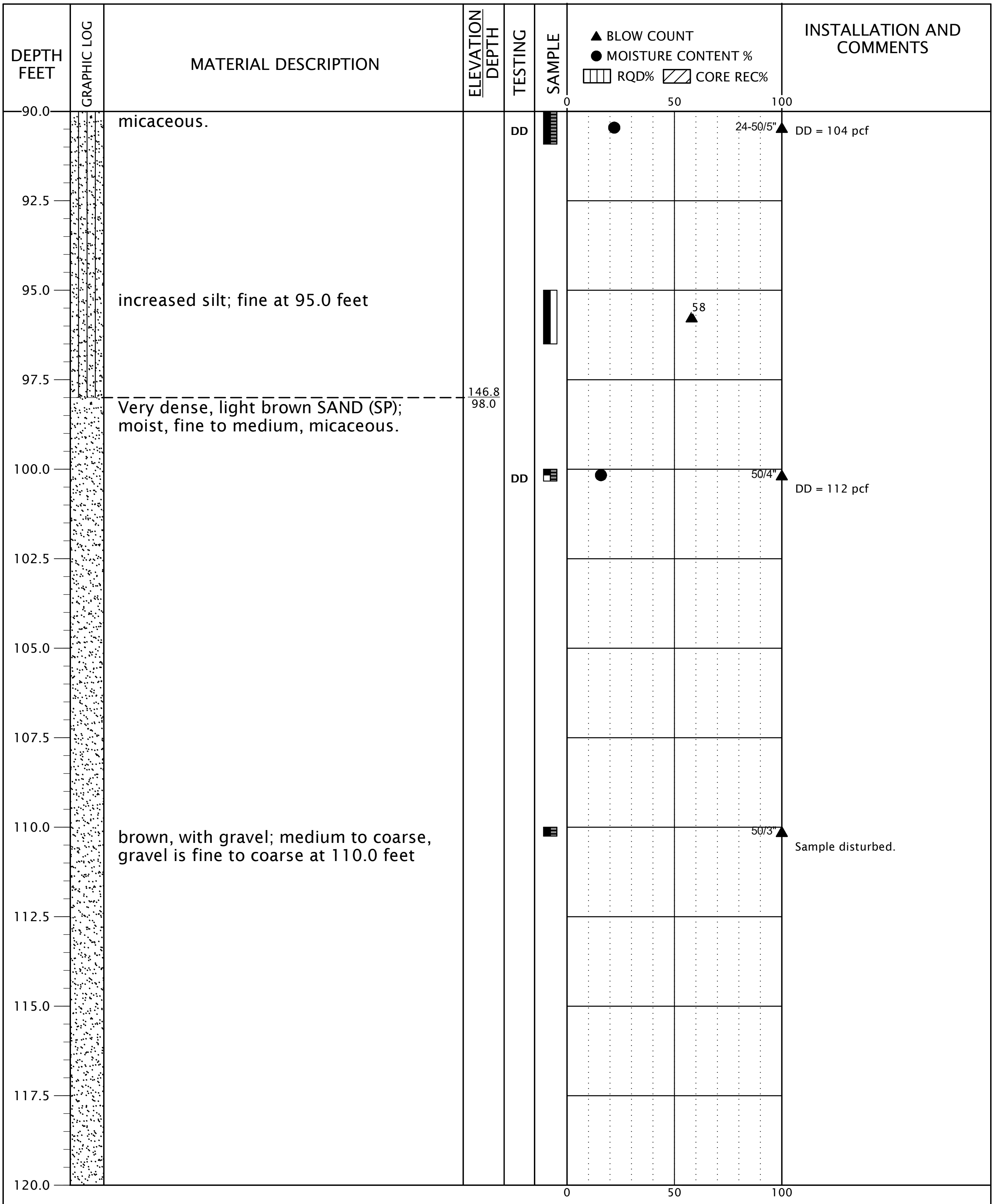
DTLASITE3-1-01

JUNE 2018

**BORING B-8**
(continued)

PROPOSED HIGH-RISE TOWER - SITE 3
LOS ANGELES, CA

**FIGURE A-2**



| DEPTH FEET | MATERIAL DESCRIPTION | ELEVATION / DEPTH | TESTING | BLOW COUNT | INSTALLATION AND COMMENTS |
|---|---|---|---|---|---|
| 90.0 | micaceous. | | DD | 24-50/5" | DD = 104 pcf |
| 95.0 | increased silt; fine at 95.0 feet | | | 58 | |
| 98.0 | Very dense, light brown SAND (SP); moist, fine to medium, micaceous. | 146.8<br>98.0 | | | |
| 100.0 | | | DD | 50/4" | DD = 112 pcf |
| 110.0 | brown, with gravel; medium to coarse, gravel is fine to coarse at 110.0 feet | | | 50/3" | Sample disturbed. |

DRILLED BY: SoCal Drilling
LOGGED BY: AJA
COMPLETED: 11/30/17

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

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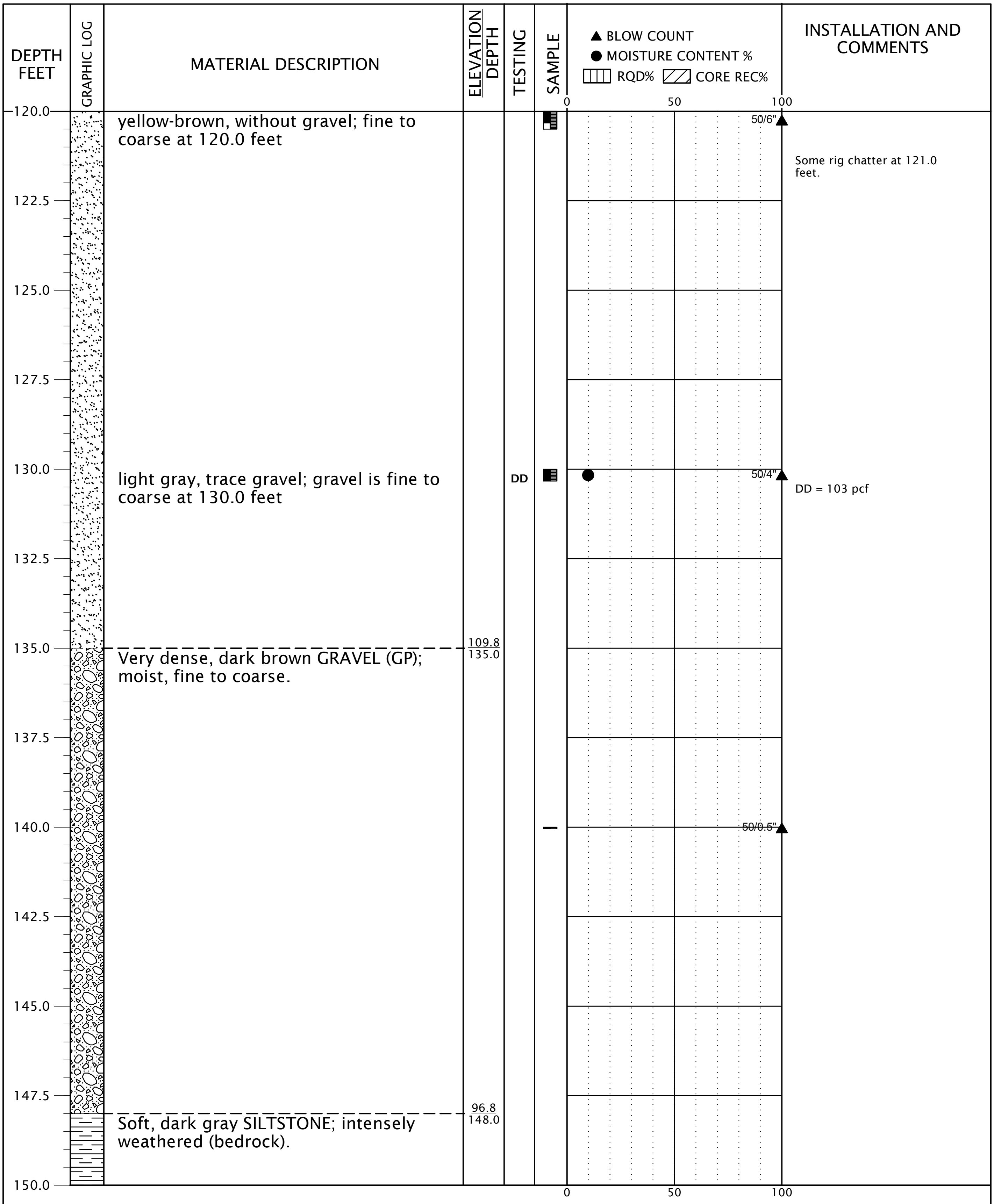
**BORING B-8**
(continued)

PROPOSED HIGH-RISE TOWER - SITE 3
LOS ANGELES, CA

**FIGURE A-2**



| DEPTH FEET | MATERIAL DESCRIPTION | ELEVATION DEPTH | TESTING | ▲ BLOW COUNT ● MOISTURE CONTENT % ▥ RQD% ▨ CORE REC% | INSTALLATION AND COMMENTS |
|---|---|---|---|---|---|
| 120.0 | yellow-brown, without gravel; fine to coarse at 120.0 feet | | | ▲ 50/6" | Some rig chatter at 121.0 feet. |
| 122.5 | | | | | |
| 125.0 | | | | | |
| 127.5 | | | | | |
| 130.0 | light gray, trace gravel; gravel is fine to coarse at 130.0 feet | | DD | ● ▲ 50/4" | DD = 103 pcf |
| 132.5 | | | | | |
| 135.0 | Very dense, dark brown GRAVEL (GP); moist, fine to coarse. | 109.8<br>135.0 | | | |
| 137.5 | | | | | |
| 140.0 | | | | ▲ 50/0.5" | |
| 142.5 | | | | | |
| 145.0 | | | | | |
| 147.5 | | | | | |
| | Soft, dark gray SILTSTONE; intensely weathered (bedrock). | 96.8<br>148.0 | | | |
| 150.0 | | | | | |

DRILLED BY: SoCal Drilling

LOGGED BY: AJA

COMPLETED: 11/30/17

BORING METHOD: mud rotary (see document text)

BORING BIT DIAMETER: 4 inches

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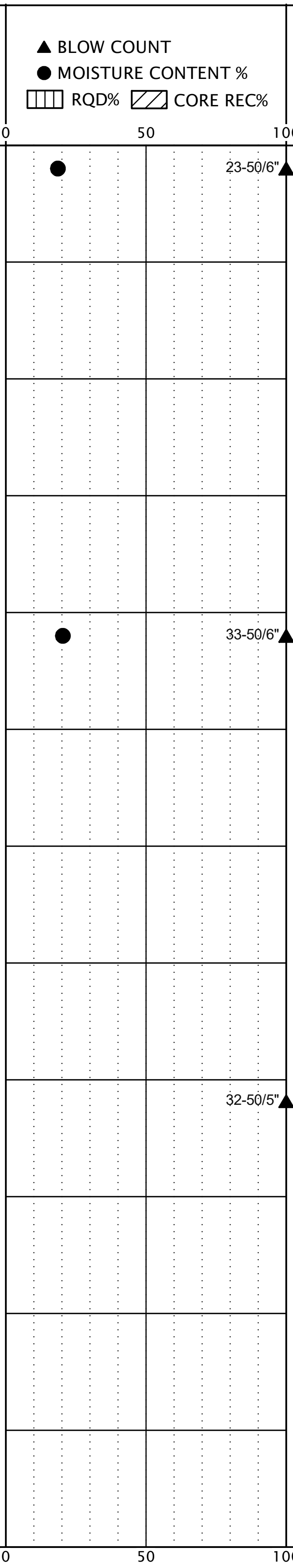
**BORING B-8**
(continued)

PROPOSED HIGH-RISE TOWER - SITE 3
LOS ANGELES, CA

**FIGURE A-2**

| DEPTH FEET | MATERIAL DESCRIPTION | ELEVATION / DEPTH | TESTING | SAMPLE | BLOW COUNT | INSTALLATION AND COMMENTS |
|---|---|---|---|---|---|---|
| 150.0 | (continued from previous page) | | DD | ▮ | 23-50/6" | DD = 111 pcf |
| 160.0 | sandy at 160.0 feet | | DD | ▮ | 33-50/6" | DD = 107 pcf |
| 170.0 | olive-green at 170.0 feet | | | ▮ | 32-50/5" | |
| 172.0 | Medium hard, light gray, silty SANDSTONE; slightly weathered. | 72.8<br>172.0 | | | | Heavy rig chatter at 172.0 feet. |



Depth scale: 150.0, 152.5, 155.0, 157.5, 160.0, 162.5, 165.0, 167.5, 170.0, 172.5, 175.0, 177.5, 180.0

DRILLED BY: SoCal Drilling
LOGGED BY: AJA
COMPLETED: 11/30/17

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

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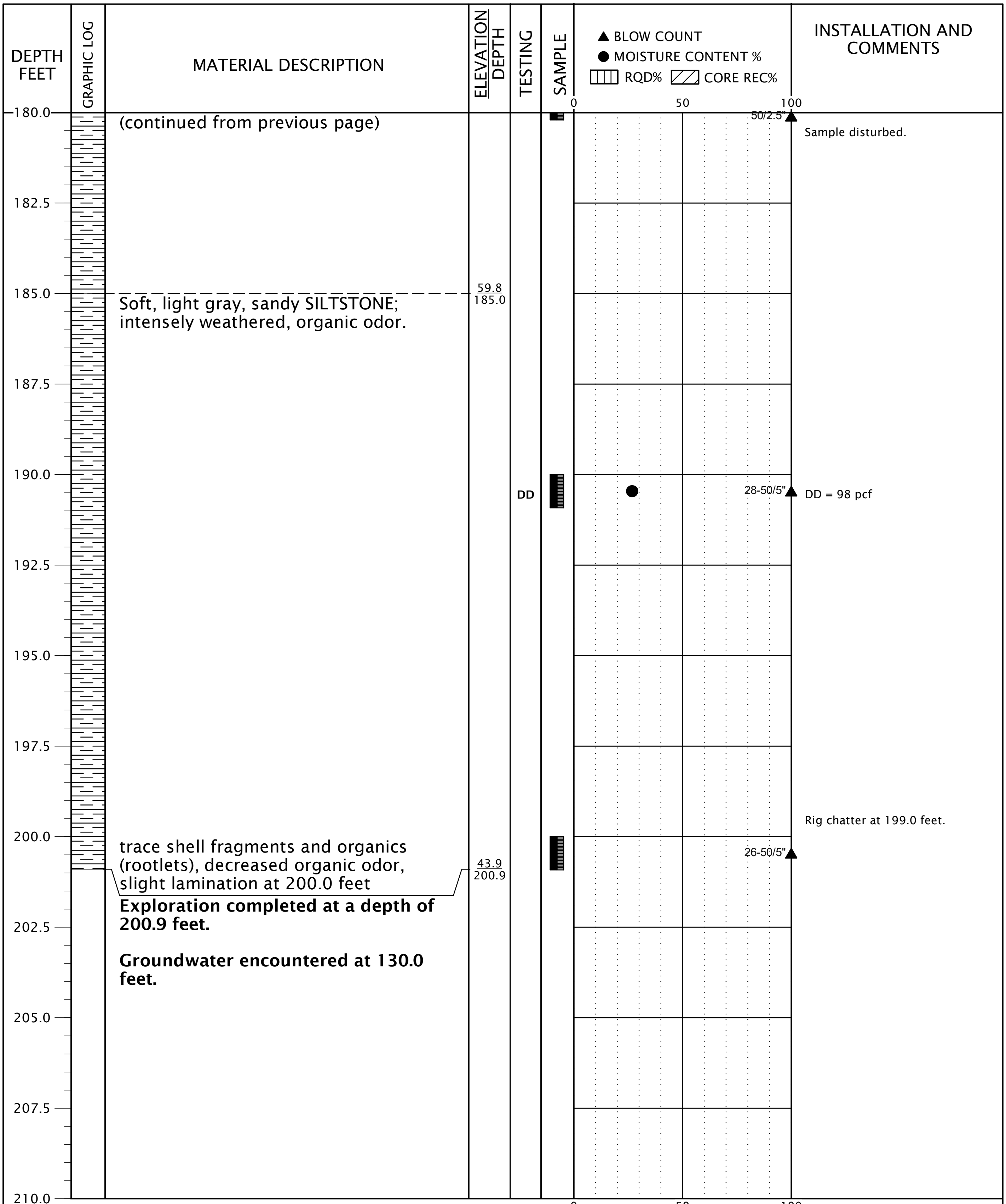
JUNE 2018

**BORING B-8**
(continued)

PROPOSED HIGH-RISE TOWER - SITE 3
LOS ANGELES, CA

**FIGURE A-2**



| DEPTH FEET | MATERIAL DESCRIPTION | ELEVATION DEPTH | TESTING | BLOW COUNT | INSTALLATION AND COMMENTS |
|---|---|---|---|---|---|
| 180.0 | (continued from previous page) | | | 50/2.5" | Sample disturbed. |
| 185.0 | Soft, light gray, sandy SILTSTONE; intensely weathered, organic odor. | 59.8<br>185.0 | | | |
| 190.0 | | | DD | 28-50/5" | DD = 98 pcf |
| | | | | | Rig chatter at 199.0 feet. |
| 200.0 | trace shell fragments and organics (rootlets), decreased organic odor, slight lamination at 200.0 feet | 43.9<br>200.9 | | 26-50/5" | |

**Exploration completed at a depth of 200.9 feet.**

**Groundwater encountered at 130.0 feet.**

DRILLED BY: SoCal Drilling  LOGGED BY: AJA  COMPLETED: 11/30/17

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

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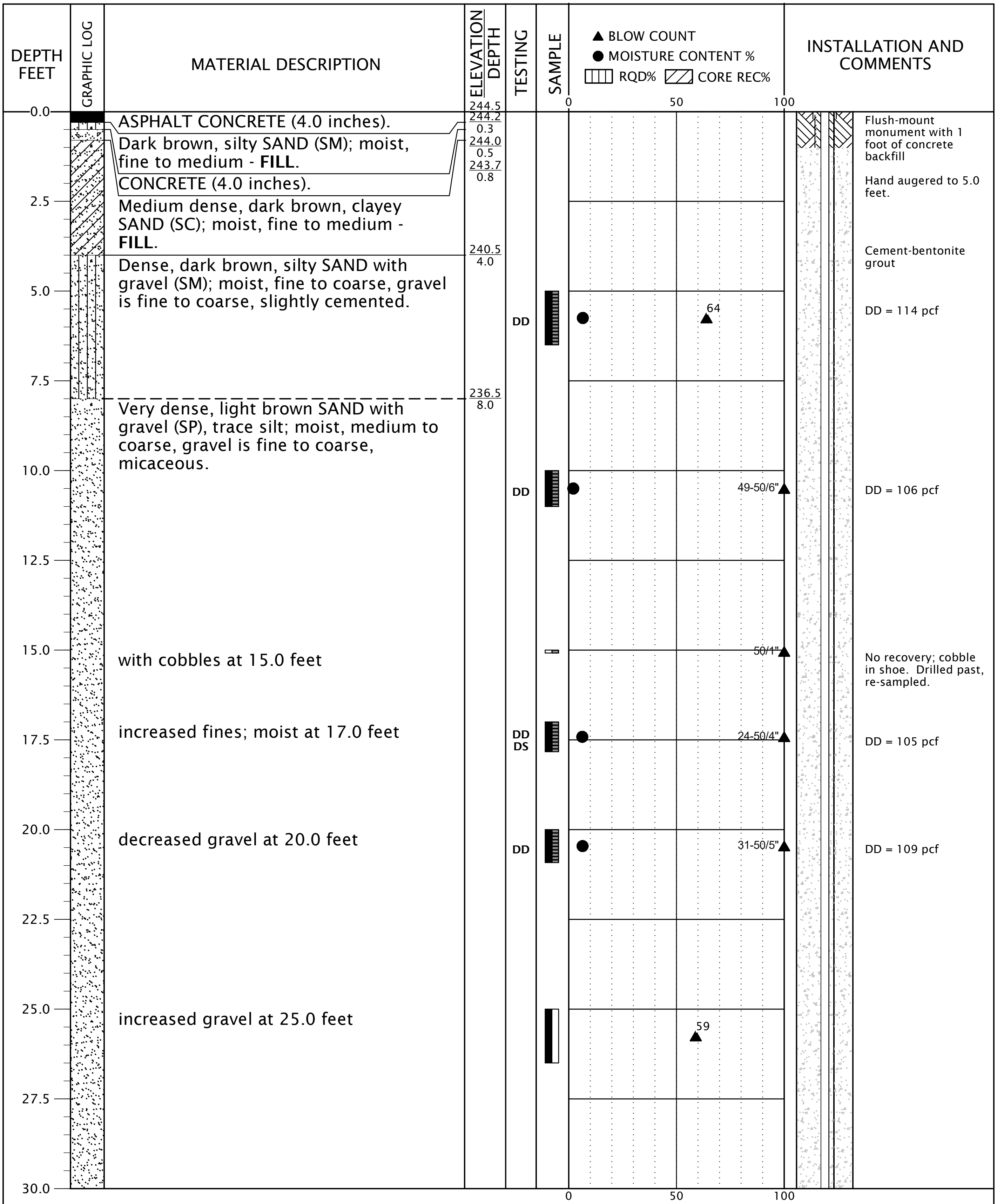
**BORING B-8**
(continued)

PROPOSED HIGH-RISE TOWER - SITE 3
LOS ANGELES, CA

**FIGURE A-2**



| DEPTH FEET | MATERIAL DESCRIPTION | ELEVATION DEPTH | TESTING | BLOW COUNT | INSTALLATION AND COMMENTS |
|---|---|---|---|---|---|
| 0.0 | | 244.5 | | | |
| | ASPHALT CONCRETE (4.0 inches). | 244.2<br>0.3 | | | Flush-mount monument with 1 foot of concrete backfill |
| | Dark brown, silty SAND (SM); moist, fine to medium - **FILL**. | 244.0<br>0.5 | | | |
| | CONCRETE (4.0 inches). | 243.7<br>0.8 | | | Hand augered to 5.0 feet. |
| 2.5 | Medium dense, dark brown, clayey SAND (SC); moist, fine to medium - **FILL**. | 240.5<br>4.0 | | | Cement-bentonite grout |
| 5.0 | Dense, dark brown, silty SAND with gravel (SM); moist, fine to coarse, gravel is fine to coarse, slightly cemented. | | DD | 64 | DD = 114 pcf |
| 7.5 | | 236.5<br>8.0 | | | |
| 10.0 | Very dense, light brown SAND with gravel (SP), trace silt; moist, medium to coarse, gravel is fine to coarse, micaceous. | | DD | 49-50/6" | DD = 106 pcf |
| 15.0 | with cobbles at 15.0 feet | | | 50/1" | No recovery; cobble in shoe. Drilled past, re-sampled. |
| 17.5 | increased fines; moist at 17.0 feet | | DD<br>DS | 24-50/4" | DD = 105 pcf |
| 20.0 | decreased gravel at 20.0 feet | | DD | 31-50/5" | DD = 109 pcf |
| 25.0 | increased gravel at 25.0 feet | | | 59 | |
| 30.0 | | | | | |

DRILLED BY: Martini Drilling LOGGED BY: AJA COMPLETED: 12/06/17

BORING METHOD: hollow-stem auger (see document text) BORING BIT DIAMETER: 8 inches

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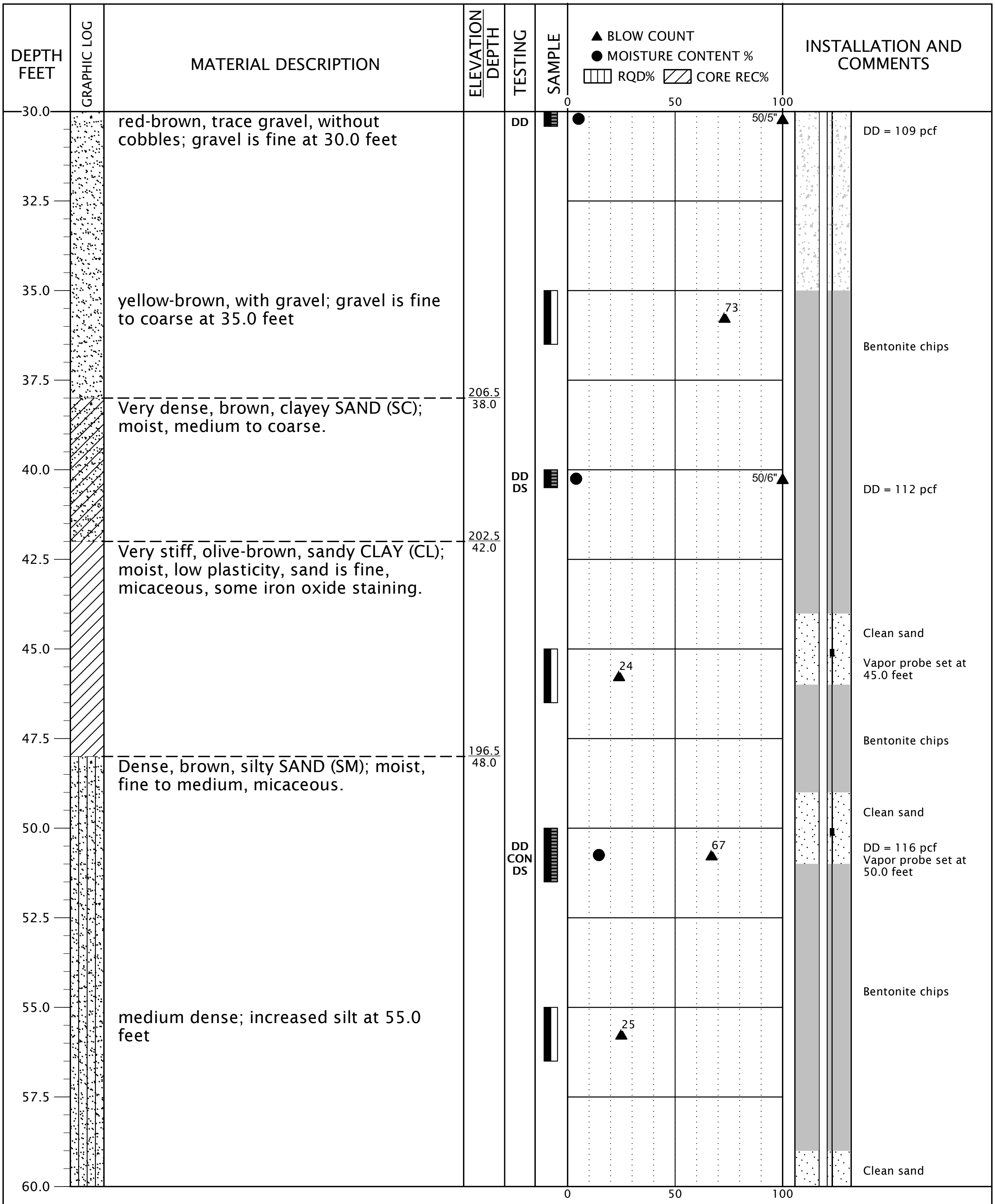
**BORING B-9**

PROPOSED HIGH-RISE TOWER - SITE 3
LOS ANGELES, CA

**FIGURE A-3**

BORING LOG DTLASITE3-1-01-B7_9.GPJ GEODESIGN.GDT PRINT DATE: 6/19/18:RC:KT



| DEPTH FEET | MATERIAL DESCRIPTION | ELEVATION DEPTH | TESTING | BLOW COUNT | INSTALLATION AND COMMENTS |
|---|---|---|---|---|---|
| 30.0 | red-brown, trace gravel, without cobbles; gravel is fine at 30.0 feet | | DD | 50/5" | DD = 109 pcf |
| 35.0 | yellow-brown, with gravel; gravel is fine to coarse at 35.0 feet | | | 73 | Bentonite chips |
| 38.0 | Very dense, brown, clayey SAND (SC); moist, medium to coarse. | 206.5<br>38.0 | | | |
| 40.0 | | | DD<br>DS | 50/6" | DD = 112 pcf |
| 42.0 | Very stiff, olive-brown, sandy CLAY (CL); moist, low plasticity, sand is fine, micaceous, some iron oxide staining. | 202.5<br>42.0 | | | Clean sand |
| 45.0 | | | | 24 | Vapor probe set at 45.0 feet<br>Bentonite chips |
| 48.0 | Dense, brown, silty SAND (SM); moist, fine to medium, micaceous. | 196.5<br>48.0 | | | Clean sand |
| 50.0 | | | DD<br>CON<br>DS | 67 | DD = 116 pcf<br>Vapor probe set at 50.0 feet<br>Bentonite chips |
| 55.0 | medium dense; increased silt at 55.0 feet | | | 25 | Clean sand |

DRILLED BY: Martini Drilling LOGGED BY: AJA COMPLETED: 12/06/17

BORING METHOD: hollow-stem auger (see document text) BORING BIT DIAMETER: 8 inches

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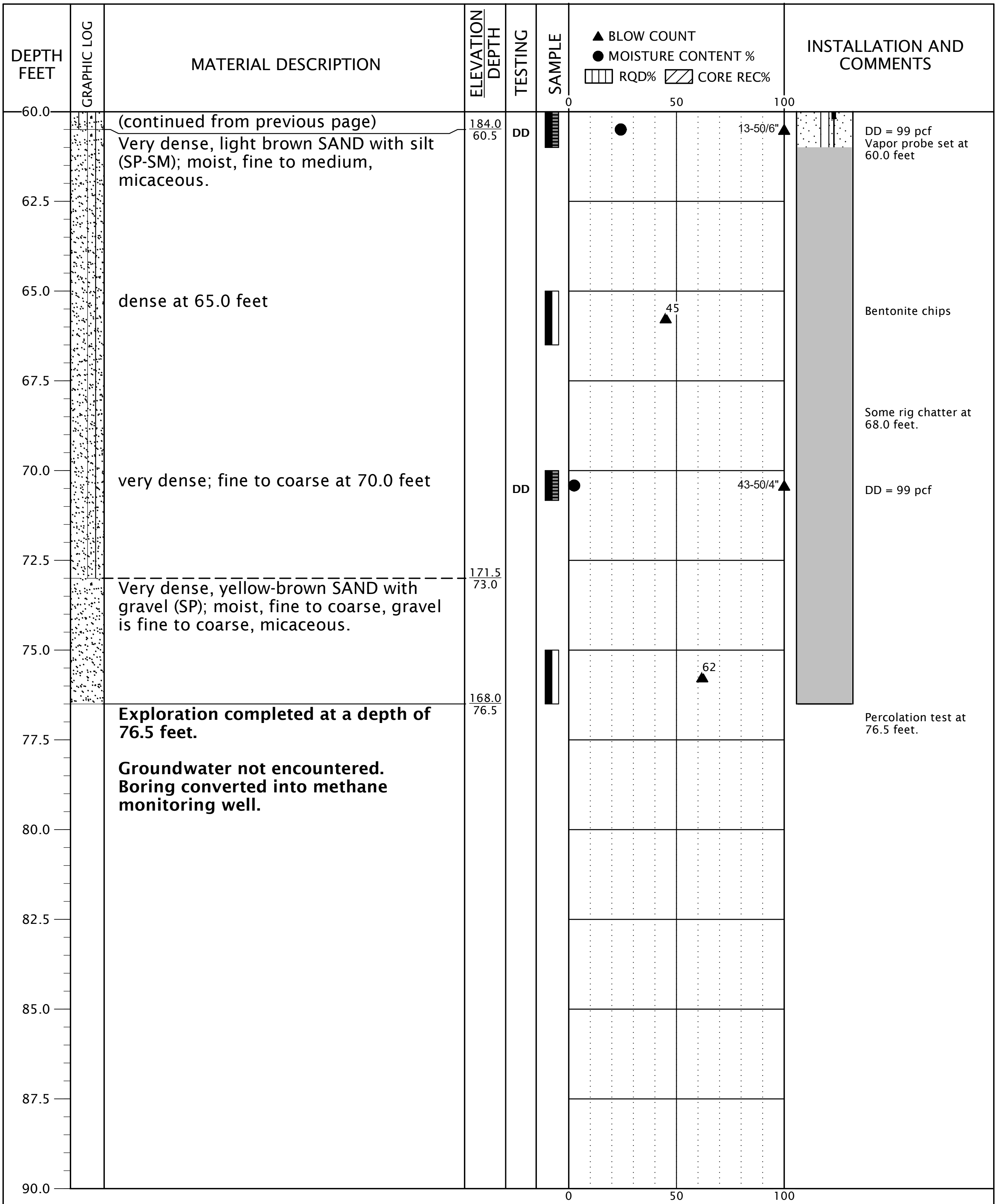
**BORING B-9**
(continued)

PROPOSED HIGH-RISE TOWER - SITE 3
LOS ANGELES, CA

**FIGURE A-3**



(continued from previous page)

Very dense, light brown SAND with silt (SP-SM); moist, fine to medium, micaceous.

dense at 65.0 feet

very dense; fine to coarse at 70.0 feet

Very dense, yellow-brown SAND with gravel (SP); moist, fine to coarse, gravel is fine to coarse, micaceous.

**Exploration completed at a depth of 76.5 feet.**

**Groundwater not encountered. Boring converted into methane monitoring well.**

DD = 99 pcf
Vapor probe set at 60.0 feet

Bentonite chips

Some rig chatter at 68.0 feet.

DD = 99 pcf

Percolation test at 76.5 feet.

DRILLED BY: Martini Drilling LOGGED BY: AJA COMPLETED: 12/06/17

BORING METHOD: hollow-stem auger (see document text) BORING BIT DIAMETER: 8 inches

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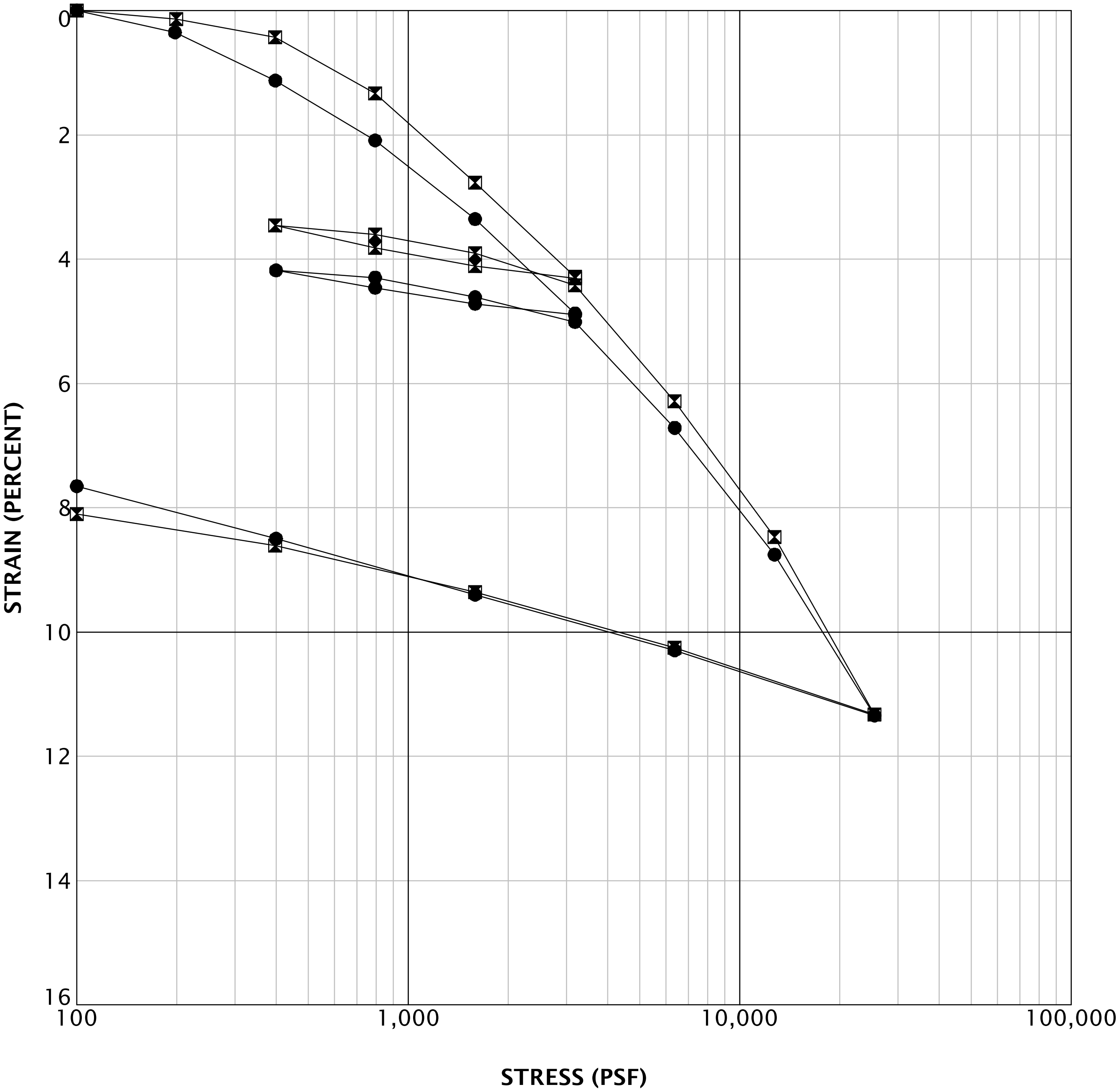
**BORING B-9**
(continued)

PROPOSED HIGH-RISE TOWER - SITE 3
LOS ANGELES, CA

**FIGURE A-3**



| KEY | EXPLORATION NUMBER | SAMPLE DEPTH (FEET) | USCS | MOISTURE CONTENT (PERCENT) | DRY DENSITY (PCF) | WATER ADDED AT (PSF) |
|---|---|---|---|---|---|---|
| ● | B-1 | 50.0 | ML | 17.1 | 109 | 3200 |
| ⧗ | B-2 | 25.0 | ML | 19.5 | 106 | 3200 |

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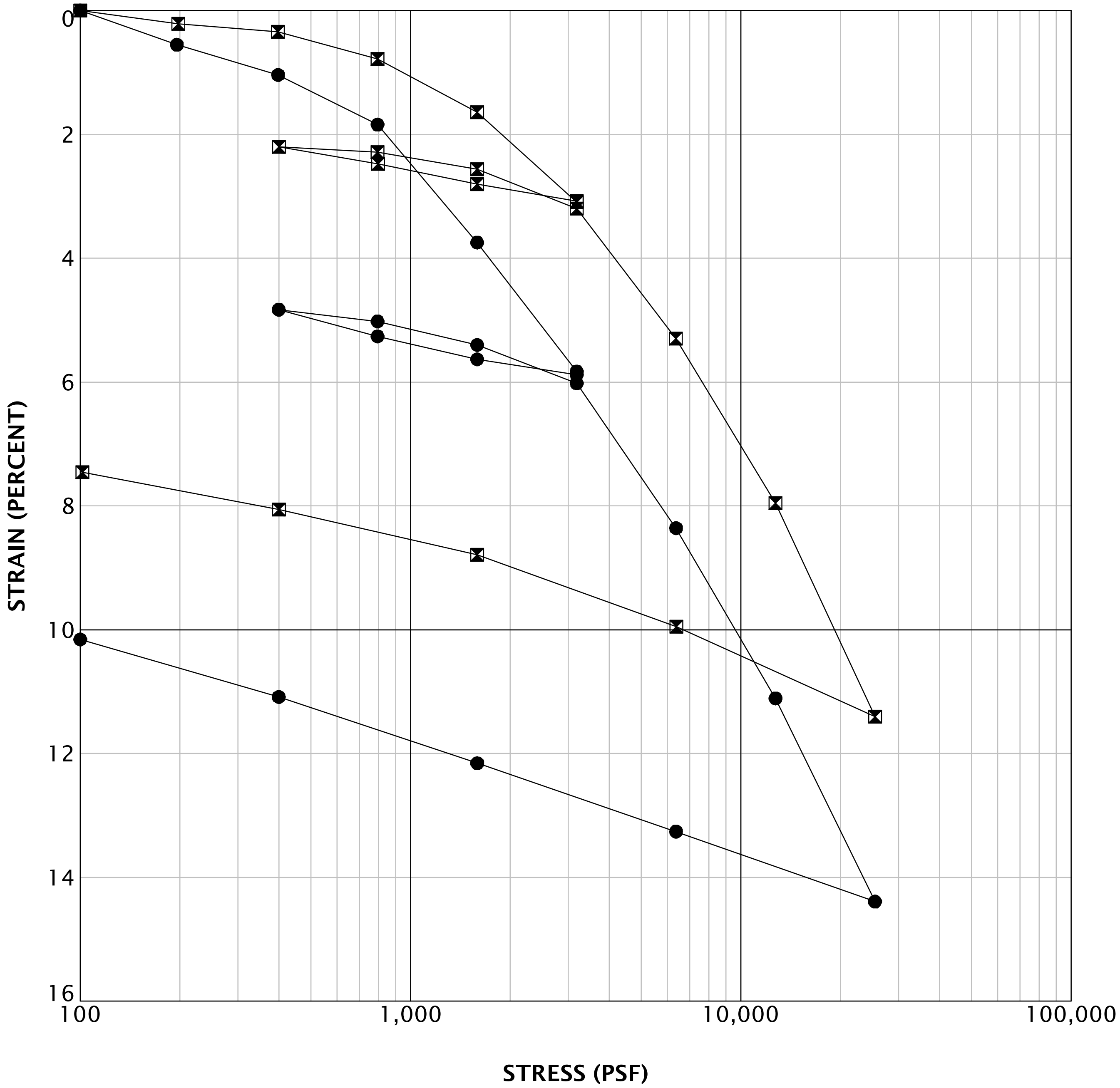
| | | |
|---|---|---|
| MERUELO-13-01 | **CONSOLIDATION TEST RESULTS** | |
| JUNE 2007 | PROPOSED 1100 SOUTH OLIVE STREET TOWER<br>LOS ANGELES, CA | **FIGURE A-7** |



| KEY | EXPLORATION NUMBER | SAMPLE DEPTH (FEET) | USCS | MOISTURE CONTENT (PERCENT) | DRY DENSITY (PCF) | WATER ADDED AT (PSF) |
|---|---|---|---|---|---|---|
| ● | B-3 | 30.0 | SP | 12.9 | 109 | 3200 |
| ☒ | B-3 | 45.0 | ML | 15.4 | 111 | 3200 |

CONSOL_STRAIN_100K_H20_ADDED_USCS - ANAH MERUELO-13-01.GPJ GEODESIGN.GDT PRINT DATE: 5/31/07:KYK

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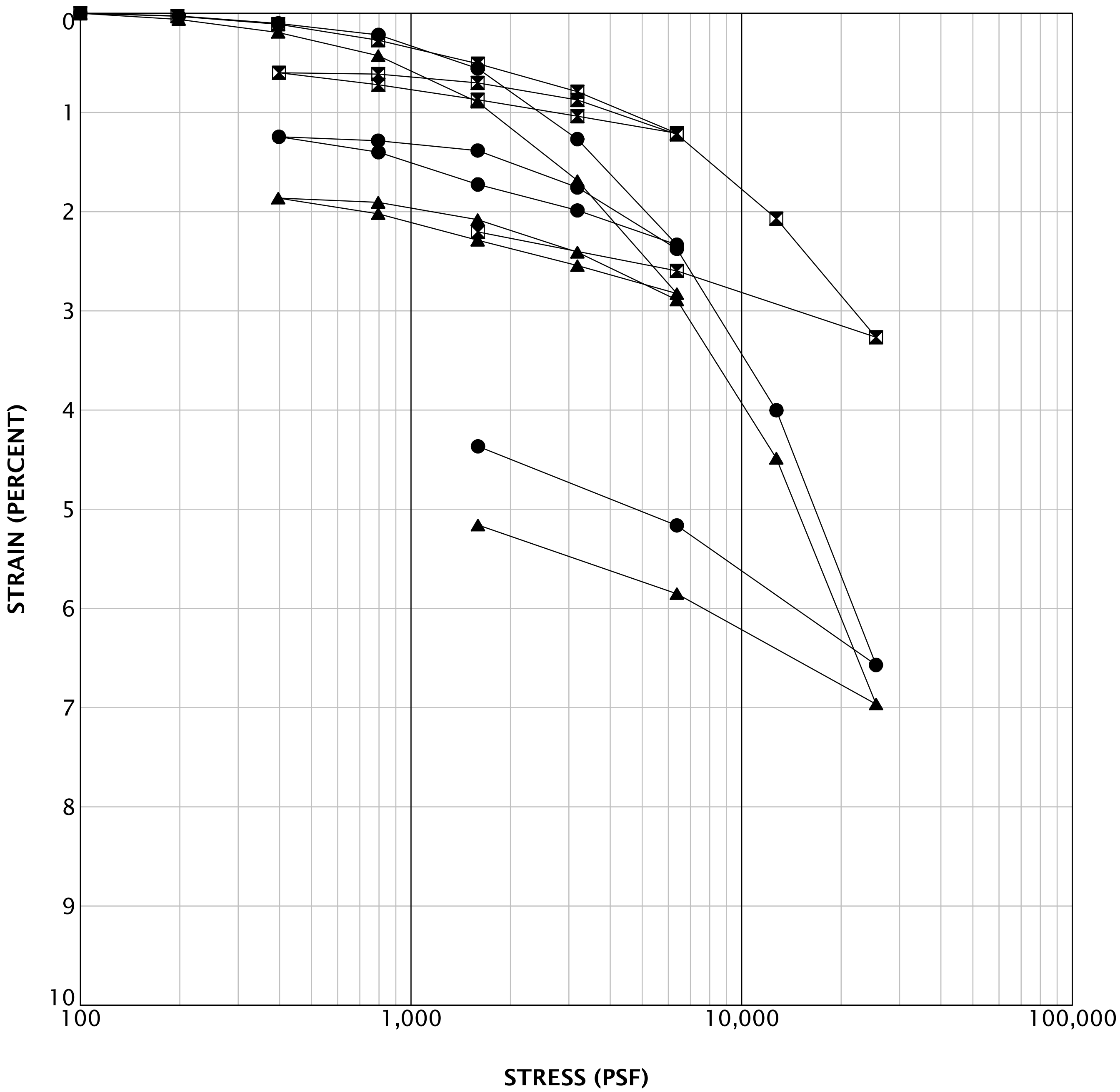
MERUELO-13-01

JUNE 2007

**CONSOLIDATION TEST RESULTS**
(continued)

PROPOSED 1100 SOUTH OLIVE STREET TOWER
LOS ANGELES, CA

**FIGURE A-7**

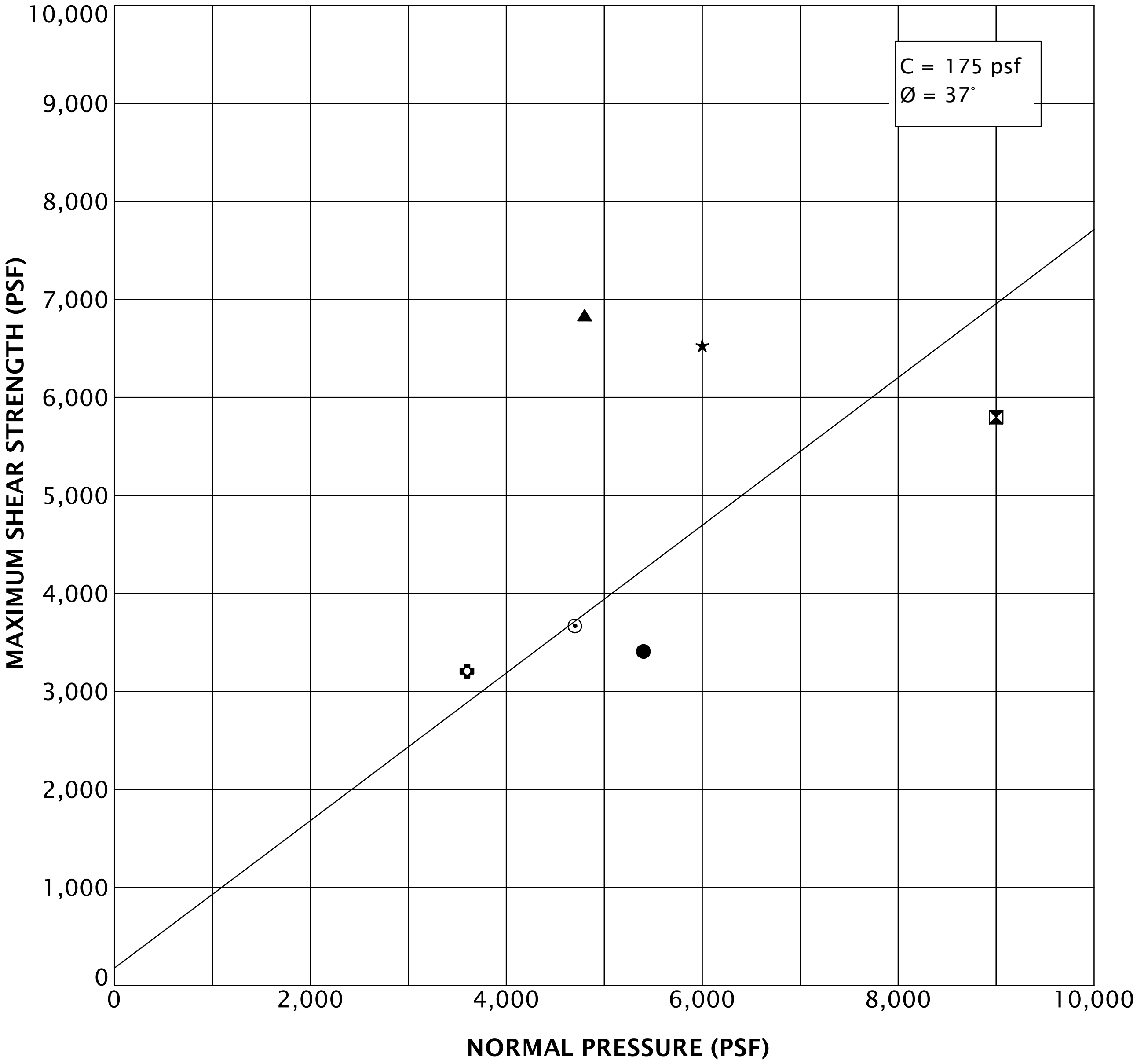


| KEY | EXPLORATION NUMBER | SAMPLE DEPTH (FEET) | MOISTURE CONTENT (PERCENT) | DRY DENSITY (PCF) |
|---|---|---|---|---|
| ● | B-7 | 45.0 | 20 | 110 |
| ☒ | B-8 | 50.0 | 14 | 117 |
| ▲ | B-9 | 50.0 | 15 | 116 |

## CONSOLIDATION TEST RESULTS

PROPOSED HIGH-RISE TOWER - SITE 3
LOS ANGELES, CA

**FIGURE A-4**



| KEY | EXPLORATION NUMBER | SAMPLE DEPTH (FEET) | USCS | MOISTURE CONTENT (PERCENT) | DRY DENSITY (PCF) | SOAKED |
|---|---|---|---|---|---|---|
| ● | B-1 | 45.0 | SM | 19.1 | 98 | YES |
| ⊠ | B-1 | 75.0 | SP | 20.1 | 109 | YES |
| ▲ | B-2 | 40.0 | SM | 18.4 | 108 | YES |
| ★ | B-2 | 50.0 | SM | 17.9 | 105 | YES |
| ⊙ | B-2 | 60.0 | SP | 8.1 | 94 | YES |
| ✿ | B-3 | 30.0 | SP | 13.0 | 98 | YES |

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MERUELO-13-01

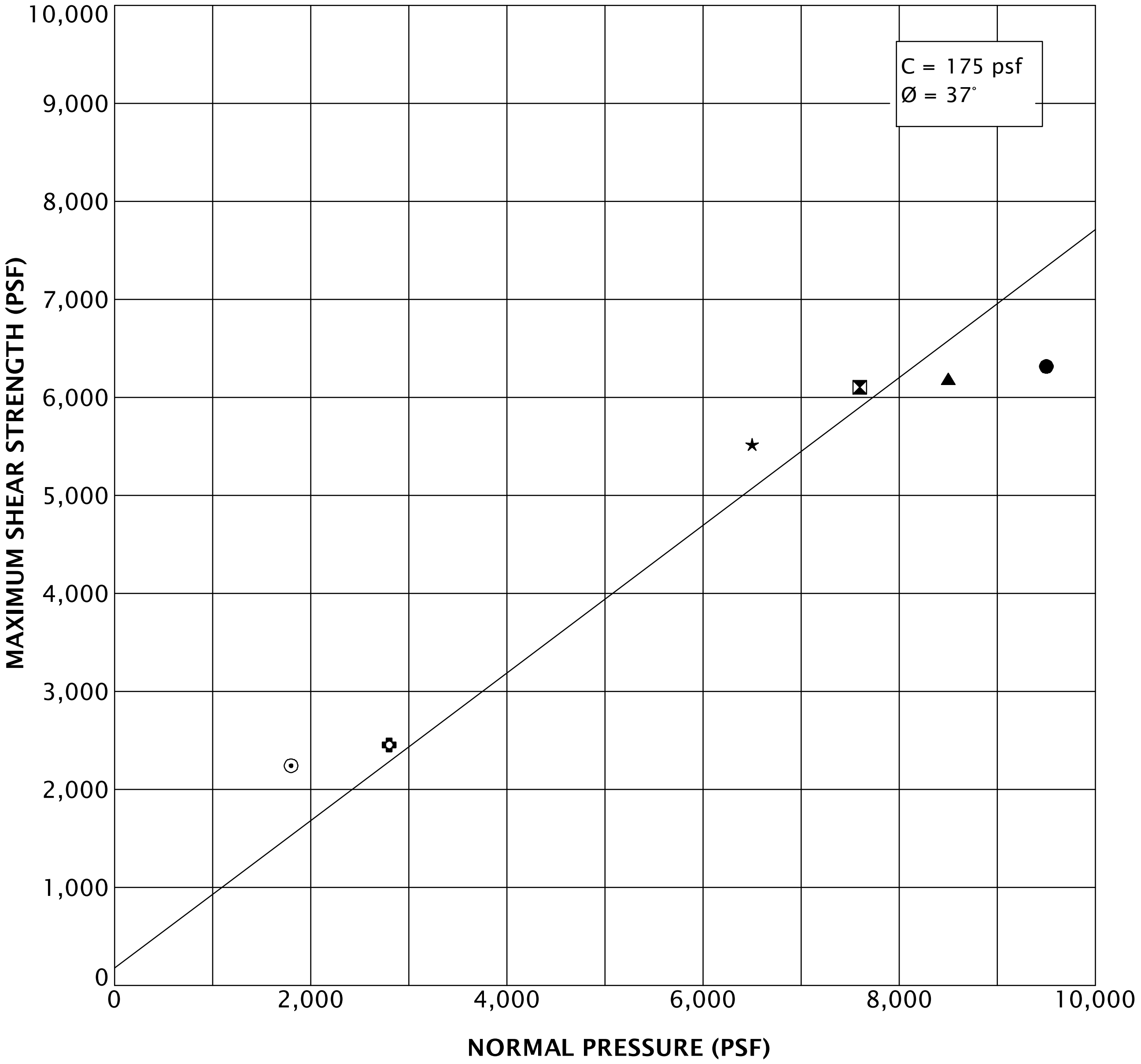
**DIRECT SHEAR TEST RESULTS (COARSE-GRAINED SOILS)**

JUNE 2007

PROPOSED 1100 SOUTH OLIVE STREET TOWER
LOS ANGELES, CA

**FIGURE A-8**

DIRECT_SHEAR_FAIL_ENV_USCS_TYPE_TITLE_6 MERUELO-13-01.GPJ GEODESIGN.GDT PRINT DATE: 6/1/07:KYK



| KEY | EXPLORATION NUMBER | SAMPLE DEPTH (FEET) | USCS | MOISTURE CONTENT (PERCENT) | DRY DENSITY (PCF) | SOAKED |
|---|---|---|---|---|---|---|
| ● | B-3 | 100.0 | SP-SM | 28.3 | 89 | YES |
| ⊠ | B-4 | 55.0 | SM | 18.3 | 105 | YES |
| ▲ | B-4 | 95.0 | SM | 23.8 | 98 | YES |
| ★ | B-5 | 50.0 | SM | 7.9 | 120 | YES |
| ⊙ | B-6 | 15.0 | SW | 7.5 | 105 | YES |
| ✿ | B-6 | 20.0 | SP | 5.4 | 104 | YES |

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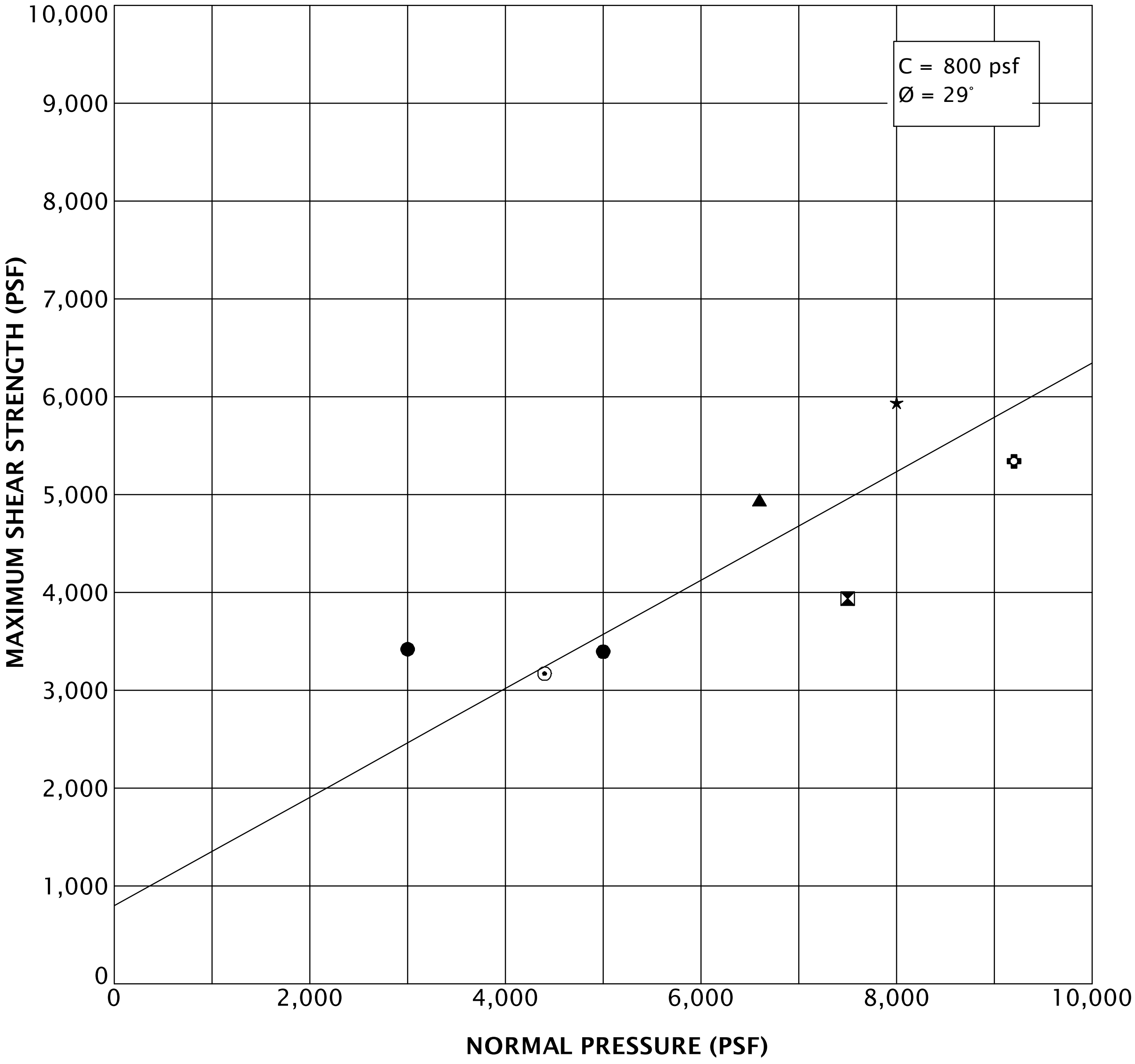
JUNE 2007

## DIRECT SHEAR TEST RESULTS (COARSE-GRAINED SOILS)

(continued)

PROPOSED 1100 SOUTH OLIVE STREET TOWER
LOS ANGELES, CA

**FIGURE A-8**



| KEY | EXPLORATION NUMBER | SAMPLE DEPTH (FEET) | USCS | MOISTURE CONTENT (PERCENT) | DRY DENSITY (PCF) | SOAKED |
|---|---|---|---|---|---|---|
| ● | B-2 | 25.0 | ML | 15.7 | 110 | YES |
| ⊠ | B-3 | 50.0 | ML | 19.0 | 101 | YES |
| ▲ | B-3 | 55.0 | ML | 14.9 | 107 | YES |
| ★ | B-3 | 90.0 | ML | 24.1 | 92 | YES |
| ⊙ | B-4 | 45.0 | ML | 16.5 | 104 | YES |
| ✿ | B-5 | 60.0 | ML | 28.3 | 94 | YES |

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MERUELO-13-01

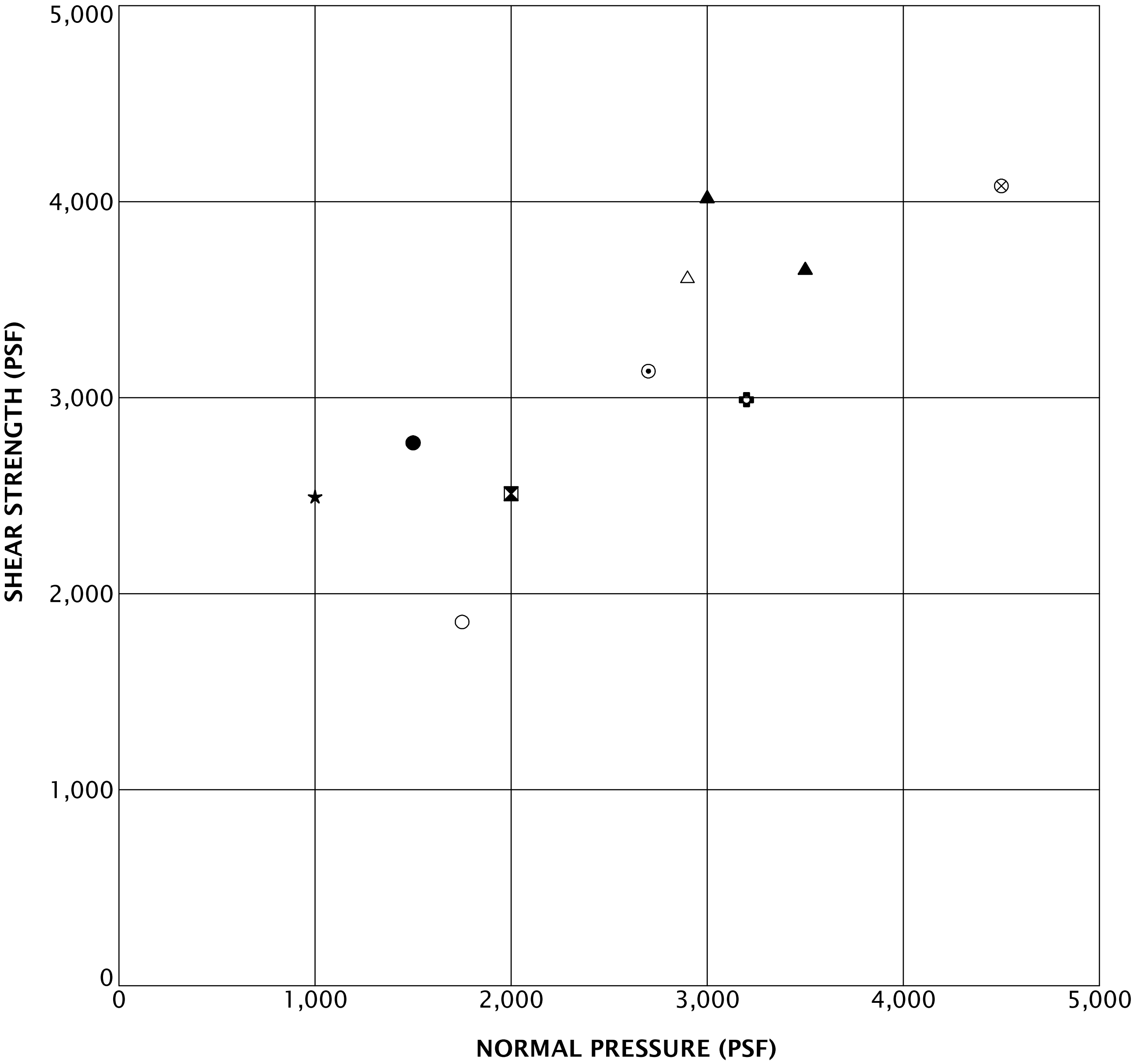
## DIRECT SHEAR TEST RESULTS (FINE-GRAINED SOILS)

JUNE 2007

PROPOSED 1100 SOUTH OLIVE STREET TOWER
LOS ANGELES, CA

**FIGURE A-9**

DIRECT_SHEAR_FAIL_ENV_USCS_TYPE_TITLE_6 MERUELO-13-01.GPJ GEODESIGN.GDT PRINT DATE: 5/31/07:KYK



| KEY | EXPLORATION NUMBER | SAMPLE DEPTH (FEET) | MOISTURE CONTENT (PERCENT) | DRY DENSITY (PCF) | SOAKED |
|---|---|---|---|---|---|
| ● | B-7 | 10.0 | 4 | 119 | YES |
| ⧗ | B-7 | 20.0 | 4 | 114 | YES |
| ▲ | B-7 | 45.0 | 20 | 107 | YES |
| ★ | B-8 | 5.0 | 5 | 136 | YES |
| ⊙ | B-8 | 30.0 | 12 | 117 | YES |
| ✤ | B-8 | 50.0 | 14 | 116 | YES |
| ○ | B-9 | 17.0 | 6 | 105 | YES |
| △ | B-9 | 40.0 | 4 | 112 | YES |
| ⊗ | B-9 | 50.0 | 15 | 118 | YES |
| | | | | | |

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**DIRECT SHEAR TEST RESULTS**

JUNE 2018

PROPOSED HIGH-RISE TOWER - SITE 3
LOS ANGELES, CA

**FIGURE A-5**

DIRECT_SHEAR_FAIL_ENV_NO BOX DTLASITE3-1-01-B7_9.GPJ GEODESIGN.GDT PRINT DATE: 6/11/18: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 (PERCENT) | PLASTIC LIMIT (PERCENT) | PLASTICITY INDEX (PERCENT) |
| B-1 | 5.0 | 239.0 | 8.4 | | | | | | | |
| B-1 | 10.0 | 234.0 | 3.2 | | | | | | | |
| B-1 | 15.0 | 229.0 | 3.1 | | | | | | | |
| B-1 | 20.0 | 224.0 | 4.4 | | | | | | | |
| B-1 | 25.0 | 219.0 | 14.5 | 105 | | | | | | |
| B-1 | 35.0 | 209.0 | 5.1 | | | | | | | |
| B-1 | 40.0 | 204.0 | 8.5 | | | | | | | |
| B-1 | 45.0 | 199.0 | 19.1 | 98 | | | | | | |
| B-1 | 50.0 | 194.0 | 17.1 | 109 | | | | | | |
| B-1 | 55.0 | 189.0 | 23.8 | 101 | | | | | | |
| B-1 | 60.0 | 184.0 | 10.0 | 119 | | | | | | |
| B-1 | 65.0 | 179.0 | 26.7 | 95 | | | | | | |
| B-1 | 70.0 | 174.0 | 1.9 | | | | | | | |
| B-1 | 75.0 | 169.0 | 20.1 | 109 | | | | | | |
| B-2 | 2.0 | 242.0 | 17.8 | 109 | | | | | | |
| B-2 | 5.0 | 239.0 | 4.1 | | | | | | | |
| B-2 | 10.0 | 234.0 | 4.1 | | | | | | | |
| B-2 | 15.0 | 229.0 | 9.4 | 105 | | | | | | |
| B-2 | 20.0 | 224.0 | 5.3 | 106 | | | | | | |
| B-2 | 25.0 | 219.0 | 15.7 | 109 | | | | | | |
| B-2 | 30.0 | 214.0 | 6.7 | 113 | | | | | | |
| B-2 | 35.0 | 209.0 | 17.2 | 111 | | | | | | |
| B-2 | 40.0 | 204.0 | 18.4 | 108 | | | | | | |
| B-2 | 50.0 | 194.0 | 17.9 | 105 | | | | | | |
| B-2 | 60.0 | 184.0 | 8.1 | 94 | | | | | | |
| B-2 | 65.0 | 179.0 | 1.8 | 96 | | | | | | |
| B-2 | 70.0 | 174.0 | 2.9 | | | | | | | |

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**SUMMARY OF LABORATORY DATA**

PROPOSED 1100 SOUTH OLIVE STREET TOWER
LOS ANGELES, CA

**FIGURE A-10**

| 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 (PERCENT) | PLASTIC LIMIT (PERCENT) | PLASTICITY INDEX (PERCENT) |
| B-2 | 75.0 | 169.0 | 3.9 | | | | | | | |
| B-3 | 5.0 | 239.0 | 8.8 | 118 | | | | | | |
| B-3 | 15.0 | 229.0 | 4.9 | 108 | | | | | | |
| B-3 | 20.0 | 224.0 | 4.8 | | | | | | | |
| B-3 | 25.0 | 219.0 | 4.7 | | | | | | | |
| B-3 | 30.0 | 214.0 | 13.0 | 98 | | | | | | |
| B-3 | 35.0 | 209.0 | 4.0 | | | | | | | |
| B-3 | 40.0 | 204.0 | 5.5 | 108 | | | | | | |
| B-3 | 45.0 | 199.0 | 18.1 | 110 | | | | | | |
| B-3 | 50.0 | 194.0 | 19.0 | 101 | | | | | | |
| B-3 | 55.0 | 189.0 | 14.9 | 107 | | | | | | |
| B-3 | 60.0 | 184.0 | 25.7 | 99 | | | | | | |
| B-3 | 65.0 | 179.0 | 2.2 | | | | | | | |
| B-3 | 70.0 | 174.0 | 2.3 | | | | | | | |
| B-3 | 75.0 | 169.0 | 2.7 | | | | | | | |
| B-3 | 85.0 | 159.0 | 2.7 | | | | | | | |
| B-3 | 90.0 | 154.0 | 24.1 | 92 | | | | | | |
| B-3 | 95.0 | 149.0 | 5.5 | 91 | | | | | | |
| B-3 | 100.0 | 144.0 | 28.3 | 89 | | | | | | |
| B-3 | 105.0 | 139.0 | 3.9 | | | | | | | |
| B-3 | 110.0 | 134.0 | 8.0 | 90 | | | | | | |
| B-3 | 115.0 | 129.0 | 13.8 | 92 | | | | | | |
| B-3 | 120.0 | 124.0 | 3.5 | | | | | | | |
| B-4 | 2.0 | 241.0 | 10.6 | 102 | | | | | | |
| B-4 | 5.0 | 238.0 | 8.6 | 104 | | | | | | |
| B-4 | 10.0 | 233.0 | 2.8 | | | | | | | |
| B-4 | 15.0 | 228.0 | 2.8 | | | | | | | |

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**SUMMARY OF LABORATORY DATA**
(continued)

PROPOSED 1100 SOUTH OLIVE STREET TOWER
LOS ANGELES, CA

**FIGURE A-10**

LAB SUMMARY - ANAHEIM MERUELO-13-01.GPJ GEODESIGN.GDT PRINT DATE: 5/31/07:KYK

| 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 (PERCENT) | PLASTIC LIMIT (PERCENT) | PLASTICITY INDEX (PERCENT) |
| B-4 | 20.0 | 223.0 | 5.7 | | | | | | | |
| B-4 | 25.0 | 218.0 | 7.0 | | | | | | | |
| B-4 | 30.0 | 213.0 | 6.2 | | | | | | | |
| B-4 | 35.0 | 208.0 | 8.4 | | | | | | | |
| B-4 | 40.0 | 203.0 | 6.9 | | | | | | | |
| B-4 | 45.0 | 198.0 | 16.5 | 104 | | | | | | |
| B-4 | 55.0 | 188.0 | 18.3 | 105 | | | | | | |
| B-4 | 60.0 | 183.0 | 6.9 | 94 | | | | | | |
| B-4 | 65.0 | 178.0 | 2.4 | 96 | | | | | | |
| B-4 | 75.0 | 168.0 | 3.5 | | | | | | | |
| B-4 | 80.0 | 163.0 | 3.5 | 100 | | | | | | |
| B-4 | 90.0 | 153.0 | 2.1 | | | | | | | |
| B-4 | 95.0 | 148.0 | 23.8 | 98 | | | | | | |
| B-4 | 105.0 | 138.0 | 1.3 | | | | | | | |
| B-4 | 125.0 | 118.0 | 1.2 | | | | | | | |
| B-5 | 5.0 | 237.0 | 3.9 | | | | | | | |
| B-5 | 10.0 | 232.0 | 18.3 | | | | | | | |
| B-5 | 15.0 | 227.0 | 3.9 | | | | | | | |
| B-5 | 20.0 | 222.0 | 6.2 | | | | | | | |
| B-5 | 25.0 | 217.0 | 6.6 | | | | | | | |
| B-5 | 35.0 | 207.0 | 5.7 | 112 | | | | | | |
| B-5 | 40.0 | 202.0 | 4.5 | | | | | | | |
| B-5 | 45.0 | 197.0 | 17.3 | 108 | | | | | | |
| B-5 | 50.0 | 192.0 | 7.9 | 120 | | | | | | |
| B-5 | 55.0 | 187.0 | 23.0 | 102 | | | | | | |
| B-5 | 60.0 | 182.0 | 28.3 | 94 | | | | | | |
| B-5 | 70.0 | 172.0 | 2.3 | | | | | | | |

MERUELO-13-01

JUNE 2007

**SUMMARY OF LABORATORY DATA**
(continued)

PROPOSED 1100 SOUTH OLIVE STREET TOWER
LOS ANGELES, CA

**FIGURE A-10**

| 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 (PERCENT) | PLASTIC LIMIT (PERCENT) | PLASTICITY INDEX (PERCENT) |
| B-6 | 2.0 | 240.0 | 12.2 | 108 | | | | | | |
| B-6 | 5.0 | 237.0 | 3.6 | | | | | | | |
| B-6 | 10.0 | 232.0 | 2.5 | | | | | | | |
| B-6 | 15.0 | 227.0 | 7.5 | 105 | | | | | | |
| B-6 | 20.0 | 222.0 | 5.4 | 104 | | | | | | |
| B-6 | 25.0 | 217.0 | 7.8 | 104 | | | | | | |
| B-6 | 30.0 | 212.0 | 4.2 | | | | | | | |
| B-6 | 35.0 | 207.0 | 6.2 | | | | | | | |
| B-6 | 40.0 | 202.0 | 6.8 | | | | | | | |
| B-6 | 45.0 | 197.0 | 16.2 | 102 | | | | | | |
| B-6 | 55.0 | 187.0 | 22.0 | 98 | | | | | | |
| B-6 | 60.0 | 182.0 | 17.3 | 100 | | | | | | |
| B-6 | 65.0 | 177.0 | 1.6 | | | | | | | |
| B-6 | 75.0 | 167.0 | 1.5 | | | | | | | |

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**SUMMARY OF LABORATORY DATA**
(continued)

PROPOSED 1100 SOUTH OLIVE STREET TOWER
LOS ANGELES, CA

**FIGURE A-10**

| 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-7 | 5.0 | | 7 | 126 | | | | | | |
| B-7 | 10.0 | | 4 | 119 | | | | | | |
| B-7 | 15.0 | | 5 | 110 | | | | | | |
| B-7 | 20.0 | | 4 | 114 | | | | | | |
| B-7 | 25.0 | | 9 | 126 | | | | | | |
| B-7 | 35.0 | | 6 | 94 | | | | | | |
| B-7 | 45.0 | | 20 | 110 | | | | | | |
| B-7 | 55.0 | | 10 | 101 | | | | | | |
| B-7 | 65.0 | | 17 | 101 | | | | | | |
| B-7 | 75.0 | | 3 | 99 | | | | | | |
| B-8 | 5.0 | | 5 | 136 | | | | | | |
| B-8 | 15.0 | | 8 | 122 | | | | | | |
| B-8 | 20.0 | | 10 | 120 | | | | | | |
| B-8 | 25.0 | | 10 | 127 | | | | | | |
| B-8 | 30.0 | | 12 | 117 | | | | | | |
| B-8 | 50.0 | | 14 | 117 | | | | | | |
| B-8 | 60.0 | | 24 | 102 | | | | | | |
| B-8 | 70.0 | | 14 | 111 | | | | | | |
| B-8 | 90.0 | | 22 | 104 | | | | | | |
| B-8 | 100.0 | | 16 | 112 | | | | | | |
| B-8 | 130.0 | | 10 | 103 | | | | | | |
| B-8 | 150.0 | | 19 | 111 | | | | | | |
| B-8 | 160.0 | | 20 | 107 | | | | | | |
| B-8 | 190.0 | | 27 | 98 | | | | | | |
| B-9 | 5.0 | | 7 | 114 | | | | | | |
| B-9 | 10.0 | | 2 | 106 | | | | | | |
| B-9 | 17.0 | | 6 | 105 | | | | | | |

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DTLASITE3-1-01

JUNE 2018

## SUMMARY OF LABORATORY DATA

PROPOSED HIGH-RISE TOWER - SITE 3
LOS ANGELES, CA

**FIGURE A-6**

| 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-9 | 20.0 | | 6 | 109 | | | | | | |
| B-9 | 30.0 | | 5 | 109 | | | | | | |
| B-9 | 40.0 | | 4 | 112 | | | | | | |
| B-9 | 50.0 | | 15 | 116 | | | | | | |
| B-9 | 60.0 | | 24 | 99 | | | | | | |
| B-9 | 70.0 | | 3 | 99 | | | | | | |

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DTLASITE3-1-01

JUNE 2018

**SUMMARY OF LABORATORY DATA**
(continued)

PROPOSED HIGH-RISE TOWER - SITE 3
LOS ANGELES, CA

**FIGURE A-6**

# APPENDIX B

## Prior Percolation Test Results

**Project Number:** DTLASite3-1-01
**Boring Number:** B-7
**Boring Diameter** 8 in
**Hours Pre-Soak:** <1
**Date Pre-Soak Initiated:** 12/5/2017
**Depth of Bottom (Below Grade):** 75.80 ft
**Casing Stick-up:** 0 ft
**Total Length of Well Casing** 75.80 ft
**Name of Tester:** AJA
**Date Tested:** 12/5/2017
**Method to Prevent Caving:** Well Casing
**Checked by:** **Date:**

| Status | **t-intial** | **t-final** | **Δt (min)** | **Δt (hours)** | **Initial Depth to Water (ft)** | **Final Depth to Water (ft)** | **Initial Head (ft)** | **Final Head (ft)** | **Water Level Drop (ft)** | **Water Level Drop (in)** | **Measured Infiltration Rate (in/hr)** | **CF** | **Design Infiltration Rate (in/hr)[1]** |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Presoak | 9:54 | 9:55 | 1.50 | 0.03 | 74.80 | 75.80 | 1.00 | 0.00 | 1.00 | 12.0 | 480.0 | - | |
| | 9:57 | 10:01 | 3.75 | 0.06 | 74.80 | 75.80 | 1.00 | 0.00 | 1.00 | 12.0 | 192.0 | - | |
| Time Trial Tests | 10:02 | 10:07 | 5.40 | 0.09 | 74.80 | 75.80 | 1.00 | 0.00 | 1.00 | 12.0 | 133.3 | - | |
| | 10:08 | 10:16 | 7.18 | 0.12 | 74.80 | 75.80 | 1.00 | 0.00 | 1.00 | 12.0 | 100.3 | - | |
| | 10:17 | 10:25 | 7.75 | 0.13 | 74.80 | 75.80 | 1.00 | 0.00 | 1.00 | 12.0 | 92.9 | - | |
| | 10:28 | 10:37 | 8.25 | 0.14 | 74.80 | 75.80 | 1.00 | 0.00 | 1.00 | 12.0 | 87.3 | - | |
| | 10:37 | 10:47 | 10.00 | 0.17 | 74.80 | 75.67 | 1.00 | 0.13 | 0.87 | 10.4 | 62.6 | - | |
| | 10:48 | 10:58 | 10.00 | 0.17 | 74.80 | 75.65 | 1.00 | 0.15 | 0.85 | 10.2 | 61.2 | - | |
| | 11:00 | 11:10 | 10.00 | 0.17 | 74.80 | 75.67 | 1.00 | 0.13 | 0.87 | 10.4 | 62.6 | - | |
| | 11:11 | 11:21 | 10.00 | 0.17 | 74.80 | 75.65 | 1.00 | 0.15 | 0.85 | 10.2 | 61.2 | - | |
| | 11:22 | 11:32 | 10.00 | 0.17 | 74.80 | 75.70 | 1.00 | 0.10 | 0.90 | 10.8 | 64.8 | 3.0 | 21.0 |

notes:

1. Average of the stabilized rate over the last three consecutive readings

**Project Number:** DTLASite3-1-01
**Boring Number:** B-9
**Boring Diameter** 8 in
**Hours Pre-Soak:** <1
**Date Pre-Soak Initiated:** 12/6/2017
**Depth of Bottom (Below Grade):** 76.50 ft
**Casing Stick-up:** 0 ft
**Total Length of Well Casing** 76.50 ft
**Name of Tester:** AJA
**Date Tested:** 12/6/2017
**Method to Prevent Caving:** Well Casing
**Checked by:** **Date:**

| Status | t-intial | t-final | Δt (min) | Δt (hours) | Initial Depth to Water (ft) | Final Depth to Water (ft) | Initial Head (ft) | Final Head (ft) | Water Level Drop (ft) | Water Level Drop (in) | Measured Infiltration Rate (in/hr) | CF | Design Infiltration Rate (in/hr)[1] |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Presoak---> | 9:53 | 9:53 | 0.50 | 0.01 | 75.50 | 76.50 | 1.00 | 0.00 | 1.00 | 12.0 | 1440.0 | | |
| | 9:54 | 9:55 | 2.50 | 0.04 | 75.50 | 76.50 | 1.00 | 0.00 | 1.00 | 12.0 | 288.0 | | |
| Time Trial---> | 9:57 | 10:05 | 8.50 | 0.14 | 75.50 | 76.50 | 1.00 | 0.00 | 1.00 | 12.0 | 84.7 | | |
| Tests---> | 10:06 | 10:16 | 10.00 | 0.17 | 75.50 | 76.45 | 1.00 | 0.05 | 0.95 | 11.4 | 68.4 | | |
| | 10:17 | 10:27 | 10.00 | 0.17 | 75.50 | 76.35 | 1.00 | 0.15 | 0.85 | 10.2 | 61.2 | | |
| | 10:28 | 10:38 | 10.00 | 0.17 | 75.50 | 76.25 | 1.00 | 0.25 | 0.75 | 9.0 | 54.0 | | |
| | 10:40 | 10:50 | 10.00 | 0.17 | 75.50 | 76.20 | 1.00 | 0.30 | 0.70 | 8.4 | 50.4 | | |
| | 10:51 | 11:01 | 10.00 | 0.17 | 75.50 | 76.15 | 1.00 | 0.35 | 0.65 | 7.8 | 46.8 | | |
| | 11:02 | 11:12 | 10.00 | 0.17 | 75.50 | 76.14 | 1.00 | 0.36 | 0.64 | 7.7 | 46.1 | | |
| | 11:14 | 11:24 | 10.00 | 0.17 | 75.50 | 76.10 | 1.00 | 0.40 | 0.60 | 7.2 | 43.2 | | |
| | 11:27 | 11:37 | 10.00 | 0.17 | 75.50 | 76.12 | 1.00 | 0.38 | 0.62 | 7.4 | 44.6 | 3.0 | 14.9 |

notes:

1. Average of the stabilized rate over the last three consecutive readings

# APPENDIX C

## Prior P-S Suspension Seismic Velocity Logging

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geophysical services

# DTLA SITE 3-1-01

# BOREHOLE B-8

# SUSPENSION PS VELOCITIES

**December 06, 2017**

**Report 17436-01 rev 0**

# DTLA SITE 3-1-01

# BOREHOLE B-8

# SUSPENSION PS VELOCITIES

**Prepared for**

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**Project 16139**

**December 06, 2017**

**Report 17436-01 rev 0**

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# APPENDICES

# INTRODUCTION

**GEO***Vision* acquired borehole geophysical data in one borehole at 1100 Olive St. in Los Angeles, California. The work was performed for GeoDesign, Inc. Data, analysis and report were reviewed by a **GEO***Vision* Professional Geophysicist or Engineer.

# SCOPE OF WORK

This report presents results of Suspension PS velocity data acquired in one borehole on November 30, 2017 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 PS 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 and compressional-wave source, 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 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 at least every twelve months using a NIST traceable frequency source and counter, as presented in Appendix B.

# MEASUREMENT PROCEDURES

## Suspension Velocity Measurement Procedures

Borehole B-8 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 file accompanies 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:

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

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.

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

# RESULTS

## Suspension Velocity Results

Suspension R1-R2 P- and $S_H$-wave velocities for borehole B-8 are plotted in Figure 4 and 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 displayed in this figure 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. Drilling fluid could not be maintained above 6.5 feet, thus data could not be acquired above this depth.

Overall, Suspension PS velocity data quality is judged on 5 criteria, as summarized below.

| | Criteria | B-08 |
|---|---|---|
| **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<br>Saturation in B-08 occurs at about 145 ft. |
| **4** | Clarity of P-wave and $S_H$-wave onset, as well as damping of later oscillations. | Generally S-wave and P-wave onsets were clear with P-wave arrivals slightly more difficult to identify near ground surface. |
| **5** | Consistency of profile between adjacent borings, if available. | NA |

## Quality Assurance

These borehole geophysical measurements were performed using industry-standard or better methods for measurements and analysis. 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

J Jordan

12/06/2017

J. Jonathan Jordan
Sr Staff Geophysicist
**GEO***Vision* Geophysical Services

Date

Reviewed and approved by

VICTOR GONZALEZ
PGp 1074

12/06/2017

Victor Gonzalez
California Professional Geophysicist, PGp. 1074
**GEO***Vision* Geophysical Services

Date

∗ 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 DESIGNATION | DATES LOGGED | COORDINATES | | ELEVATION (TOP OF WELL CASING) (1) (FEET) |
|---|---|---|---|---|
| | | LATITUDE | LONGITUDE | |
| B-8 | 11/30/2017 | 34.04038 | -118.26085 | |

(1) Coordinates estimated from phone GPS. Elevation not provided

## Table 2. Logging dates and depth ranges

| BOREHOLE NUMBER | TOOL AND RUN NUMBER | DEPTH RANGE (FEET) | OPEN HOLE (FEET) | SAMPLE INTERVAL (FEET) | DATE LOGGED |
|---|---|---|---|---|---|
| B-8 | SUSPENSION DOWN01 | 1.6 – 187.0 | 200 | 1.6 | 11/30/2017 |

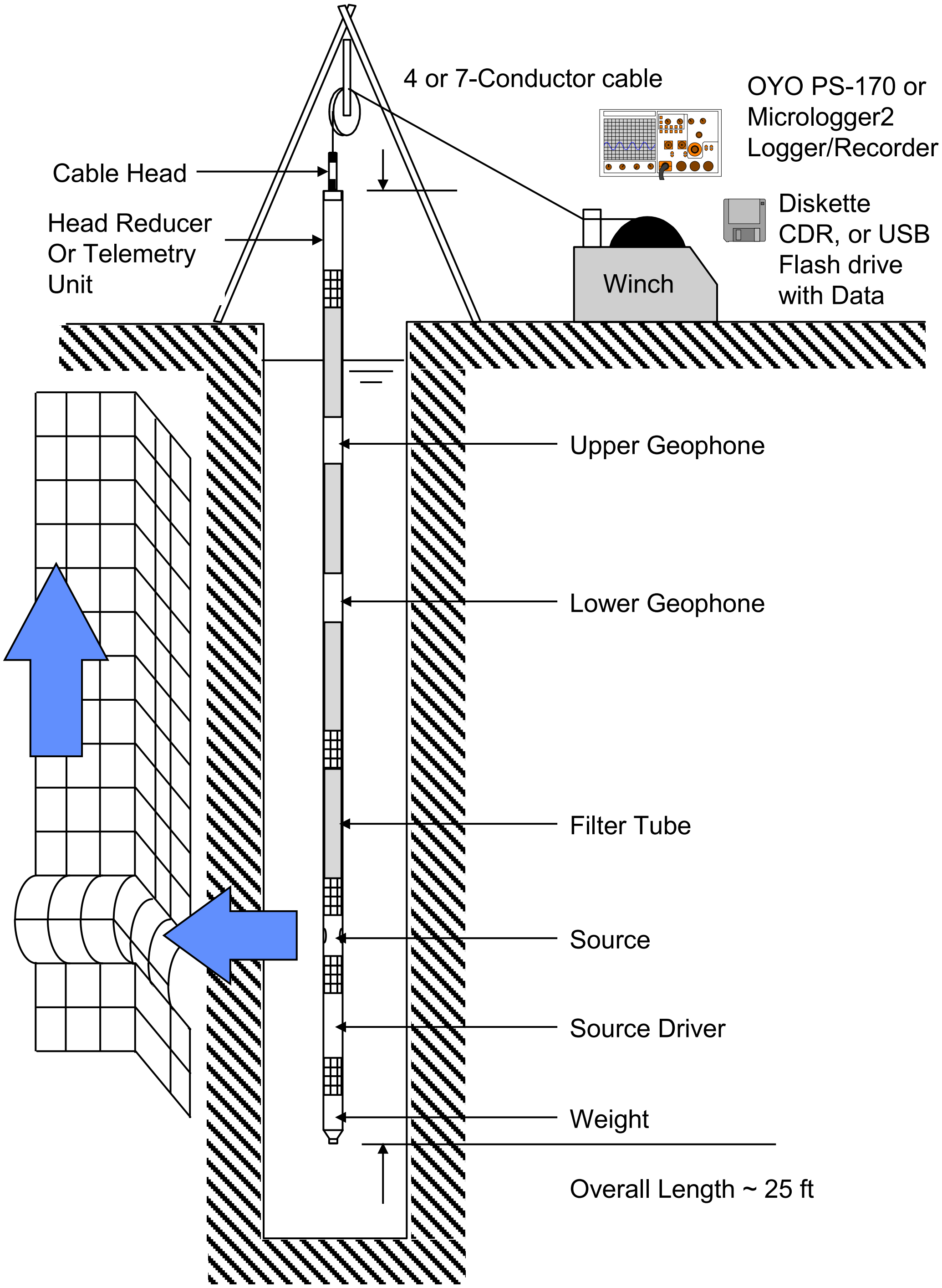

Figure 1: Concept illustration of P-S logging system

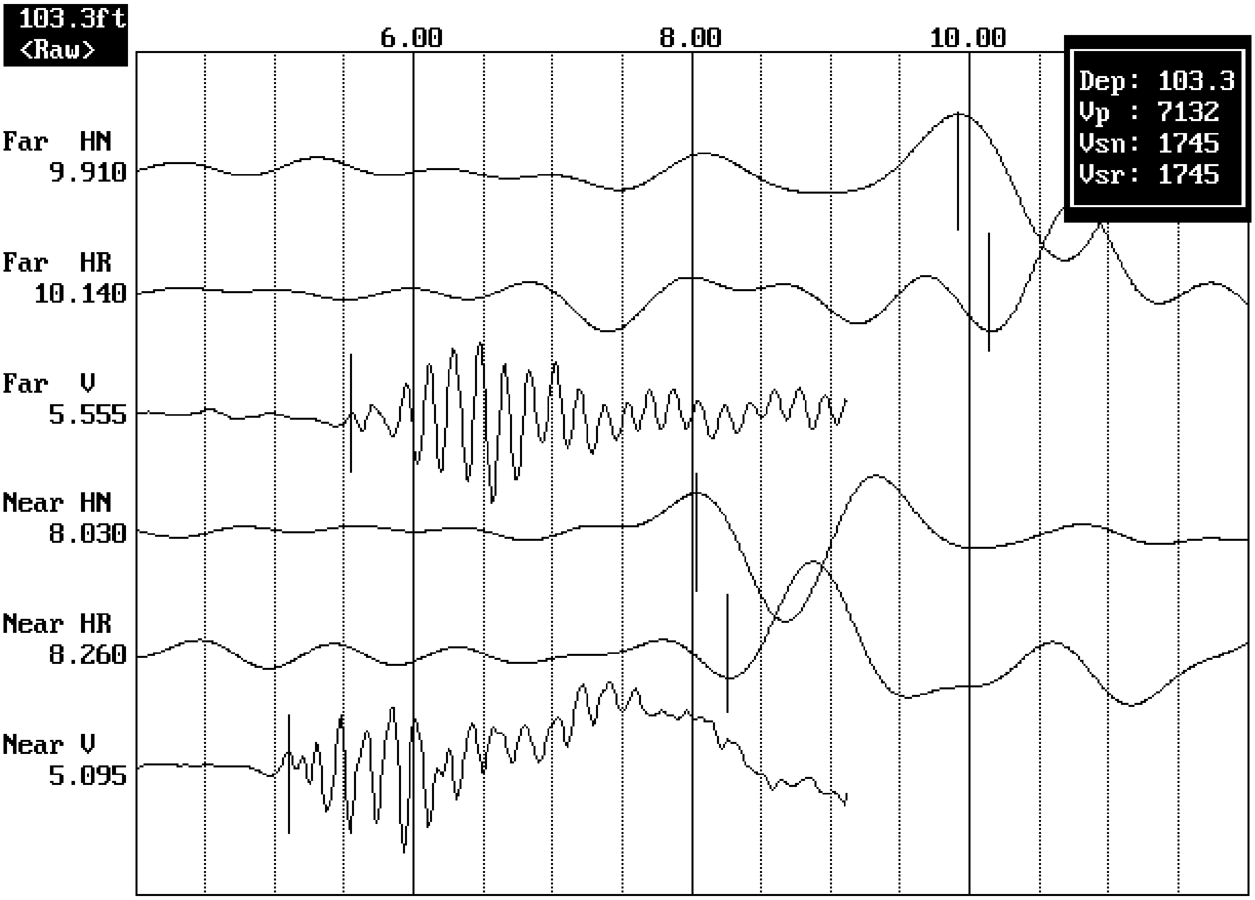

Figure 2: Example of filtered (1400 Hz lowpass) suspension record

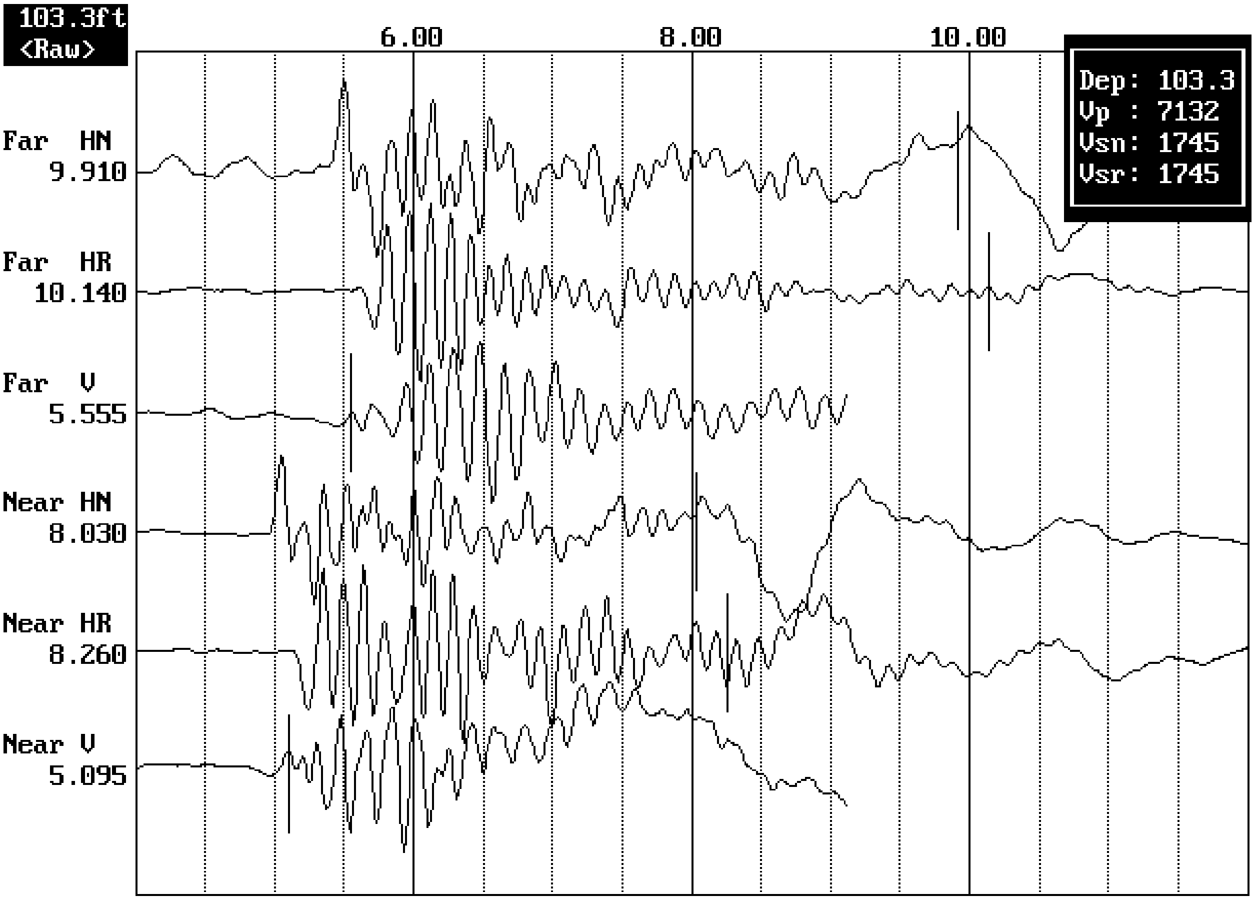

Figure 3. Example of unfiltered suspension record

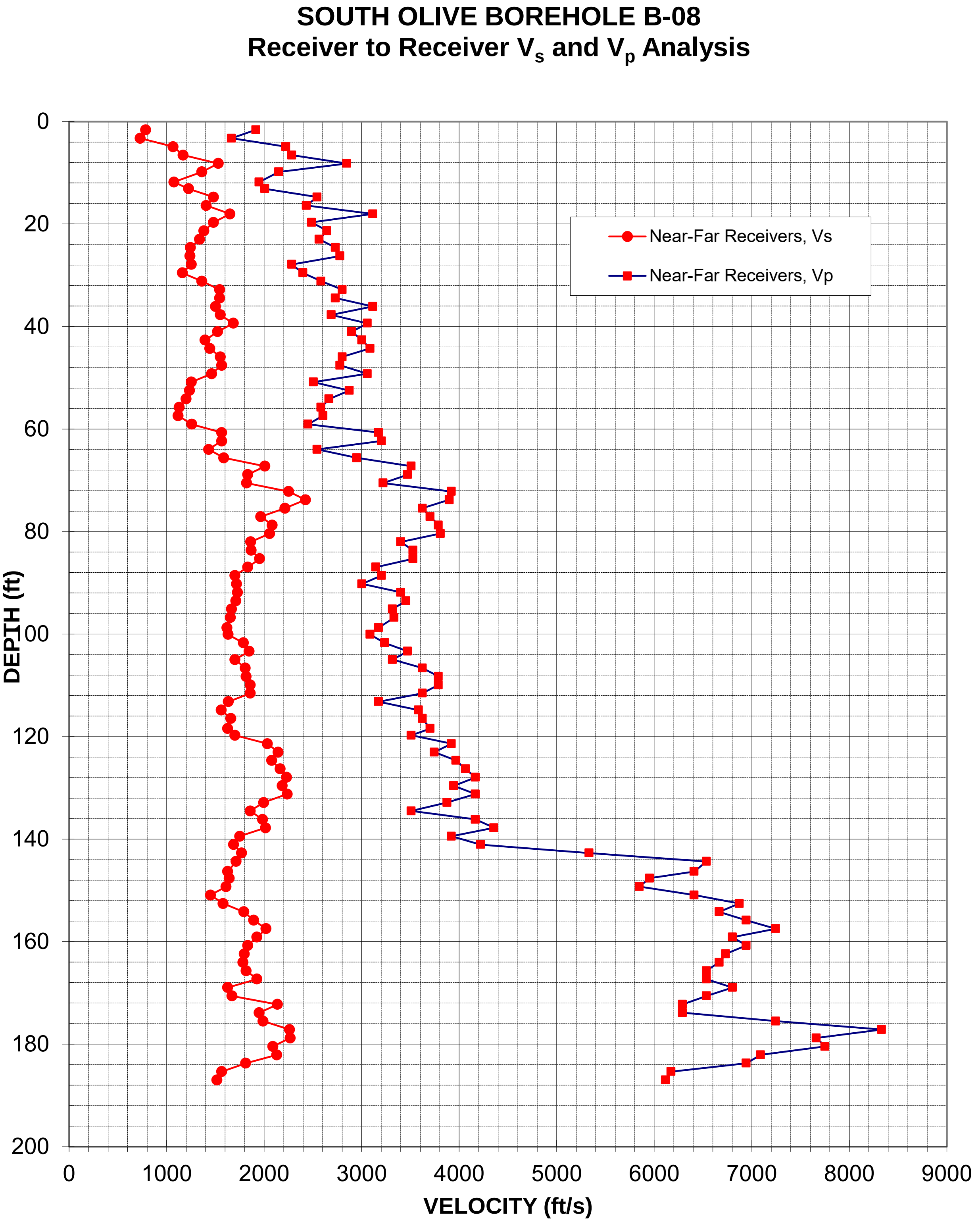

Figure 4: Borehole B-8, Suspension R1-R2 P- and $S_H$-wave velocities

Table 3. Borehole B-8, 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-8**

| American Units | | | |
|---|---|---|---|
| **Depth at Midpoint Between Receivers** | **Velocity** | | **Poisson's Ratio** |
| | **$V_s$** | **$V_p$** | |
| (ft) | (ft/s) | (ft/s) | |
| 1.6 | 780 | 1920 | 0.40 |
| 3.3 | 730 | 1670 | 0.38 |
| 4.9 | 1070 | 2220 | 0.35 |
| 6.6 | 1170 | 2280 | 0.32 |
| 8.2 | 1530 | 2850 | 0.30 |
| 9.8 | 1360 | 2150 | 0.17 |
| 11.8 | 1080 | 1950 | 0.28 |
| 13.1 | 1230 | 2010 | 0.20 |
| 14.8 | 1480 | 2540 | 0.24 |
| 16.4 | 1410 | 2430 | 0.25 |
| 18.0 | 1650 | 3120 | 0.30 |
| 19.7 | 1480 | 2490 | 0.23 |
| 21.3 | 1380 | 2650 | 0.31 |
| 23.0 | 1340 | 2560 | 0.31 |
| 24.6 | 1240 | 2730 | 0.37 |
| 26.3 | 1240 | 2780 | 0.38 |
| 27.9 | 1250 | 2280 | 0.28 |
| 29.5 | 1160 | 2400 | 0.35 |
| 31.2 | 1360 | 2580 | 0.31 |
| 32.8 | 1540 | 2800 | 0.28 |
| 34.5 | 1540 | 2730 | 0.27 |
| 36.1 | 1500 | 3120 | 0.35 |
| 37.7 | 1550 | 2690 | 0.25 |
| 39.4 | 1680 | 3060 | 0.28 |
| 41.0 | 1520 | 2900 | 0.31 |
| 42.7 | 1390 | 3000 | 0.36 |
| 44.3 | 1440 | 3090 | 0.36 |
| 45.9 | 1550 | 2800 | 0.28 |
| 47.6 | 1560 | 2780 | 0.27 |
| 49.2 | 1460 | 3060 | 0.35 |
| 50.9 | 1250 | 2510 | 0.33 |
| 52.5 | 1230 | 2870 | 0.39 |
| 54.1 | 1200 | 2670 | 0.37 |
| 55.8 | 1130 | 2580 | 0.38 |
| 57.4 | 1120 | 2600 | 0.39 |
| 59.1 | 1260 | 2450 | 0.32 |

| Metric Units | | | |
|---|---|---|---|
| **Depth at Midpoint Between Receivers** | **Velocity** | | **Poisson's Ratio** |
| | **$V_s$** | **$V_p$** | |
| (m) | (m/s) | (m/s) | |
| 0.5 | 240 | 580 | 0.40 |
| 1.0 | 220 | 510 | 0.38 |
| 1.5 | 330 | 680 | 0.35 |
| 2.0 | 360 | 700 | 0.32 |
| 2.5 | 470 | 870 | 0.30 |
| 3.0 | 410 | 660 | 0.17 |
| 3.6 | 330 | 590 | 0.28 |
| 4.0 | 370 | 610 | 0.20 |
| 4.5 | 450 | 780 | 0.24 |
| 5.0 | 430 | 740 | 0.25 |
| 5.5 | 500 | 950 | 0.30 |
| 6.0 | 450 | 760 | 0.23 |
| 6.5 | 420 | 810 | 0.31 |
| 7.0 | 410 | 780 | 0.31 |
| 7.5 | 380 | 830 | 0.37 |
| 8.0 | 380 | 850 | 0.38 |
| 8.5 | 380 | 700 | 0.28 |
| 9.0 | 350 | 730 | 0.35 |
| 9.5 | 410 | 790 | 0.31 |
| 10.0 | 470 | 850 | 0.28 |
| 10.5 | 470 | 830 | 0.27 |
| 11.0 | 460 | 950 | 0.35 |
| 11.5 | 470 | 820 | 0.25 |
| 12.0 | 510 | 930 | 0.28 |
| 12.5 | 460 | 880 | 0.31 |
| 13.0 | 430 | 920 | 0.36 |
| 13.5 | 440 | 940 | 0.36 |
| 14.0 | 470 | 850 | 0.28 |
| 14.5 | 480 | 850 | 0.27 |
| 15.0 | 450 | 930 | 0.35 |
| 15.5 | 380 | 760 | 0.33 |
| 16.0 | 380 | 880 | 0.39 |
| 16.5 | 370 | 810 | 0.37 |
| 17.0 | 340 | 790 | 0.38 |
| 17.5 | 340 | 790 | 0.39 |
| 18.0 | 380 | 750 | 0.32 |

## Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio
## Based on Receiver-to-Receiver Travel Time Data - Borehole B-8

| American Units | | | |
|---|---|---|---|
| Depth at Midpoint Between Receivers | Velocity | | Poisson's Ratio |
| | $V_s$ | $V_p$ | |
| (ft) | (ft/s) | (ft/s) | |
| 60.7 | 1560 | 3170 | 0.34 |
| 62.3 | 1560 | 3210 | 0.34 |
| 64.0 | 1430 | 2540 | 0.27 |
| 65.6 | 1590 | 2950 | 0.30 |
| 67.3 | 2010 | 3510 | 0.26 |
| 68.9 | 1830 | 3470 | 0.31 |
| 70.5 | 1820 | 3220 | 0.26 |
| 72.2 | 2250 | 3920 | 0.25 |
| 73.8 | 2420 | 3900 | 0.18 |
| 75.5 | 2210 | 3620 | 0.20 |
| 77.1 | 1970 | 3700 | 0.30 |
| 78.7 | 2080 | 3790 | 0.28 |
| 80.4 | 2060 | 3810 | 0.29 |
| 82.0 | 1860 | 3400 | 0.29 |
| 83.7 | 1870 | 3530 | 0.31 |
| 85.3 | 1960 | 3530 | 0.28 |
| 86.9 | 1830 | 3140 | 0.24 |
| 88.6 | 1700 | 3210 | 0.30 |
| 90.2 | 1720 | 3000 | 0.26 |
| 91.9 | 1730 | 3400 | 0.33 |
| 93.5 | 1710 | 3450 | 0.34 |
| 95.1 | 1670 | 3320 | 0.33 |
| 96.8 | 1650 | 3330 | 0.34 |
| 98.8 | 1620 | 3170 | 0.32 |
| 100.1 | 1630 | 3090 | 0.31 |
| 101.7 | 1790 | 3240 | 0.28 |
| 103.4 | 1850 | 3470 | 0.30 |
| 105.0 | 1700 | 3320 | 0.32 |
| 106.6 | 1810 | 3620 | 0.33 |
| 108.3 | 1820 | 3790 | 0.35 |
| 109.9 | 1860 | 3790 | 0.34 |
| 111.6 | 1860 | 3620 | 0.32 |
| 113.2 | 1630 | 3170 | 0.32 |
| 114.8 | 1560 | 3580 | 0.38 |
| 116.5 | 1660 | 3620 | 0.37 |
| 118.4 | 1630 | 3700 | 0.38 |
| 119.8 | 1700 | 3510 | 0.35 |
| 121.4 | 2030 | 3920 | 0.32 |

| Metric Units | | | |
|---|---|---|---|
| Depth at Midpoint Between Receivers | Velocity | | Poisson's Ratio |
| | $V_s$ | $V_p$ | |
| (m) | (m/s) | (m/s) | |
| 18.5 | 480 | 970 | 0.34 |
| 19.0 | 480 | 980 | 0.34 |
| 19.5 | 440 | 780 | 0.27 |
| 20.0 | 480 | 900 | 0.30 |
| 20.5 | 610 | 1070 | 0.26 |
| 21.0 | 560 | 1060 | 0.31 |
| 21.5 | 560 | 980 | 0.26 |
| 22.0 | 690 | 1200 | 0.25 |
| 22.5 | 740 | 1190 | 0.18 |
| 23.0 | 680 | 1100 | 0.20 |
| 23.5 | 600 | 1130 | 0.30 |
| 24.0 | 640 | 1150 | 0.28 |
| 24.5 | 630 | 1160 | 0.29 |
| 25.0 | 570 | 1040 | 0.29 |
| 25.5 | 570 | 1080 | 0.31 |
| 26.0 | 600 | 1080 | 0.28 |
| 26.5 | 560 | 960 | 0.24 |
| 27.0 | 520 | 980 | 0.30 |
| 27.5 | 520 | 920 | 0.26 |
| 28.0 | 530 | 1040 | 0.33 |
| 28.5 | 520 | 1050 | 0.34 |
| 29.0 | 510 | 1010 | 0.33 |
| 29.5 | 500 | 1020 | 0.34 |
| 30.1 | 490 | 970 | 0.32 |
| 30.5 | 500 | 940 | 0.31 |
| 31.0 | 540 | 990 | 0.28 |
| 31.5 | 560 | 1060 | 0.30 |
| 32.0 | 520 | 1010 | 0.32 |
| 32.5 | 550 | 1100 | 0.33 |
| 33.0 | 550 | 1150 | 0.35 |
| 33.5 | 570 | 1150 | 0.34 |
| 34.0 | 570 | 1100 | 0.32 |
| 34.5 | 500 | 970 | 0.32 |
| 35.0 | 480 | 1090 | 0.38 |
| 35.5 | 510 | 1100 | 0.37 |
| 36.1 | 500 | 1130 | 0.38 |
| 36.5 | 520 | 1070 | 0.35 |
| 37.0 | 620 | 1200 | 0.32 |

## Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio Based on Receiver-to-Receiver Travel Time Data - Borehole B-8

| American Units | | | |
|---|---|---|---|
| Depth at Midpoint Between Receivers | Velocity | | Poisson's Ratio |
| | $V_s$ | $V_p$ | |
| (ft) | (ft/s) | (ft/s) | |
| 123.0 | 2140 | 3750 | 0.26 |
| 124.7 | 2080 | 3970 | 0.31 |
| 126.3 | 2160 | 4070 | 0.30 |
| 128.0 | 2230 | 4170 | 0.30 |
| 129.6 | 2190 | 3940 | 0.28 |
| 131.2 | 2240 | 4170 | 0.30 |
| 132.9 | 2000 | 3880 | 0.32 |
| 134.5 | 1860 | 3510 | 0.31 |
| 136.2 | 1980 | 4170 | 0.35 |
| 137.8 | 2010 | 4360 | 0.36 |
| 139.4 | 1750 | 3920 | 0.38 |
| 141.1 | 1690 | 4220 | 0.40 |
| 142.7 | 1770 | 5330 | 0.44 |
| 144.4 | 1710 | 6540 | 0.46 |
| 146.3 | 1630 | 6410 | 0.47 |
| 147.6 | 1640 | 5950 | 0.46 |
| 149.3 | 1610 | 5850 | 0.46 |
| 150.9 | 1450 | 6410 | 0.47 |
| 152.6 | 1580 | 6870 | 0.47 |
| 154.2 | 1790 | 6670 | 0.46 |
| 155.8 | 1890 | 6940 | 0.46 |
| 157.5 | 2020 | 7250 | 0.46 |
| 159.1 | 1930 | 6800 | 0.46 |
| 160.8 | 1830 | 6940 | 0.46 |
| 162.4 | 1800 | 6730 | 0.46 |
| 164.0 | 1780 | 6670 | 0.46 |
| 165.7 | 1820 | 6540 | 0.46 |
| 167.3 | 1930 | 6540 | 0.45 |
| 169.0 | 1630 | 6800 | 0.47 |
| 170.6 | 1670 | 6540 | 0.47 |
| 172.2 | 2140 | 6290 | 0.43 |
| 173.9 | 1950 | 6290 | 0.45 |
| 175.5 | 1990 | 7250 | 0.46 |
| 177.2 | 2260 | 8330 | 0.46 |
| 178.8 | 2270 | 7660 | 0.45 |
| 180.5 | 2090 | 7750 | 0.46 |
| 182.1 | 2130 | 7090 | 0.45 |
| 183.7 | 1810 | 6940 | 0.46 |

| Metric Units | | | |
|---|---|---|---|
| Depth at Midpoint Between Receivers | Velocity | | Poisson's Ratio |
| | $V_s$ | $V_p$ | |
| (m) | (m/s) | (m/s) | |
| 37.5 | 650 | 1140 | 0.26 |
| 38.0 | 630 | 1210 | 0.31 |
| 38.5 | 660 | 1240 | 0.30 |
| 39.0 | 680 | 1270 | 0.30 |
| 39.5 | 670 | 1200 | 0.28 |
| 40.0 | 680 | 1270 | 0.30 |
| 40.5 | 610 | 1180 | 0.32 |
| 41.0 | 570 | 1070 | 0.31 |
| 41.5 | 600 | 1270 | 0.35 |
| 42.0 | 610 | 1330 | 0.36 |
| 42.5 | 530 | 1200 | 0.38 |
| 43.0 | 510 | 1290 | 0.40 |
| 43.5 | 540 | 1630 | 0.44 |
| 44.0 | 520 | 1990 | 0.46 |
| 44.6 | 500 | 1950 | 0.47 |
| 45.0 | 500 | 1810 | 0.46 |
| 45.5 | 490 | 1780 | 0.46 |
| 46.0 | 440 | 1950 | 0.47 |
| 46.5 | 480 | 2090 | 0.47 |
| 47.0 | 550 | 2030 | 0.46 |
| 47.5 | 580 | 2120 | 0.46 |
| 48.0 | 620 | 2210 | 0.46 |
| 48.5 | 590 | 2070 | 0.46 |
| 49.0 | 560 | 2120 | 0.46 |
| 49.5 | 550 | 2050 | 0.46 |
| 50.0 | 540 | 2030 | 0.46 |
| 50.5 | 550 | 1990 | 0.46 |
| 51.0 | 590 | 1990 | 0.45 |
| 51.5 | 500 | 2070 | 0.47 |
| 52.0 | 510 | 1990 | 0.47 |
| 52.5 | 650 | 1920 | 0.43 |
| 53.0 | 590 | 1920 | 0.45 |
| 53.5 | 610 | 2210 | 0.46 |
| 54.0 | 690 | 2540 | 0.46 |
| 54.5 | 690 | 2340 | 0.45 |
| 55.0 | 640 | 2360 | 0.46 |
| 55.5 | 650 | 2160 | 0.45 |
| 56.0 | 550 | 2120 | 0.46 |

## Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio Based on Receiver-to-Receiver Travel Time Data - Borehole B-8

| American Units | | | |
|---|---|---|---|
| **Depth at Midpoint Between Receivers** | **Velocity** | | **Poisson's Ratio** |
| | **$V_s$** | **$V_p$** | |
| (ft) | (ft/s) | (ft/s) | |
| 185.4 | 1560 | 6170 | 0.47 |
| 187.0 | 1520 | 6120 | 0.47 |

| Metric Units | | | |
|---|---|---|---|
| **Depth at Midpoint Between Receivers** | **Velocity** | | **Poisson's Ratio** |
| | **$V_s$** | **$V_p$** | |
| (m) | (m/s) | (m/s) | |
| 56.5 | 480 | 1880 | 0.47 |
| 57.0 | 460 | 1860 | 0.47 |

**Notes:** "-" means no data available at that particular interval of depth.

# APPENDIX A

# SUSPENSION VELOCITY MEASUREMENT QUALITY ASSURANCE SUSPENSION SOURCE TO RECEIVER ANALYSIS RESULTS

## SOUTH OLIVE BOREHOLE B-08
## Source to Receiver and Receiver to Receiver Analysis

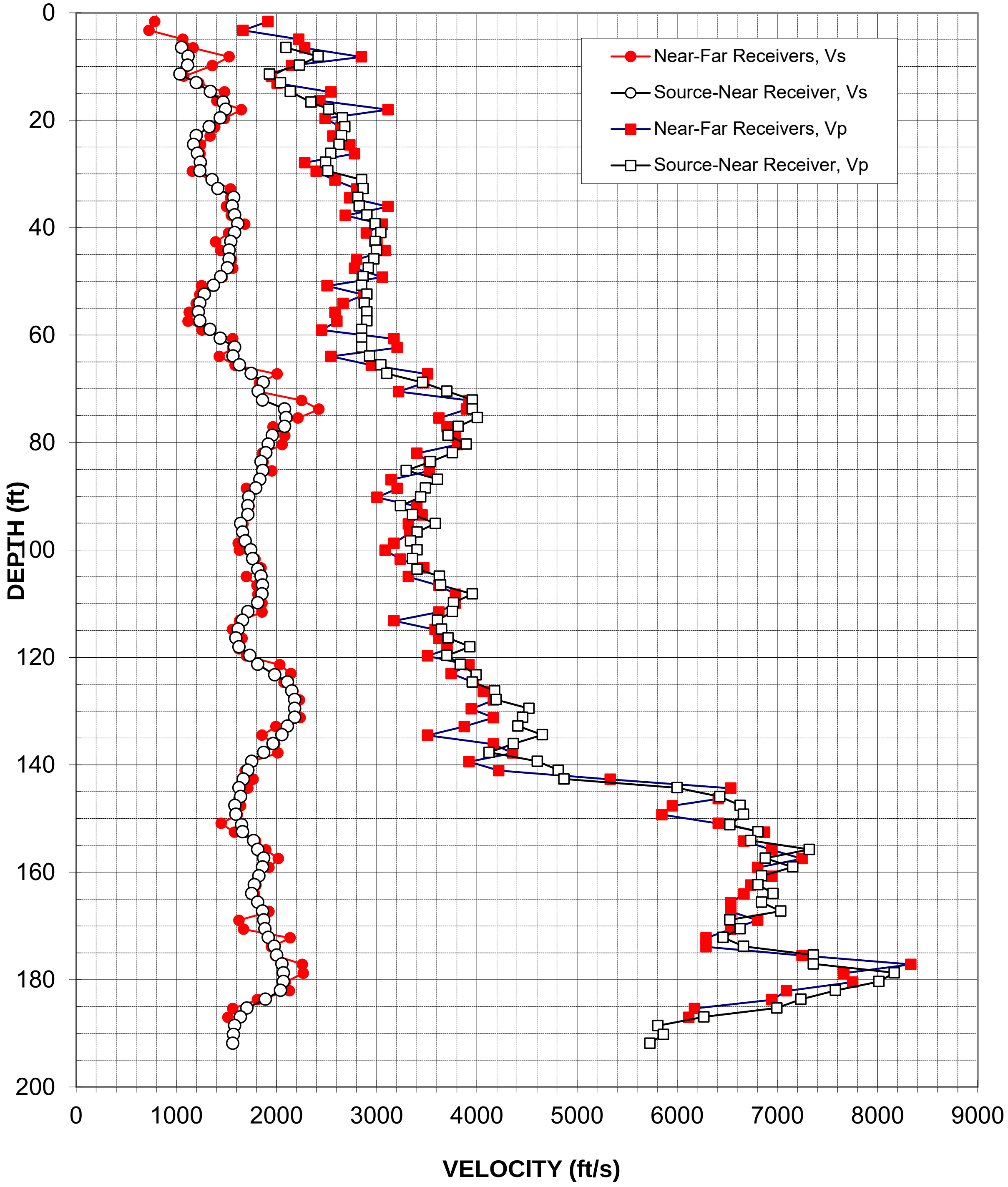

Figure A-1: Borehole B-8, Suspension S-R1 P- and $S_H$-wave velocities

## Table A-1. Borehole B-8, 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-8

| American Units | | | |
|---|---|---|---|
| Depth at Midpoint Between Source and Near Receiver (ft) | Velocity | | Poisson's Ratio |
| | $V_s$ (ft/s) | $V_p$ (ft/s) | |
| 6.5 | 1050 | 2100 | 0.33 |
| 8.1 | 1120 | 2420 | 0.36 |
| 9.8 | 1110 | 2230 | 0.33 |
| 11.4 | 1040 | 1930 | 0.30 |
| 13.0 | 1200 | 2040 | 0.24 |
| 14.7 | 1340 | 2140 | 0.18 |
| 16.6 | 1470 | 2340 | 0.18 |
| 18.0 | 1490 | 2520 | 0.23 |
| 19.6 | 1440 | 2660 | 0.29 |
| 21.2 | 1330 | 2680 | 0.34 |
| 22.9 | 1200 | 2650 | 0.37 |
| 24.5 | 1170 | 2630 | 0.38 |
| 26.2 | 1210 | 2540 | 0.35 |
| 27.8 | 1240 | 2490 | 0.34 |
| 29.4 | 1240 | 2510 | 0.34 |
| 31.1 | 1360 | 2850 | 0.35 |
| 32.7 | 1420 | 2860 | 0.34 |
| 34.4 | 1570 | 2810 | 0.27 |
| 36.0 | 1560 | 2830 | 0.28 |
| 37.6 | 1580 | 2900 | 0.29 |
| 39.3 | 1610 | 2990 | 0.29 |
| 40.9 | 1580 | 3040 | 0.31 |
| 42.6 | 1540 | 2990 | 0.32 |
| 44.2 | 1530 | 3000 | 0.32 |
| 45.8 | 1530 | 2970 | 0.32 |
| 47.5 | 1510 | 2920 | 0.32 |
| 49.1 | 1450 | 2860 | 0.33 |
| 50.8 | 1370 | 2850 | 0.35 |
| 52.4 | 1280 | 2900 | 0.38 |
| 54.0 | 1240 | 2880 | 0.39 |
| 55.7 | 1220 | 2900 | 0.39 |
| 57.3 | 1240 | 2900 | 0.39 |
| 59.0 | 1340 | 2850 | 0.36 |
| 60.6 | 1440 | 2850 | 0.33 |
| 62.2 | 1580 | 2850 | 0.28 |
| 63.9 | 1570 | 2930 | 0.30 |

| Metric Units | | | |
|---|---|---|---|
| Depth at Midpoint Between Source and Near Receiver (m) | Velocity | | Poisson's Ratio |
| | $V_s$ (m/s) | $V_p$ (m/s) | |
| 2.0 | 320 | 640 | 0.33 |
| 2.5 | 340 | 740 | 0.36 |
| 3.0 | 340 | 680 | 0.33 |
| 3.5 | 320 | 590 | 0.30 |
| 4.0 | 370 | 620 | 0.24 |
| 4.5 | 410 | 650 | 0.18 |
| 5.1 | 450 | 710 | 0.18 |
| 5.5 | 460 | 770 | 0.23 |
| 6.0 | 440 | 810 | 0.29 |
| 6.5 | 410 | 820 | 0.34 |
| 7.0 | 370 | 810 | 0.37 |
| 7.5 | 360 | 800 | 0.38 |
| 8.0 | 370 | 770 | 0.35 |
| 8.5 | 380 | 760 | 0.34 |
| 9.0 | 380 | 770 | 0.34 |
| 9.5 | 410 | 870 | 0.35 |
| 10.0 | 430 | 870 | 0.34 |
| 10.5 | 480 | 860 | 0.27 |
| 11.0 | 480 | 860 | 0.28 |
| 11.5 | 480 | 890 | 0.29 |
| 12.0 | 490 | 910 | 0.29 |
| 12.5 | 480 | 930 | 0.31 |
| 13.0 | 470 | 910 | 0.32 |
| 13.5 | 470 | 910 | 0.32 |
| 14.0 | 470 | 910 | 0.32 |
| 14.5 | 460 | 890 | 0.32 |
| 15.0 | 440 | 870 | 0.33 |
| 15.5 | 420 | 870 | 0.35 |
| 16.0 | 390 | 890 | 0.38 |
| 16.5 | 380 | 880 | 0.39 |
| 17.0 | 370 | 890 | 0.39 |
| 17.5 | 380 | 890 | 0.39 |
| 18.0 | 410 | 870 | 0.36 |
| 18.5 | 440 | 870 | 0.33 |
| 19.0 | 480 | 870 | 0.28 |
| 19.5 | 480 | 890 | 0.30 |

## Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio
## Based on Source-to-Receiver Travel Time Data - Borehole B-8

| American Units | | | |
|---|---|---|---|
| **Depth at Midpoint Between Source and Near Receiver** | **Velocity** | | **Poisson's Ratio** |
| | **$V_s$** | **$V_p$** | |
| (ft) | (ft/s) | (ft/s) | |
| 65.5 | 1630 | 3040 | 0.30 |
| 67.2 | 1750 | 3100 | 0.27 |
| 68.8 | 1870 | 3460 | 0.29 |
| 70.5 | 1820 | 3700 | 0.34 |
| 72.1 | 1860 | 3960 | 0.36 |
| 73.7 | 2080 | 3960 | 0.31 |
| 75.4 | 2100 | 4010 | 0.31 |
| 77.0 | 2080 | 3810 | 0.29 |
| 78.7 | 1960 | 3710 | 0.31 |
| 80.3 | 1920 | 3900 | 0.34 |
| 81.9 | 1900 | 3760 | 0.33 |
| 83.6 | 1850 | 3540 | 0.31 |
| 85.2 | 1860 | 3300 | 0.26 |
| 86.9 | 1830 | 3610 | 0.33 |
| 88.5 | 1790 | 3490 | 0.32 |
| 90.1 | 1720 | 3440 | 0.33 |
| 91.8 | 1720 | 3240 | 0.30 |
| 93.4 | 1720 | 3360 | 0.32 |
| 95.1 | 1640 | 3590 | 0.37 |
| 96.7 | 1660 | 3400 | 0.34 |
| 98.3 | 1690 | 3340 | 0.33 |
| 100.0 | 1740 | 3400 | 0.32 |
| 101.6 | 1760 | 3360 | 0.31 |
| 103.6 | 1810 | 3400 | 0.30 |
| 104.9 | 1850 | 3630 | 0.33 |
| 106.5 | 1860 | 3640 | 0.32 |
| 108.2 | 1860 | 3960 | 0.36 |
| 109.8 | 1810 | 3770 | 0.35 |
| 111.5 | 1720 | 3760 | 0.37 |
| 113.1 | 1660 | 3610 | 0.37 |
| 114.7 | 1620 | 3650 | 0.38 |
| 116.4 | 1590 | 3710 | 0.39 |
| 118.0 | 1630 | 3930 | 0.40 |
| 119.7 | 1730 | 3700 | 0.36 |
| 121.3 | 1810 | 3840 | 0.36 |
| 123.3 | 1980 | 3990 | 0.34 |
| 124.6 | 2110 | 3960 | 0.30 |
| 126.2 | 2150 | 4180 | 0.32 |
| 127.9 | 2180 | 4190 | 0.31 |

| Metric Units | | | |
|---|---|---|---|
| **Depth at Midpoint Between Source and Near Receiver** | **Velocity** | | **Poisson's Ratio** |
| | **$V_s$** | **$V_p$** | |
| (m) | (m/s) | (m/s) | |
| 20.0 | 500 | 930 | 0.30 |
| 20.5 | 530 | 950 | 0.27 |
| 21.0 | 570 | 1050 | 0.29 |
| 21.5 | 550 | 1130 | 0.34 |
| 22.0 | 570 | 1210 | 0.36 |
| 22.5 | 630 | 1210 | 0.31 |
| 23.0 | 640 | 1220 | 0.31 |
| 23.5 | 630 | 1160 | 0.29 |
| 24.0 | 600 | 1130 | 0.31 |
| 24.5 | 580 | 1190 | 0.34 |
| 25.0 | 580 | 1150 | 0.33 |
| 25.5 | 560 | 1080 | 0.31 |
| 26.0 | 570 | 1000 | 0.26 |
| 26.5 | 560 | 1100 | 0.33 |
| 27.0 | 550 | 1060 | 0.32 |
| 27.5 | 530 | 1050 | 0.33 |
| 28.0 | 520 | 990 | 0.30 |
| 28.5 | 520 | 1020 | 0.32 |
| 29.0 | 500 | 1090 | 0.37 |
| 29.5 | 510 | 1040 | 0.34 |
| 30.0 | 510 | 1020 | 0.33 |
| 30.5 | 530 | 1040 | 0.32 |
| 31.0 | 540 | 1020 | 0.31 |
| 31.6 | 550 | 1040 | 0.30 |
| 32.0 | 560 | 1110 | 0.33 |
| 32.5 | 570 | 1110 | 0.32 |
| 33.0 | 570 | 1210 | 0.36 |
| 33.5 | 550 | 1150 | 0.35 |
| 34.0 | 520 | 1150 | 0.37 |
| 34.5 | 510 | 1100 | 0.37 |
| 35.0 | 490 | 1110 | 0.38 |
| 35.5 | 490 | 1130 | 0.39 |
| 36.0 | 500 | 1200 | 0.40 |
| 36.5 | 530 | 1130 | 0.36 |
| 37.0 | 550 | 1170 | 0.36 |
| 37.6 | 600 | 1220 | 0.34 |
| 38.0 | 640 | 1210 | 0.30 |
| 38.5 | 660 | 1270 | 0.32 |
| 39.0 | 670 | 1280 | 0.31 |

## Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio Based on Source-to-Receiver Travel Time Data - Borehole B-8

| American Units | | | |
|---|---|---|---|
| Depth at Midpoint Between Source and Near Receiver | Velocity | | Poisson's Ratio |
| | $V_s$ | $V_p$ | |
| (ft) | (ft/s) | (ft/s) | |
| 129.5 | 2180 | 4520 | 0.35 |
| 131.1 | 2180 | 4460 | 0.34 |
| 132.8 | 2110 | 4410 | 0.35 |
| 134.4 | 2060 | 4650 | 0.38 |
| 136.1 | 1970 | 4370 | 0.37 |
| 137.7 | 1870 | 4120 | 0.37 |
| 139.3 | 1750 | 4600 | 0.42 |
| 141.0 | 1720 | 4810 | 0.43 |
| 142.6 | 1670 | 4870 | 0.43 |
| 144.3 | 1630 | 6000 | 0.46 |
| 145.9 | 1640 | 6430 | 0.46 |
| 147.6 | 1590 | 6630 | 0.47 |
| 149.2 | 1590 | 6660 | 0.47 |
| 151.2 | 1650 | 6530 | 0.47 |
| 152.5 | 1660 | 6810 | 0.47 |
| 154.1 | 1770 | 6730 | 0.46 |
| 155.8 | 1810 | 7320 | 0.47 |
| 157.4 | 1870 | 6880 | 0.46 |
| 159.0 | 1860 | 7150 | 0.46 |
| 160.7 | 1820 | 6840 | 0.46 |
| 162.3 | 1780 | 6810 | 0.46 |
| 164.0 | 1750 | 6960 | 0.47 |
| 165.6 | 1810 | 6840 | 0.46 |
| 167.2 | 1860 | 7030 | 0.46 |
| 168.9 | 1870 | 6530 | 0.46 |
| 170.5 | 1880 | 6630 | 0.46 |
| 172.2 | 1920 | 6460 | 0.45 |
| 173.8 | 1980 | 6660 | 0.45 |
| 175.4 | 2000 | 7360 | 0.46 |
| 177.1 | 2060 | 7360 | 0.46 |
| 178.7 | 2070 | 8170 | 0.47 |
| 180.4 | 2070 | 8010 | 0.46 |
| 182.0 | 2040 | 7580 | 0.46 |
| 183.6 | 1890 | 7230 | 0.46 |
| 185.3 | 1710 | 6990 | 0.47 |
| 186.9 | 1640 | 6270 | 0.46 |
| 188.6 | 1580 | 5810 | 0.46 |
| 190.2 | 1570 | 5860 | 0.46 |
| 191.8 | 1560 | 5730 | 0.46 |

| Metric Units | | | |
|---|---|---|---|
| Depth at Midpoint Between Source and Near Receiver | Velocity | | Poisson's Ratio |
| | $V_s$ | $V_p$ | |
| (m) | (m/s) | (m/s) | |
| 39.5 | 670 | 1380 | 0.35 |
| 40.0 | 670 | 1360 | 0.34 |
| 40.5 | 640 | 1340 | 0.35 |
| 41.0 | 630 | 1420 | 0.38 |
| 41.5 | 600 | 1330 | 0.37 |
| 42.0 | 570 | 1260 | 0.37 |
| 42.5 | 530 | 1400 | 0.42 |
| 43.0 | 520 | 1470 | 0.43 |
| 43.5 | 510 | 1480 | 0.43 |
| 44.0 | 500 | 1830 | 0.46 |
| 44.5 | 500 | 1960 | 0.46 |
| 45.0 | 480 | 2020 | 0.47 |
| 45.5 | 490 | 2030 | 0.47 |
| 46.1 | 500 | 1990 | 0.47 |
| 46.5 | 510 | 2070 | 0.47 |
| 47.0 | 540 | 2050 | 0.46 |
| 47.5 | 550 | 2230 | 0.47 |
| 48.0 | 570 | 2100 | 0.46 |
| 48.5 | 570 | 2180 | 0.46 |
| 49.0 | 560 | 2090 | 0.46 |
| 49.5 | 540 | 2070 | 0.46 |
| 50.0 | 530 | 2120 | 0.47 |
| 50.5 | 550 | 2090 | 0.46 |
| 51.0 | 570 | 2140 | 0.46 |
| 51.5 | 570 | 1990 | 0.46 |
| 52.0 | 570 | 2020 | 0.46 |
| 52.5 | 580 | 1970 | 0.45 |
| 53.0 | 600 | 2030 | 0.45 |
| 53.5 | 610 | 2240 | 0.46 |
| 54.0 | 630 | 2240 | 0.46 |
| 54.5 | 630 | 2490 | 0.47 |
| 55.0 | 630 | 2440 | 0.46 |
| 55.5 | 620 | 2310 | 0.46 |
| 56.0 | 580 | 2210 | 0.46 |
| 56.5 | 520 | 2130 | 0.47 |
| 57.0 | 500 | 1910 | 0.46 |
| 57.5 | 480 | 1770 | 0.46 |
| 58.0 | 480 | 1790 | 0.46 |
| 58.5 | 480 | 1750 | 0.46 |

# APPENDIX B

# BOREHOLE GEOPHYSICAL LOGGING SYSTEMS - NIST TRACEABLE CALIBRATION RECORDS

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

AC-1969.03

# Certificate of Calibration

**Date: Oct 9, 2017**

**Cert No. 512200813056866**

**Customer:**

GEOVISION

1124 OLYMPIC DRIVE
CORONA CA 92881

Work Order #: LA-90029989
Purchase Order #: 17341-170929-01

MPC Control #: AM6767
Asset ID: 160023
Gage Type: LOGGER
Manufacturer: OYO
Model Number: 3403
Size: N/A
Temp/RH: 68.8°F / 40.5%
Location: Calibration performed at MPC facility

Serial Number: 160023
Department: N/A
Performed By: NIKOLAS GROHMAN
Received Condition: IN TOLERANCE
Returned Condition: IN TOLERANCE
Cal. Date: September 29, 2017
Cal. Interval: 12 MONTHS
Cal. Due Date: September 29, 2018

**Calibration Notes:**

This Certificate Supersedes Cert No. 512200813056466

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: 5KKSA84231
Calibrated To 4:1 Accuracy Ratio

Calibration performed in accordance with approved GEOVision calibration procedures included in work Instruction No. 17.

Software: ML PS 4.00 Suspension Logger, GVLog.jar ( 2004 ) and pslog.exe ver 1.00 software.

**Standards Used to Calibrate Equipment**

| I.D. | Description. | Model | Serial | Manufacturer | Cal. Due Date | Traceability # |
|---|---|---|---|---|---|---|
| DB8748 | GPS TIME AND FREQUENCY RECEIVER | 58503A | 3625A01225 | HEWLETT PACKARD | Jun 16, 2019 | 512200812919221 |
| LAS0018 | ARB / FUNC GENERATOR | 33250A | US40001522 | AGILENT | Dec 7, 2017 | 512200812632023 |
| BD7715 | UNIVERSAL COUNTER | 53131A | 3416A05377 | HEWLETT PACKARD | Oct 3, 2017 | 512200812523201 |

Calibrating Technician:

NIKOLAS GROHMAN

QC Approval:

Tyler McKeen

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/IEC 17025:2005, ANSI/NCSL Z540-1-1994, ANSI/NCSL Z540.3-2006, 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
2165 N. Glassell St.,
Orange, CA 92865
714-901-5659

AC-1969.03

# Certificate of Calibration

**Date: Oct 9, 2017** **Cert No. 512200813056866**

## Procedures Used in this Event

| Procedure Name | Description |
|---|---|
| GEOVISION SEISMIC | Seismic Logger/Recorder Calibration Procedure, Rev. 2.1 |

Calibrating Technician: NIKOLAS GROHMAN

QC Approval: Tyler McKeen

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/IEC 17025:2005, ANSI/NCSL Z540-1-1994, ANSI/NCSL Z540.3-2006, 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 — Model no.: 3403
Serial no.: 160023 — Calibration date: 9/29/2017
By: Micro Precision — Due date: 9/29/2018

Counter mfg.: Hewlett Packard — Model no.: 53131A
Serial no.: 3416A05377 — Calibration date: 10/3/2016
By: Micro Precision — Due date: 10/3/2017

Signal generator mfg.: Agilent — Model no.: 33250A
Serial no.: US40001522 — Calibration date: 12/07/2016
By: Micro Precision — Due date: 12/07/2017

Laptop controller mfg.: Panasonic — Model no.: CF-29
Serial no.: 5KKSA84231 — Calibration date: N/A

**SYSTEM SETTINGS:**

Gain: ~~out (2)~~ 0 EFF 9/29/2017
Filter: Low Pass 10k
Range: 5-200
Delay: 0
Stack (1 std): 1
System date = correct date and time: ✓ 10:20AM 9/29/2017

**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. Acquired using ML PS4.00
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 0.34% As left 0.34%

| Target Frequency (Hz) | Actual Frequency (Hz) | Sample Period (microS) | File Name F001 | 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 | 179.8 | 50.06 | 179.8 | 50.06 | 180.6 | 49.83 |
| 100.0 | 100.0 | 100 | 002 | 90.1 | 99.89 | 90.1 | 99.89 | 90.0 | 100.0 |
| 200.0 | 200.0 | 50 | 003 | 45.05 | 199.8 | 45.0 | 200.0 | 45.05 | 199.8 |
| 500.0 | 500.0 | 20 | 004 | 18.04 | 498.9 | 17.98 | 500.6 | 18.02 | 499.5 |
| 1000 | 1000 | 10 | 005 | 8.99 | 1001 | 9.01 | 999 | 9.02 | 998 |
| 2000 | 2000 | 5 | 006 | 4.50 | 2000 | 4.51 | 1996 | 4.505 | 1998 |

Calibrated by: Name: Nik Grohman — Date: 9/29/2017 — Signature:

Witnessed by: Name: Emily Feldman — Date: 9/29/2017 — Signature:

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

# APPENDIX E.2

## Geology and Soils Report Approval Letters

BOARD OF
**BUILDING AND SAFETY COMMISSIONERS**

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MAYOR

DEPARTMENT OF
**BUILDING AND SAFETY**
201 NORTH FIGUEROA STREET
LOS ANGELES, CA 90012

OSAMA YOUNAN, P.E.
GENERAL MANAGER
SUPERINTENDENT OF BUILDING

JOHN WEIGHT
EXECUTIVE OFFICER

# SOILS REPORT APPROVAL LETTER

December 13, 2021

LOG # 115696-02
SOILS/GEOLOGY FILE - 2

MREG 1105 Olive LLC
1150 South Olive Street, Suite 2250
Los Angeles, CA 90015

| | |
|---|---|
| TRACT: | 1304 // SUBDIVISION OF BLOCK 78 ORD'S SURVEY (M R 43-74) |
| BLOCK: | 1304 // SUBDIVISION OF BLOCK 78 ORD'S SURVEY (M R 43-74) |
| LOT(S): | FR LT A // FR 7 / FR 8 (Arb. 1) / FR 8 (Arb.2) / FR 9 |
| LOCATION: | 1105 S Olive St., 1115 - 1123 S Olive St., |

| CURRENT REFERENCE REPORT/LETTER(S) | REPORT No. | DATE OF DOCUMENT | PREPARED BY |
|---|---|---|---|
| Soils Report | 700070601 | 10/14/2021 | Langan Engineering, Inc. |

| PREVIOUS REFERENCE REPORT/LETTER(S) | REPORT No. | DATE OF DOCUMENT | PREPARED BY |
|---|---|---|---|
| Dept. Review Letter | 115696-01 | 07/01/2021 | LADBS |
| Soils Report | 700070601 | 06/07/2021 | Langan Engineering, Inc. |
| Dept. Review Letter | 115696 | 01/14/2021 | LADBS |
| Soils Report | 700070601 | 12/15/2020 | Langan Engineering, Inc. |

The Grading Division of the Department of Building and Safety has reviewed the referenced report that provides recommendations for the proposed construction of a 51-story, approximately 600-foot-tall residential building over 6-level subterranean parking.

The project site includes five lots and it is bound by W 11th Street on the northeast; an existing 6-level, at-grade parking structure on the southwest; Margo Street, a 7-story residential structure over one subterranean level, and a 37-story tower and contiguous 6-story parking garage over one subterranean level on the northwest; and S Olive Street on the southeast. The earth materials at the subsurface exploration locations consist of uncertified fill underlain by alluvial deposits. The consultants recommend to support the proposed building on conventional and/or mat-type foundations bearing on native undisturbed soils.

Engineering analyses provided by Langan Engineering, Inc. are based on field and laboratory testing performed by GeoDesign Inc. and GEOVision Geophysical Services. Langan Engineering, Inc. is accepting responsibility for use of the data in accordance to LABC section 91.7008.5.

The referenced reports are acceptable, provided the following conditions are complied with during site development:

(Note: Numbers in parenthesis ( ) refer to applicable sections of the 2020 City of LA Building Code. P/BC numbers refer the applicable Information Bulletin. Information Bulletins can be accessed on the internet at LADBS.ORG.)

1. Provide a notarized letter from all adjoining property owners allowing tie-back anchors on their property (7006.6).

2. The soils engineer shall review and approve the detailed plans prior to issuance of any permit. This approval shall be by signature on the plans that clearly indicates the soils engineer has reviewed the plans prepared by the design engineer; and, that the plans included the recommendations contained in their reports (7006.1).

3. All recommendations of the reports that are in addition to or more restrictive than the conditions contained herein shall be incorporated into the plans.

4. A copy of the subject and appropriate referenced reports and this approval letter shall be attached to the District Office and field set of plans (7006.1). Submit one copy of the above reports to the Building Department Plan Checker prior to issuance of the permit.

5. A grading permit shall be obtained for all structural fill/retaining wall backfill (106.1.2).

6. All man-made fill shall be compacted to a minimum 90 percent of the maximum dry density of the fill material per the latest version of ASTM D 1557. Where cohesionless soil having less than 15 percent finer than 0.005 millimeters is used for fill, it shall be compacted to a minimum of 95 percent relative compaction based on maximum dry density. Placement of gravel in lieu of compacted fill is only allowed if complying with LAMC Section 91.7011.3.

7. Existing uncertified fill shall not be used for support of footings, concrete slabs or new fill.

8. Drainage in conformance with the provisions of the Code shall be maintained during and subsequent to construction (7013.12).

9. The applicant is advised that the approval of this report does not waive the requirements for excavations contained in the General Safety Orders of the California Department of Industrial Relations (3301.1).

10. Temporary excavations that remove lateral support to the public way, adjacent property, or adjacent structures shall be supported by shoring. Note: Lateral support shall be considered to be removed when the excavation extends below a plane projected downward at an angle of 45 degrees from the bottom of a footing of an existing structure, from the edge of the public way or an adjacent property. (3307.3.1)

11. Where any excavation, not addressed in the approved reports, would remove lateral support (as defined in 3307.3.1) from a public way, adjacent property or structures, a supplemental report shall be submitted to the Grading Division of the Department containing recommendations for shoring, underpinning, and sequence of construction.

12. Prior to the issuance of any permit that authorizes an excavation where the excavation is to be of a greater depth than are the walls or foundation of any adjoining building or structure and located closer to the property line than the depth of the excavation, the owner of the subject site shall provide the Department with evidence that the adjacent property owner has been given a 30-day written notice of such intent to make an excavation (3307.1).

13. The soils engineer shall review and approve the shoring plans prior to issuance of the permit (3307.3.2).

14. Prior to the issuance of the permits, the soils engineer and/or the structural designer shall evaluate the surcharge loads used in the report calculations for the design of the retaining walls and shoring. If the surcharge loads used in the calculations do not conform to the actual surcharge loads, the soil engineer shall submit a supplementary report with revised recommendations to the Department for approval.

15. Unsurcharged temporary excavation may be cut vertical up to 4 feet. For excavations over 4 feet, the portion of the excavation above the vertical cut shall be trimmed back at a uniform gradient not exceeding 1:1 (horizontal to vertical), as recommended.

16. Shoring shall be designed for a minimum triangular distributed lateral earth pressure of 26 PCF (cantilever) or a trapezoidal distributed lateral earth pressure of 30H PSF (braced); all surcharge loads shall be included into the design, as shown on Figures 11 and 12, included in the 10/14/2021 report. Total lateral load on shoring piles shall be determined by multiplying the recommended EFP by the pile spacing.

17. Shoring shall be designed for a maximum lateral deflection of 1 inch, provided there are no structures within a 1:1 plane projected up from the base of the excavation. Where a structure is within a 1:1 plane projected up from the base of the excavation, shoring shall be designed for a maximum lateral deflection of ½ inch, or to a lower deflection determined by the consultant that does not present any potential hazard to the adjacent structure.

18. A shoring monitoring program shall be implemented to the satisfaction of the soils engineer

19. In the event shoring soldier piles are installed using vibrating/driving equipment in the vicinity of existing structures, the following conditions shall be complied with:
    a. Ground vibrations shall be monitored during shoring installation adjacent to the pile driving operation.
    b. Peak particle velocities (PPV) for any single axis shall be limited to ½ inch/second.
    c. In the event any PPV is measured above the specified threshold (½ inch/second) or any settlement is observed, pile driving shall be stopped and corrective actions shall be submitted to the Department for review before resuming pile driving.

20. All foundations shall derive entire support from native undisturbed soils, as recommended and approved by the geologist and soils engineer by inspection.

21. The pile foundation of the existing parking structure will transfer part of the lateral load generated by strong ground shaking to the proposed basement wall(s). The consultants assumed a pile head movement of 0.1 inch, which will result in a uniform lateral surcharge pressure of 150 PSF on the proposed basement walls.

22. The consultants indicated that the installation of the proposed tie-back anchors will not reduce the capacity of the existing pile foundations.

23. The proposed tie-backs shall be setback from the existing piles as shown on Figures 3B and 4B. A minimum distance of 10 feet shall be maintained between existing piles and the bonded zone of the proposed tie-back anchors.

24. Footings supported on approved compacted fill or expansive soil shall be reinforced with a minimum of four (4), ½-inch diameter (#4) deformed reinforcing bars. Two (2) bars shall be placed near the bottom and two (2) bars placed near the top of the footing.

25. Slabs placed on approved compacted fill shall be at least 3½ inches thick and shall be reinforced with ½-inch diameter (#4) reinforcing bars spaced a maximum of 16 inches on center each way.

26. The seismic design shall be based on a Site Class C, as recommended. All other seismic design parameters shall be reviewed by LADBS building plan check.

27. Retaining walls shall be designed for the lateral earth pressures specified in the response to comment #4 and shown on Figures 9 and 10, included in the 10/14/2021 report. The calculation sheets included in the Appendix B of 10/14/2021 do not appear to match the recommended values and were disregarded. All surcharge loads shall be included into the design.

28. All retaining walls shall be provided with a standard surface backdrain system and all drainage shall be conducted in a non-erosive device to the street in an acceptable manner (7013.11).

29. With the exception of retaining walls designed for hydrostatic pressure, all retaining walls shall be provided with a subdrain system to prevent possible hydrostatic pressure behind the wall. Prior to issuance of any permit, the retaining wall subdrain system recommended in the soils report shall be incorporated into the foundation plan which shall be reviewed and approved by the soils engineer of record (1805.4).

30. Installation of the subdrain system shall be inspected and approved by the soils engineer of record and the City grading/building inspector (108.9).

31. Basement walls and floors shall be waterproofed/damp-proofed with an LA City approved "Below-grade" waterproofing/damp-proofing material with a research report number.

32. Prefabricated drainage composites (Miradrain, Geotextiles) may be only used in addition to traditionally accepted methods of draining retained earth.

33. Where the ground water table is lowered and maintained at an elevation not less than 6 inches below the bottom of the lowest floor, or where hydrostatic pressures will not occur, the floor and basement walls shall be damp-proofed. (1803.5.4, 1805.1.3, 1805.2, 1805.3)

34. The structure shall be connected to the public sewer system per P/BC 2020-027.

35. The infiltration facility design and construction shall comply with the minimum requirements specified in the Information Bulletin P/BC 2020-118.

36. The infiltration system shall be constructed at the location shown on the Figure 2 attached to the 06/07/2021 report. The construction of the infiltration system shall be provided under the inspection and approval of the soils engineer.

37. An overflow outlet shall be provided to conduct water to the street in the event that the infiltration system capacity is exceeded. (P/BC 2020-118)

38. Approval for the proposed infiltration system from the Bureau of Sanitation, Department of Public Works shall be secured. A minimum distance of 10 feet (in any direction) shall be provided from adjacent proposed/existing footings to the discharge of the proposed infiltration system. A minimum distance of 10 feet horizontally shall be provided from private property lines to the proposed infiltration system.

39. The dry well area between the blank casing and the surround soils shall be sealed to a minimum depth of 10 feet below the bottom of any adjacent foundation with bentonite slurry (or equivalent) to prevent unintended leakage or horizontal infiltration. An emergency pump shall be provided and properly connected to the dry well in case of disfunction or overflow of the dry well.

40. All concentrated drainage shall be conducted in an approved device and disposed of in a manner approved by the LADBS (7013.10).

41. The soils engineer shall inspect all excavations to determine that conditions anticipated in the report have been encountered and to provide recommendations for the correction of hazards found during grading (7008, 1705.6 & 1705.8).

42. Prior to pouring concrete, a representative of the consulting soils engineer shall inspect and approve the footing excavations. The representative shall post a notice on the job site for the LADBS Inspector and the Contractor stating that the work inspected meets the conditions of the report. No concrete shall be poured until the LADBS Inspector has also inspected and approved the footing excavations. A written certification to this effect shall be filed with the Grading Division upon completion of the work. (108.9 & 7008.2)

43. Prior to excavation an initial inspection shall be called with the LADBS Inspector. During the initial inspection, the sequence of construction; shoring; protection fences; and, dust and traffic control will be scheduled (108.9.1).

44. Installation of shoring shall be performed under the inspection and approval of the soils engineer and deputy grading inspector (1705.6, 1705.8).

45. Prior to the placing of compacted fill, a representative of the soils engineer shall inspect and approve the bottom excavations. The representative shall post a notice on the job site for the LADBS Inspector and the Contractor stating that the soil inspected meets the conditions of the report. No fill shall be placed until the LADBS Inspector has also inspected and approved the bottom excavations. A written certification to this effect shall be included in the final compaction report filed with the Grading Division of the Department. All fill shall be placed under the inspection and approval of the soils engineer. A compaction report together with the approved soil report and Department approval letter shall be submitted to the Grading Division of the Department upon completion of the compaction. An Engineer's Certificate of Compliance with the legal description as indicated in the grading permit and the permit number shall be included (7011.3).

46. No footing/slab shall be poured until the compaction report is submitted and approved by the Grading Division of the Department.

Dan

DAN L. STOICA
Geotechnical Engineer I

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213-482-0480

cc: Langan Engineering, Inc., Project Consultant
LA District Office

CITY OF LOS ANGELES
CALIFORNIA

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# SOILS REPORT APPROVAL LETTER

December 10, 2021

LOG # 115699-02
SOILS/GEOLOGY FILE - 2

MREG 1105 Olive LLC
1150 South Olive Street, Suite 2250
Los Angeles, CA 90015

TRACT: ORD'S SURVEY (M R 53-66/73) // 904 // AMES PROPERTY (M B 12-41)
BLOCK: 77 // --- // ---
LOT(S): FR 8 (Arb.2) / FR 10 (Arb.1) / FR 10 (Arb.2) / FR 10 (Arb.4) // FR A // FR LT A
LOCATION: 1100 S Olive St., 1114-1130 S Olive St., 218-228 W 11th St.

| CURRENT REFERENCE REPORT/LETTER(S) | REPORT No. | DATE OF DOCUMENT | PREPARED BY |
|---|---|---|---|
| Soils Report | 700070701 | 10/25/2021 | Langan Engineering, Inc. |

| PREVIOUS REFERENCE REPORT/LETTER(S) | REPORT No. | DATE OF DOCUMENT | PREPARED BY |
|---|---|---|---|
| Dept. Review Letter | 115699-01 | 07/01/2021 | LADBS |
| Soils Report | 700070701 | 06/08/2021 | Langan Engineering, Inc. |
| Dept. Review Letter | 115699 | 01/14/2021 | LADBS |
| Soils Report | 700070601 | 12/16/2020 | Langan Engineering, Inc. |

The Grading Division of the Department of Building and Safety has reviewed the referenced report that provides recommendations for the proposed construction of a 60-story, 698-foot-tall mixed-use tower and a contiguous 5-level podium structure over 6-level subterranean parking.

The project site includes six lots and it located in the vicinity of existing one- and two-story masonry buildings located at 1127 and 1111 South Hill Street; a 32-story, high-rise tower located at 1155 South Hill Street (AT&T Center Tower); and an existing six-level parking garage located at 1127 – 1143 South Olive Street. In addition, another high-rise tower development is planned at 1105 South Olive Street. The earth materials at the subsurface exploration locations consist of uncertified fill underlain by alluvial deposits. Siltstone and sandstone bedrock of Puente Formation was encountered at approximate depth of 148 feet below ground surface. The consultants recommend to support the proposed building on conventional and/or mat-type foundations bearing on native undisturbed soils.

Engineering analyses provided by Langan Engineering, Inc. are based on field and laboratory testing performed by GeoDesign Inc. and GEOVision Geophysical Services. Langan Engineering, Inc. is accepting responsibility for use of the data in accordance to LABC section 91.7008.5.

The referenced reports are acceptable, provided the following conditions are complied with during site development:

(Note: Numbers in parenthesis ( ) refer to applicable sections of the 2020 City of LA Building Code. P/BC numbers refer the applicable Information Bulletin. Information Bulletins can be accessed on the internet at LADBS.ORG.)

1. Provide a notarized letter from all adjoining property owners allowing tie-back anchors on their property (7006.6).

2. The soils engineer shall review and approve the detailed plans prior to issuance of any permit. This approval shall be by signature on the plans that clearly indicates the soils engineer has reviewed the plans prepared by the design engineer; and, that the plans included the recommendations contained in their reports (7006.1).

3. All recommendations of the reports that are in addition to or more restrictive than the conditions contained herein shall be incorporated into the plans.

4. A copy of the subject and appropriate referenced reports and this approval letter shall be attached to the District Office and field set of plans (7006.1). Submit one copy of the above reports to the Building Department Plan Checker prior to issuance of the permit.

5. A grading permit shall be obtained for all structural fill/retaining wall backfill (106.1.2).

6. All man-made fill shall be compacted to a minimum 90 percent of the maximum dry density of the fill material per the latest version of ASTM D 1557. Where cohesionless soil having less than 15 percent finer than 0.005 millimeters is used for fill, it shall be compacted to a minimum of 95 percent relative compaction based on maximum dry density. Placement of gravel in lieu of compacted fill is only allowed if complying with LAMC Section 91.7011.3.

7. Existing uncertified fill shall not be used for support of footings, concrete slabs or new fill.

8. Drainage in conformance with the provisions of the Code shall be maintained during and subsequent to construction (7013.12).

9. The applicant is advised that the approval of this report does not waive the requirements for excavations contained in the General Safety Orders of the California Department of Industrial Relations (3301.1).

10. Temporary excavations that remove lateral support to the public way, adjacent property, or adjacent structures shall be supported by shoring. Note: Lateral support shall be considered to be removed when the excavation extends below a plane projected downward at an angle of 45 degrees from the bottom of a footing of an existing structure, from the edge of the public way or an adjacent property. (3307.3.1)

11. Where any excavation, not addressed in the approved reports, would remove lateral support (as defined in 3307.3.1) from a public way, adjacent property or structures, a supplemental report shall be submitted to the Grading Division of the Department containing recommendations for shoring, underpinning, and sequence of construction.

12. Prior to the issuance of any permit that authorizes an excavation where the excavation is to be of a greater depth than are the walls or foundation of any adjoining building or structure and located closer to the property line than the depth of the excavation, the owner of the subject site shall provide the Department with evidence that the adjacent property owner has been given a 30-day written notice of such intent to make an excavation (3307.1).

13. The soils engineer shall review and approve the shoring plans prior to issuance of the permit (3307.3.2).

14. Prior to the issuance of the permits, the soils engineer and/or the structural designer shall evaluate the surcharge loads used in the report calculations for the design of the retaining walls and shoring. If the surcharge loads used in the calculations do not conform to the actual surcharge loads, the soil engineer shall submit a supplementary report with revised recommendations to the Department for approval.

15. Unsurcharged temporary excavation may be cut vertical up to 4 feet. For excavations over 4 feet, the portion of the excavation above the vertical cut shall be trimmed back at a uniform gradient not exceeding 1:1 (horizontal to vertical), as recommended.

16. Shoring shall be designed for a minimum triangular distributed lateral earth pressure of 26 PCF (cantilever) or a trapezoidal distributed lateral earth pressure of 24H PSF (braced); all surcharge loads shall be included into the design, as recommended. Total lateral load on shoring piles shall be determined by multiplying the recommended EFP by the pile spacing.

17. Shoring shall be designed for a maximum lateral deflection of 1 inch, provided there are no structures within a 1:1 plane projected up from the base of the excavation. Where a structure is within a 1:1 plane projected up from the base of the excavation, shoring shall be designed for a maximum lateral deflection of ½ inch, or to a lower deflection determined by the consultant that does not present any potential hazard to the adjacent structure.

18. A shoring monitoring program shall be implemented to the satisfaction of the soils engineer

19. In the event shoring soldier piles are installed using vibrating/driving equipment in the vicinity of existing structures, the following conditions shall be complied with:
    a. Ground vibrations shall be monitored during shoring installation adjacent to the pile driving operation.
    b. Peak particle velocities (PPV) for any single axis shall be limited to ½ inch/second.
    c. In the event any PPV is measured above the specified threshold (½ inch/second) or any settlement is observed, pile driving shall be stopped and corrective actions shall be submitted to the Department for review before resuming pile driving.

20. All foundations shall derive entire support from native undisturbed soils, as recommended and approved by the geologist and soils engineer by inspection.

21. Footings supported on approved compacted fill or expansive soil shall be reinforced with a minimum of four (4), ½-inch diameter (#4) deformed reinforcing bars. Two (2) bars shall be placed near the bottom and two (2) bars placed near the top of the footing.

22. Slabs placed on approved compacted fill shall be at least 3½ inches thick and shall be reinforced with ½-inch diameter (#4) reinforcing bars spaced a maximum of 16 inches on center each way.

23. The seismic design shall be based on a Site Class C, as recommended. All other seismic design parameters shall be reviewed by LADBS building plan check.

24. Retaining walls shall be designed for the lateral earth pressures specified in the response to comment #3 and shown on Figures 9 and 10, included in the 10/14/2021 report. The calculation sheets included in the Appendix B of 10/14/2021 do not appear to match the recommended values and were disregarded. All surcharge loads shall be included into the design.

25. All retaining walls shall be provided with a standard surface backdrain system and all drainage shall be conducted in a non-erosive device to the street in an acceptable manner (7013.11).

26. With the exception of retaining walls designed for hydrostatic pressure, all retaining walls shall be provided with a subdrain system to prevent possible hydrostatic pressure behind the wall. Prior to issuance of any permit, the retaining wall subdrain system recommended in the soils report shall be incorporated into the foundation plan which shall be reviewed and approved by the soils engineer of record (1805.4).

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33. The infiltration system shall be constructed at the location shown on the Figure 2 attached to the 06/08/2021 report. The construction of the infiltration system shall be provided under the inspection and approval of the soils engineer.

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