Appendix IS-2

Geotechnical Investigation

GEOTECHNICAL INVESTIGATION

PROPOSED HIGH-RISE DEVELOPMENT
5700 & 5750 WEST WILSHIRE BOULEVARD
LOS ANGELES, CALIFORNIA

TRACT: TR 5798
LOTS: 1, 44-51, 85-92, 127-134, 290-293,
FR 2-8, FR 52, FR 93, FR 135, AND
VAC 86-1092114
ARB: 1 AND 2

GEOCON
WEST, INC.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

ONNI CONTRACTING (CALIFORNIA), INC.
LOS ANGELES, CA

PROJECT NO. W1038-06-01A

MARCH 16, 2020

GEOCON
WEST, INC.
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. W1038-06-01A
March 16, 2020

Ms. Kirsten Morris
ONNI Contracting (California), Inc.
315 West 9th Street, Suite 801
Los Angeles, California 90015

Subject: GEOTECHNICAL INVESTIGATION
PROPOSED HIGH-RISE DEVELOPMENT
5700 & 5750 WEST WILSHIRE BOULEVARD
LOS ANGELES, CALIFORNIA
TRACT: TR 5798, LOTS: 1, 44-51, 85-92, 127-134, 290-293, FR 2-8, FR 52, FR 93, FR 135, AND VAC 86-1092114, ARB: 1 AND 2

Dear Ms. Morris:

In accordance with your authorization of our proposal dated November 25, 2019, we have performed a geotechnical investigation for the proposed high-rise development located at 5700 & 5750 West Wilshire Boulevard in the City of Los Angeles, California. The accompanying report presents the findings of our study, and our conclusions and recommendations pertaining to the geotechnical aspects of proposed design and construction. Based on the results of our investigation, it is our opinion that the site can be developed as proposed, provided the recommendations of this report are followed and implemented during design and construction.

If you have any questions regarding this report, or if we may be of further service, please contact the undersigned.

Very truly yours,

GEOCON WEST, INC.

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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a geotechnical investigation for the proposed high-rise development located at 5700 and 5750 West Wilshire Boulevard in the City of Los Angeles, California (see Vicinity Map, Figure 1). The purpose of the investigation was to evaluate subsurface soil and geologic conditions underlying the site and, based on conditions encountered, to provide conclusions and recommendations pertaining to the geotechnical aspects of proposed design and construction.

The scope of this investigation included a site reconnaissance, review of prior reports prepared for the site, field exploration, laboratory testing, engineering analysis, and the preparation of this report. The site was explored on February 14 and 15, 2020 by excavating one 4⅞-inch diameter boring to a depth of approximately 150 feet below existing ground surface using a truck-mounted mud-rotary drilling machine. A geophysical survey consisting of down-hole suspension PS logging was performed in the boring as a part of the site exploration. The approximate location of the exploratory boring is depicted on the Site Plan (see Figure 2A). A detailed discussion of the field investigation, including the boring log, is presented in Appendix A.

Laboratory tests were performed on selected soil samples obtained during the investigation to determine pertinent physical and chemical soil properties. Appendix B presents a summary of the laboratory test results.

The recommendations presented herein are based on analyses of the data obtained during our investigation, as well as the data obtained during the previous geotechnical investigation by LeRoy Crandall and Associates, and our experience with similar soil and geologic conditions. The prior investigation is summarized in Section 3, Background Review. References reviewed to prepare this report are provided in the *List of References* section.

If project details vary significantly from those described herein, Geocon should be contacted to determine the necessity for review and possible revision of this report.

2. SITE AND PROJECT DESCRIPTION

The subject site is located at 5700 and 5750 West Wilshire Boulevard in the City of Los Angeles, California. The site consists of 32 adjacent lots currently occupied by two six-story structures, each underlain by three subterranean levels, which were constructed in the 1980's. The existing structures are supported on a driven pile foundation system with a 28-inch concrete mat slab. The property is bisected by an asphalt paved cul-de-sac, Courtyard Place (Sierra Bonita Avenue). The site is bounded by West Wilshire Boulevard to the north, by 8th Street to the south, by South Masselin Avenue to the east, and by South Curson Avenue to the west. The property is relatively level, and surface water drainage at the site appears to be by sheet flow along the existing ground contours to the city streets. Vegetation on-site consists of trees and shrubs typical of confined to planter area.

Based on the information provided by the Client, it is our understanding that the southern portion of the existing structures will remain in place. However, northern portions of the existing structures will be partially demolished for the construction of two high-rise office towers. The east tower (5700 Wilshire) is proposed to be 38 stories, while the west tower (5750 Wilshire) is proposed to be 33 stories in height. It is our understanding that the proposed towers will be constructed as deep as the existing subterranean levels and will not extend deeper. It is desired to support the proposed towers on new foundations constructed at the lowest existing subterranean level. Based on the preliminary concept plans provided to us, it is anticipated that the two towers will be linked together in several of the upper floors above Courtyard Place. The proposed development is depicted on the Site Plan and Cross Section (see Figures 2A and 2B).

Based on information provided by the project structural engineer, Glotman Simpson, it is anticipated that loads underneath the proposed tower cores will be up to a total of 265,000 kips, and column loads may be up to 12,000 kips elsewhere.

The subject site is located adjacent to the proposed alignment of the Metro Purple Line Subway Extension. Based on our experience, the Metropolitan Transportation Authority (MTA) may require special studies or analyses to evaluate the impact of the proposed construction of the towers on the subway tunnels. Any additional studies required to satisfy MTA can be provided under separate cover, if necessary.

Once the design phase and foundation loading configuration proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, if necessary. Any changes in the design, location or elevation of any structure, as outlined in this report, should be reviewed by this office. Geocon should be contacted to determine the necessity for review and possible revision of this report.

3. BACKGROUND REVIEW

As a part of this investigation, we reviewed a prior geotechnical report and other associated documents for the subject site provided to us by the client. An abbreviated list of the documents we obtained is as follows:

> *Report of Foundation Investigation, Proposed Office Structures and Townhouses, Museum Square, Wilshire Boulevard between Curson Street and Masselin Avenue, Los Angeles, California*, prepared by LeRoy Crandall and Associates, dated March 12, 1984;
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> *Supplementary Information, Driven Pile Capacities, Proposed Wilshire Courtyard, Wilshire Boulevard and Sierra Bonita Avenue, Los Angeles, California*, prepared by LeRoy Crandall and Associates, dated December 20, 1984;
>
> *Supplementary Information for Excavation and Shoring, Proposed Wilshire Courtyard, Wilshire Boulevard and Sierra Bonita Avenue, Los Angeles, California*, prepared by LeRoy Crandall and Associates, dated February 25, 1985;
>
> *Lateral Pressures on Shoring, Proposed Wilshire Courtyard, Wilshire Boulevard and Sierra Bonita Avenue, Los Angeles, California*, prepared by LeRoy Crandall and Associates, dated April 24, 1985;
>
> *Lateral Pressures on Shoring, Proposed Wilshire Courtyard, Wilshire Boulevard and Sierra Bonita Avenue, Los Angeles, California*, prepared by LeRoy Crandall and Associates, dated May 1, 1985;
>
> *Driven Pile Capacities, Proposed Wilshire Courtyard, Wilshire Boulevard and Sierra Bonita Avenue, Los Angeles, California*, prepared by LeRoy Crandall and Associates, dated July 9, 1985;
>
> *Report of Supplementary Pile Load Tests and Modification of Pile Capacities, Proposed Wilshire Courtyard, Wilshire Boulevard between Masselin and Curson Avenue, Los Angeles, California*, prepared by LeRoy Crandall and Associates, dated August 29, 1985;
>
> *Lateral Earth Pressure on North Basement Wall, Proposed Wilshire Courtyard, Wilshire Boulevard between Masselin and Curson Avenue, Los Angeles, California*, prepared by LeRoy Crandall and Associates, dated September 23, 1985.

A prior investigation of the subject site was performed in 1984 by LeRoy Crandall and Associates and included the excavation and logging of 18 borings. The locations of the prior borings are indicated on the Site Plan (Figure 2A). The borings were excavated to depths between 49 and 81 feet below the existing ground surface. Groundwater was encountered in three of the borings at depths of 8 to 9 feet below the ground surface.

Geocon West, Inc. has reviewed the referenced report by LeRoy Crandall and Associates, and the recommendations presented herein are based on analysis of the subsurface and laboratory data obtained from the prior investigations by LeRoy Crandall and Associates, as well as our own subsurface and laboratory data. Furthermore, we assume responsibility for the utilization of the exploration and laboratory data presented within the geotechnical report by LeRoy Crandall and Associates. Copies of the reports prepared by LeRoy Crandall and Associates are provided in Appendix C.

4. GEOLOGIC SETTING

The site is located in the north-central portion of the Los Angeles Basin, a coastal plain bounded by the Santa Monica Mountains on the north, the Elysian Hills and Repetto Hills on the northeast, the Puente Hills and Whittier Fault on the east, the Palos Verdes Peninsula and Pacific Ocean on the west and south, and the Santa Ana Mountains and San Joaquin Hills on the southeast. The basin is underlain by a deep structural depression which has been filled by both marine and continental sedimentary deposits underlain by a basement complex of igneous and metamorphic composition (Yerkes et al., 1965). Regionally, the site is located within the northern portion of the Peninsular Ranges geomorphic province. This geomorphic province is characterized by northwest-trending physiographic and geologic features such as the nearby Newport-Inglewood Fault Zone located approximately 1.9 miles to the west (California Geological Survey [CGS], 2014; CGS, 2017a).

5. SOIL AND GEOLOGIC CONDITIONS

Based on our field investigation and published geologic maps of the area, the site is underlain by artificial fill and Pleistocene age alluvial fan deposits consisting of silt, sand and silty sand (CGS, 2012, Dibblee, 1991). Detailed stratigraphic profiles of the materials encountered at the site are provided on the boring logs in Appendix A.

5.1 Artificial Fill

Artificial fill was encountered in our field explorations to a maximum depth of 8 feet below the existing ground surface. The artificial fill generally consists of gray sandy clay, clayey sand and sand. The artificial fill is characterized as moist and firm or loose. The fill is likely the result of past grading or construction activities at the site. Deeper fill may exist between excavations and in other portions of the site that were not directly explored.

5.2 Older Alluvial Deposits

Pleistocene age alluvial fan deposits were encountered beneath the fill. The older alluvial deposits generally consist of gray to dark gray and very dark brown to black silt, sandy silt, silty sand and sand with varying amounts of gravel. The older alluvium is characterized as predominately fine- grained, moist to wet, and stiff to hard or dense to very dense.

6. GROUNDWATER

Based on a review of the Seismic Hazard Evaluation Report for the Hollywood Quadrangle (California Division of Mines and Geology [CDMG], 1998), the historically highest groundwater level in the area is less than 10 feet beneath the existing ground surface. Groundwater information presented in this document is generated from data collected in the early 1900's to the late 1990s. Based on current groundwater basin management practices, it is unlikely that groundwater levels will ever exceed the historic high levels.

Groundwater was encountered at 31 feet below the existing ground surface in our boring drilled to a maximum depth of 150 feet beneath the existing ground surface. Groundwater was also encountered at depths of 8 to 9 feet in three borings previously drilled in 1984 (LeRoy Crandall & Associates, 1984). Considering the historic high groundwater level and the depth to groundwater encountered in our borings and the previous borings at the site, groundwater may be encountered during construction. Also, it is not uncommon for groundwater levels to vary seasonally or for groundwater seepage conditions to develop where none previously existed, especially in impermeable fine-grained soils which are heavily irrigated or after seasonal rainfall. In addition, recent requirements for stormwater infiltration could result in shallower seepage conditions in the immediate site vicinity. Proper surface drainage of irrigation and precipitation will be critical for future performance of the project. Preliminary recommendations for drainage are provided in the S*urface Drainage* section of this report (see Section 8.22).

7. GEOLOGIC HAZARDS

7.1 Surface Fault Rupture

The numerous faults in Southern California include Holocene-active, pre-Holocene, and inactive faults. The criteria for these major groups are based on criteria developed by the California Geological Survey (CGS, formerly known as CDMG) for the Alquist-Priolo Earthquake Fault Zone Program (CGS, 2018a). By definition, a Holocene-active fault is one that has had surface displacement within Holocene time (about the last 11,700 years). A pre-Holocene fault has demonstrated surface displacement during Quaternary time (approximately the last 1.6 million years), but has had no known Holocene movement. Faults that have not moved in the last 1.6 million years are considered inactive.

The site is not within a state-designated Alquist-Priolo Earthquake Fault Zone (CGS, 2020a; 2020b; 2014) nor a city-designated Preliminary Fault Rupture Study Area (City of Los Angeles, 2020) for surface fault rupture hazards. No Holocene-active or pre-Holocene faults with the potential for surface fault rupture are known to pass directly beneath the site. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed development is considered low. However, the site is located in the seismically active Southern California region, and could be subjected to moderate to strong ground shaking in the event of an earthquake on one of the many active Southern California faults. The faults in the vicinity of the site are shown in Figure 3, Regional Fault Map.

The closest active fault to the site is the Newport-Inglewood Fault Zone, located approximately 1.9 miles to the southwest (CGS, 2014). Other nearby active faults are the Hollywood Fault, the Santa Monica Fault and the Raymond Fault located approximately 2.5 miles northwest and 3.6 miles west, and 7.2 miles to the northeast of the site, respectively (CGS, 2014; CGS, 2018b). The active San Andreas Fault Zone is located approximately 37 miles northeast of the site (Ziony and Jones, 1989).

Several buried thrust faults, commonly referred to as blind thrusts, underlie the Los Angeles Basin at depth. These faults are not exposed at the ground surface and are typically identified at depths greater than 3.0 kilometers. The October 1, 1987 M_w 5.9 Whittier Narrows earthquake and the January 17, 1994 M_w 6.7 Northridge earthquake were a result of movement on the Puente Hills Blind Thrust and the Northridge Thrust, respectively. These thrust faults and others in the Los Angeles area are not exposed at the surface and do not present a potential surface fault rupture hazard at the site; however, these deep thrust faults are considered active features capable of generating future earthquakes that could result in moderate to significant ground shaking at the site.

7.2 Seismicity

As with all of Southern California, the site has experienced historic earthquakes from various regional faults. The seismicity of the region surrounding the site was formulated based on research of an electronic database of earthquake data. The epicenters of recorded earthquakes with magnitudes equal to or greater than 5.0 in the site vicinity are depicted on Figure 4, Regional Seismicity Map. A partial list of moderate to major magnitude earthquakes that have occurred in the Southern California area within the last 100 years is included in the table on the following page.

LIST OF HISTORIC EARTHQUAKES

Earthquake (Oldest to Youngest)	Date of Earthquake	Magnitude	Distance to Epicenter (Miles)	Direction to Epicenter
Near Redlands	July 23, 1923	6.3	63	E
Long Beach	March 10, 1933	6.4	38	SE
Tehachapi	July 21, 1952	7.5	75	NW
San Fernando	February 9, 1971	6.6	24	N
Whittier Narrows	October 1, 1987	5.9	16	E
Sierra Madre	June 28, 1991	5.8	24	ENE
Landers	June 28, 1992	7.3	110	E
Big Bear	June 28, 1992	6.4	88	E
Northridge	January 17, 1994	6.7	15	NW
Hector Mine	October 16, 1999	7.1	125	ENE
Ridgecrest	July 5, 2019	7.1	125	NNE

The site could be subjected to strong ground shaking in the event of an earthquake. However, this hazard is common in Southern California and the effects of ground shaking can be mitigated if the proposed structures are designed and constructed in conformance with current building codes and engineering practices.

7.3 Site Specific Shear Wave Velocity

During the site exploration program, GeoVision collected geophysical measurements for the determination of shear wave velocities as a function of depth. Suspension velocity measurements were taken in the uncased boring using an OYO PS Suspension Logging System. In-situ horizontal shear and compression wave velocity measurements were collected at 1.6-foot intervals to a depth of 136.2 feet below existing ground surface. The methodologies used by GeoVision for the data acquisition and analysis are presented in the March 12, 2020 report by GeoVision. A copy of the report is provided in Appendix D.

Based on the results of the suspension P-S logging performed by GeoVision Geophysical Services, the site-specific soil shear wave velocity for the soil to a depth of 30 meters below the ground surface (Vs30) is approximately 283 meters per second. According to the discussion in Section 1613.3.2 of the 2019 CBC and Table 20.3-1 of ASCE 7-16 and the shear wave velocity, the site falls within the boundaries of a Site Class “D”.

7.4 Seismic Design Criteria

The following table summarizes summarizes site-specific design criteria obtained from the 2019 California Building Code (CBC; Based on the 2018 International Building Code [IBC] and ASCE 7-16), Chapter 16 Structural Design, Section 1613 Earthquake Loads. The data was calculated using the online application *Seismic Design Maps*, provided by OSHPD. The short spectral response uses a period of 0.2 second. We evaluated the Site Class based on the discussion in Section 1613.2.2 of the 2019 CBC and Table 20.3-1 of ASCE 7-16. The values presented below are for the risk-targeted maximum considered earthquake (MCE_R).

2019 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2019 CBC Reference
Site Class	D	Section 1613.2.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	2.036g	Figure 1613.2.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.725g	Figure 1613.2.1(2)
Site Coefficient, F_A	1	Table 1613.2.3(1)
Site Coefficient, F_V	1.7*	Table 1613.2.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	2.036g	Section 1613.2.3 (Eqn 16-36)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	1.233g*	Section 1613.2.3 (Eqn 16-37)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.357g	Section 1613.2.4 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.822g*	Section 1613.2.4 (Eqn 16-39)

Note:

*Per Section 11.4.8 of ASCE/SEI 7-16, a ground motion hazard analysis shall be performed for projects for Site Class "E" sites with Ss greater than or equal to 1.0g and for Site Class "D" and "E" sites with S1 greater than 0.2g. Section 11.4.8 also provides exceptions which indicates that the ground motion hazard analysis may be waived provided the exceptions are followed. Using the code based values presented in the table above, in lieu of a performing a ground motion hazard analysis, requires the exceptions outlined in ASCE 7-16 Section 11.4.8 be followed.

The table below presents the mapped maximum considered geometric mean (MCE_G) seismic design parameters for projects located in Seismic Design Categories of D through F in accordance with ASCE 7-16.

ASCE 7-16 PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-16 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.871g	Figure 22-7
Site Coefficient, F_{PGA}	1.1	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.958	Section 11.8.3 (Eqn 11.8-1)

The Maximum Considered Earthquake Ground Motion (MCE) is the level of ground motion that has a 2 percent chance of exceedance in 50 years, with a statistical return period of 2,475 years. According to the 2019 California Building Code and ASCE 7-16, the MCE is to be utilized for the evaluation of liquefaction, lateral spreading, seismic settlements, and it is our understanding that the intent of the Building code is to maintain “Life Safety” during a MCE event. The Design Earthquake Ground Motion (DE) is the level of ground motion that has a 10 percent chance of exceedance in 50 years, with a statistical return period of 475 years.

Deaggregation of the MCE peak ground acceleration was performed using the USGS online Unified Hazard Tool, 2014 Conterminous U.S. Dynamic edition (v4.2.0). The result of the deaggregation analysis indicates that the predominant earthquake contributing to the MCE peak ground acceleration is characterized as a 6.8 magnitude event occurring at a hypocentral distance of 8.33 kilometers from the site.

Deaggregation was also performed for the Design Earthquake (DE) peak ground acceleration, and the result of the analysis indicates that the predominant earthquake contributing to the DE peak ground acceleration is characterized as a 6.68 magnitude occurring at a hypocentral distance of 11.91 kilometers from the site.

Conformance to the criteria in the above tables for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a large earthquake occurs. The primary goal of seismic design is to protect life, not to avoid all damage, since such design may be economically prohibitive.

7.5 Liquefaction Potential

Liquefaction is a phenomenon in which loose, saturated, relatively cohesionless soil deposits lose shear strength during strong ground motions. Primary factors controlling liquefaction include intensity and duration of ground motion, gradation characteristics of the subsurface soils, in-situ stress conditions, and the depth to groundwater. Liquefaction is typified by a loss of shear strength in the liquefied layers due to rapid increases in pore water pressure generated by earthquake accelerations.

The current standard of practice, as outlined in the "Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California" and "Special Publication 117A, Guidelines for Evaluating and Mitigating Seismic Hazards in California" requires liquefaction analysis to a depth of 50 feet below the lowest portion of the proposed structure. Liquefaction typically occurs in areas where the soils below the water table are composed of poorly consolidated, fine to medium-grained, primarily sandy soil. In addition to the requisite soil conditions, the ground acceleration and duration of the earthquake must also be of a sufficient level to induce liquefaction.

A review of the State of California Seismic Hazard Zones Map for the Hollywood Quadrangle (CDMG, 1999; CGS, 2014) indicates that the site is not located in an area designated as having a potential for liquefaction. The site is underlain by Pleistocene age alluvial fan deposits that are dense to very dense and not prone to liquefaction. Based on these considerations, the potential for liquefaction and earthquake-induced settlement is considered very low.

7.6 Slope Stability

The site is relatively level and the topography in the immediate vicinity slopes gently to the east-southeast. The site is not located within a City of Los Angeles Hillside Grading Area or within a Hillside Ordinance Area (City of Los Angeles, 2020). The County of Los Angeles Safety Element (Leighton, 1990), indicates the site is not located within an area identified as a "Hillside Area" or an area having a potential for slope instability. Additionally, the site is not within an area identified as having a potential for seismic slope instability (CDMG, 1999; CGS, 2014). There are no known landslides near the site, nor is the site in the path of any known or potential landslides. Therefore, the potential for slope stability hazards to adversely affect the proposed development is considered low.

7.7 Earthquake-Induced Flooding

Earthquake-induced flooding is inundation caused by failure of dams or other water-retaining structures due to earthquakes. The Los Angeles County Safety Element (Leighton, 1990) indicates that the site is located within the Mullholland Dam inundation area. However, this reservoir, as well as others in California, are continually monitored by various governmental agencies (such as the State of California Division of Safety of Dams and the U.S. Army Corps of Engineers) to guard against the threat of dam failure. Current design, construction practices, and ongoing programs of review, modification, or total reconstruction of existing dams are intended to ensure that all dams are capable of withstanding the maximum considered earthquake (MCE) for the site. Therefore, the potential for inundation at the site as a result of an earthquake-induced dam failure is considered low.

7.8 Tsunamis, Seiches, and Flooding

The site is not located within a coastal area. Therefore, tsunamis are not considered a significant hazard at the site.

Seiches are large waves generated in enclosed bodies of water in response to ground shaking. No major water-retaining structures are located immediately up-gradient from the project site. Therefore, flooding resulting from a seismically induced seiche is considered unlikely.

The site is located within a Flood Zone X (FEMA, 2020; LACDPW, 2020).

7.9 Oil Fields & Methane Potential

Based on a review of the California Geologic Energy Management Division (CalGEM) Well Finder Website, the site is located within the Salt Lake Oil Field and the Salt Lake South Oil Field (CalGEM, 2020). Utah California Consolidated Co. Oil & Gas No. 101 (API0403714694) well is identified as plugged and abandoned located within the eastern portion of the parcel. Due to the voluntary nature of record reporting by the oil well drilling companies, wells may be improperly located or not shown on the location map and other undocumented wells could be encountered during construction. The Chevron USA well, and any wells encountered during construction, will need to be properly abandoned in accordance with the current requirements of the CalGEM.

The site is located within the boundaries of a Methane Buffer Zone, as defined by the City of Los Angeles (2020). A methane study will be required by the city prior to site development. A methane specialist should be contacted to perform a site-specific methane study and provide design recommendations as necessary.

7.10 Subsidence

Subsidence occurs when a large portion of land is displaced vertically, usually due to the withdrawal of groundwater, oil, or natural gas. Soils that are particularly subject to subsidence include those with high silt or clay content. Subsidence commonly occurs in such small magnitude, and over such large areas, that it is generally imperceptible at an individual locality. Accordingly, it affects only regionally extensive structures sensitive to slight elevation changes such as canals and pipelines. The rate of elevation change is usually uniform over a large enough area that is does not result in differential settlements that would cause damage to individual buildings. Therefore, the potential for ground subsidence to adversely impact the site is considered low.

8. CONCLUSIONS AND RECOMMENDATIONS

8.1 General

8.1.1 It is our opinion that neither soil nor geologic conditions were encountered during the investigation that would preclude the construction of the proposed development provided the recommendations presented herein are followed and implemented during design and construction.

8.1.2 Up to 8 feet of existing artificial fill was encountered during the site investigation. The existing fill encountered is believed to be the result of past grading and construction activities at the site. Deeper fill may exist in other areas of the site that were not directly explored. It is our opinion that the existing fill in its present condition, is not suitable for direct support of proposed foundations or slabs. The existing fill and site soils are suitable for re-use as engineered fill provided the recommendations in the Grading section of this report are followed (see Section 8.5). It is anticipated that competent alluvial soils will be exposed at the proposed foundation level.

8.1.3 Groundwater was encountered at a depth of 31 feet below existing ground surface. The existing subterranean levels extend to a depth of approximately 32 feet below ground surface. Based on conditions encountered at the time of exploration, groundwater should be anticipated during construction activities and temporary dewatering measures will be required to mitigate groundwater during excavation and construction. Recommendations for temporary dewatering are discussed in Section 8.4 of this report.

8.1.4 Based on a review of the Seismic Hazard Evaluation of the Hollywood 7.5 Minute Quadrangle, Los Angeles County, California, the historically highest groundwater level is indicated to be approximately 10 feet beneath the ground surface. Assuming a design water level of approximately 10 feet below the existing ground surface, the proposed structure must be designed for hydrostatic pressure for any portion of the structure below a depth of 10 feet. The hydrostatic design will result in uplift forces on the structure that must be resisted by counterweight or structural design measures. The recommended floor slab uplift pressure to be used in design would be 62.4(H) in units of pounds per square foot (psf), where "H" is the height of the water above the bottom of the foundation in feet. If the proposed structure does not provide sufficient dead load to resist the buoyant forces then uplift mitigation will be required.

8.1.5 Based on these considerations, engineering analyses, and information provided by the project structural engineer, the proposed towers may be supported on a deepened foundation system consisting of auger-cast pressure grouted displacement (APGD) piles. The APGD piles have the benefit of not generating soil spoils; however, the City of Los Angeles will require a comprehensive load testing program. The Client should be aware that APGD piles are designed and installed by a specialty geotechnical contractor. Recommendations for the design of APGD piles are provided in Section 8.7.

8.1.6 The client should be aware that a methane mitigation system is required for this project. A qualified methane consultant should be retained for the design of the mitigation system.

8.1.7 A methane barrier will be installed below the proposed structures and pile penetrations through these barriers are undesirable. Therefore, pile caps can be constructed on top of the proposed piles and a reinforced concrete mat foundation be utilized above the barriers, if possible. This would allow for a vertical load transfer of the mat foundation to the pile foundations without penetrating through the barriers. The mat foundation should be designed to derive vertical support from the piles and may develop lateral resistance at the foundation perimeter, as well as by friction beneath the mat foundation, if necessary. If the mat foundation is not structurally connected to the piles, the piles would not be able to contribute lateral capacity or uplift resistance to the mat foundation.

8.1.8 Due to the depth of the excavation and the proximity to the property lines, city streets, substructures, and adjacent offsite structures, excavation will require shoring measures in order to provide a stable excavation. Where shoring is required, it is recommended that a soldier pile shoring system be utilized. In addition, where the proposed excavation will be deeper than and adjacent to an offsite structure, the proposed shoring should be designed to resist the surcharge imposed by the adjacent offsite structure. Recommendations for *Shoring* are provided in Section 8.16 of this report.

8.1.9 Due to the nature of the proposed design and intent for a subterranean level, permanent waterproofing of subterranean walls and slabs should be installed. Particular care should be taken in the design and installation of waterproofing to avoid moisture problems, or actual water seepage into the structure through any normal shrinkage cracks which may develop in the concrete walls, floor slab, foundations and/or construction joints. The design and inspection of the waterproofing is not the responsibility of the geotechnical engineer. A waterproofing consultant should be retained in order to recommend a product or method, which would provide protection to subterranean walls, floor slabs and foundations. In addition, an experienced waterproofing inspector should be retained to check proper installation of the system during construction.

8.1.10 Once the design and foundation loading configuration for the proposed structure proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, if necessary. Based on the final foundation loading configurations, the potential for settlement should be reevaluated by this office.

8.1.11 Any changes in the design, location or elevation, as outlined in this report, should be reviewed by this office. Geocon should be contacted to determine the necessity for review and possible revision of this report.

8.2 Soil and Excavation Characteristics

8.2.1 The in-situ soils can be excavated with moderate effort using conventional excavation equipment. Moderate to excessive caving should be anticipated in unshored excavations, especially where granular soils are exposed. Any exposed asphaltic sands will have a tendency to creep, especially when warm. Uncased boring excavations will slowly squeeze shut. Uncased excavations that extend into asphaltic sands will require casing to maintain the boring diameter.

8.2.2 It is the responsibility of the contractor to ensure that all excavations and trenches are properly shored and maintained in accordance with applicable OSHA rules and regulations to maintain safety and maintain the stability of adjacent existing improvements.

8.2.3 All onsite excavations must be conducted in such a manner that potential surcharges from existing structures, construction equipment, and vehicle loads are resisted. The surcharge area may be defined by a 1:1 projection down and away from the bottom of an existing foundation or vehicle load. Penetrations below this 1:1 projection will require special excavation measures such as sloping and shoring. Excavation recommendations are provided in the *Temporary Excavations* section of this report (see Section 8.15).

8.2.4 The upper 5 feet of existing site soils encountered during the investigation are considered to have a “very low” expansive potential (EI = 18) and are classified as “non-expansive” in accordance with the 2019 California Building Code (CBC) Section 1803.5.3. The recommendations presented herein assume that the building foundations and slabs will derive support in these materials.

8.3 Minimum Resistivity, pH, and Water-Soluble Sulfate

8.3.1 Potential of Hydrogen (pH) and resistivity testing as well as chloride content testing were performed on representative samples of soil to generally evaluate the corrosion potential to surface utilities. The tests were performed in accordance with California Test Method Nos. 643 and 422 and indicate that the soils are considered "severely corrosive" with respect to corrosion of buried ferrous metals on site. The results are presented in Appendix B (Figure B17) and should be considered for design of underground structures. Due to the corrosive potential of the soils, it is recommended that PVC, ABS or equivalent plastic piping be considered in lieu of cast-iron for sewer pipes, subdrains and retaining wall drains in direct contact with the site soils.

8.3.2 Laboratory tests were performed on representative samples of the site materials to measure the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate tests are presented in Appendix B (Figure B17) and indicate that the on-site materials possess a sulfate exposure class of "S0" to concrete structures as defined by 2019 CBC Section 1904 and ACI 318-14 Table 19.3.1.1.

8.3.3 Geocon West, Inc. does not practice in the field of corrosion engineering and mitigation. If corrosion sensitive improvements are planned, it is recommended that a corrosion engineer be retained to evaluate corrosion test results and incorporate the necessary precautions to avoid premature corrosion of buried metal pipes and concrete structures in direct contact with the soils.

8.4 Temporary Dewatering

8.4.1 Groundwater was encountered during site exploration at a depth of 31 feet below existing ground surface. Based on the conditions encountered at the time of exploration, groundwater is anticipated to be encountered during construction activities. The depth to groundwater at the time of construction can be further verified during the installation of the initial dewatering well or shoring piles. If groundwater is present above the depth of the proposed foundation excavation, temporary dewatering will be necessary to maintain a safe working environment during excavation and construction activities.

8.4.2 It is recommended that a qualified dewatering consultant be retained to design the dewatering system and determine the design flow rates for dewatering. The dewatering consultant should also provide the minimum depth that the temporary dewatering be effective to, and also the potential effects of temporary dewatering on adjacent structures and the public right of way. Temporary dewatering may consist of perimeter wells with interior well points as well as gravel filled trenches (French drains) placed adjacent to the shoring system and interior of the site. The number and locations of the wells or French drains can be adjusted during excavation activities as necessary to collect and control any encountered seepage. The French drains will then direct the collected seepage to a sump where it will be pumped out of the excavation.

8.4.3 The embedment of perimeter shoring piles should be deepened as necessary to take into account any required excavations necessary to place an adjacent French drain system, or sub-slab drainage system, should it be deemed necessary. It is not anticipated that a perimeter French drain will be more than 24 inches in depth below the proposed excavation bottom. If a French drain is to remain on a permanent basis, it must be lined with filter fabric to prevent soil migration into the gravel.

8.5 Grading

8.5.1 Grading is anticipated to include preparation of the subgrade, excavation for proposed foundations and utility trenches, as well as placement of backfill for walls, ramps, and trenches.

8.5.2 A preconstruction conference should be held at the site prior to the beginning of grading operations with the owner, contractor, civil engineer, geotechnical engineer, and building official in attendance. Special soil handling requirements can be discussed at that time.

8.5.3 Earthwork should be observed, and compacted fill tested by representatives of Geocon West, Inc. The existing fill and older alluvial soils encountered during exploration is suitable for re-use as an engineered fill, provided any encountered oversize material (greater than 6 inches) and any encountered deleterious debris is removed.

8.5.4 Grading should commence with the removal of existing vegetation and existing improvements from the area to be graded. Deleterious debris such as wood and root structures should be exported from the site and should not be mixed with the fill soils. Asphalt and concrete should not be mixed with the fill soils unless approved by the Geotechnical Engineer. Existing underground improvements planned for removal should be completely excavated and the resulting depressions properly backfilled in accordance with the procedures described herein. Once a clean excavation bottom has been established it must be approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.) and the City of Los Angeles Inspector.

8.5.5 Due to the potential for high-moisture content soils at the excavation bottom, stabilization measures may have to be implemented to prevent excessive disturbance to the excavation bottom. Should this condition exist, rubber tire equipment should not be allowed in the excavation bottom until it is stabilized or extensive soil disturbance could result.

8.5.6 Subgrade stabilization may consist of introducing a thin lift of 3- to 6-inch diameter crushed angular rock into the soft excavation bottom. The use of crushed concrete will also be acceptable. The crushed rock should be spread thinly across the excavation bottom and pressed into the soils by track rolling or wheel rolling with heavy equipment. It is very important that voids between the rock fragments are not created so the rock must be thoroughly pressed or blended into the soils. All subgrade soils must be properly compacted and proof-rolled in the presence of the Geotechnical Engineer (a representative of Geocon West, Inc.).

8.5.7 The City of Los Angeles Department of Building and Safety requires a minimum compactive effort of 95 percent of the laboratory maximum dry density in accordance with ASTM D 1557 (latest edition) where the soils placed as fill have less than 15 percent finer than 0.005 millimeters. Soils with more than 15 percent finer than 0.005 millimeters may be compacted to 90 percent of the laboratory maximum dry density in accordance with ASTM D 1557 (latest edition). Based on the soils encountered during this investigation, it is anticipated that 95 percent relative compaction will be required. All fill and backfill soils should be placed in horizontal loose layers approximately 6 to 8 inches thick, moisture conditioned to optimum moisture content, and properly compacted to the required degree of compaction in accordance with ASTM D 1557 (latest edition).

8.5.8 Although not anticipated for this project, all imported fill shall be observed, tested, and approved by Geocon West, Inc. prior to bringing soil to the site. Rocks larger than 6 inches in diameter shall not be used in the fill. If necessary, import soils used as structural fill should have an expansion index less than 20 and soil corrosivity properties that are equally or less detrimental to that of the existing onsite soils (see Figure B17).

8.5.9 Utility trenches should be properly backfilled in accordance with the requirements of the Green Book (latest edition). The pipe should be bedded with clean sands (Sand Equivalent greater than 30) to a depth of at least 1 foot over the pipe, and the bedding material must be inspected and approved in writing by the Geotechnical Engineer (a representative of Geocon). The use of gravel is not acceptable unless used in conjunction with filter fabric to prevent the gravel from having direct contact with soil. The remainder of the trench backfill may be derived from onsite soil or approved import soil, compacted as necessary, until the required compaction is obtained. The use of minimum 2-sack slurry as backfill is also acceptable provided it is placed in accordance with City requirements (IB-P-BC 2014-121 CLSM). Prior to placing any bedding materials or pipes, the excavation bottom must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon).

8.5.10 All trench and foundation excavation bottoms must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon), prior to placing bedding materials, fill, steel, gravel, or concrete.

8.6 Existing Foundations

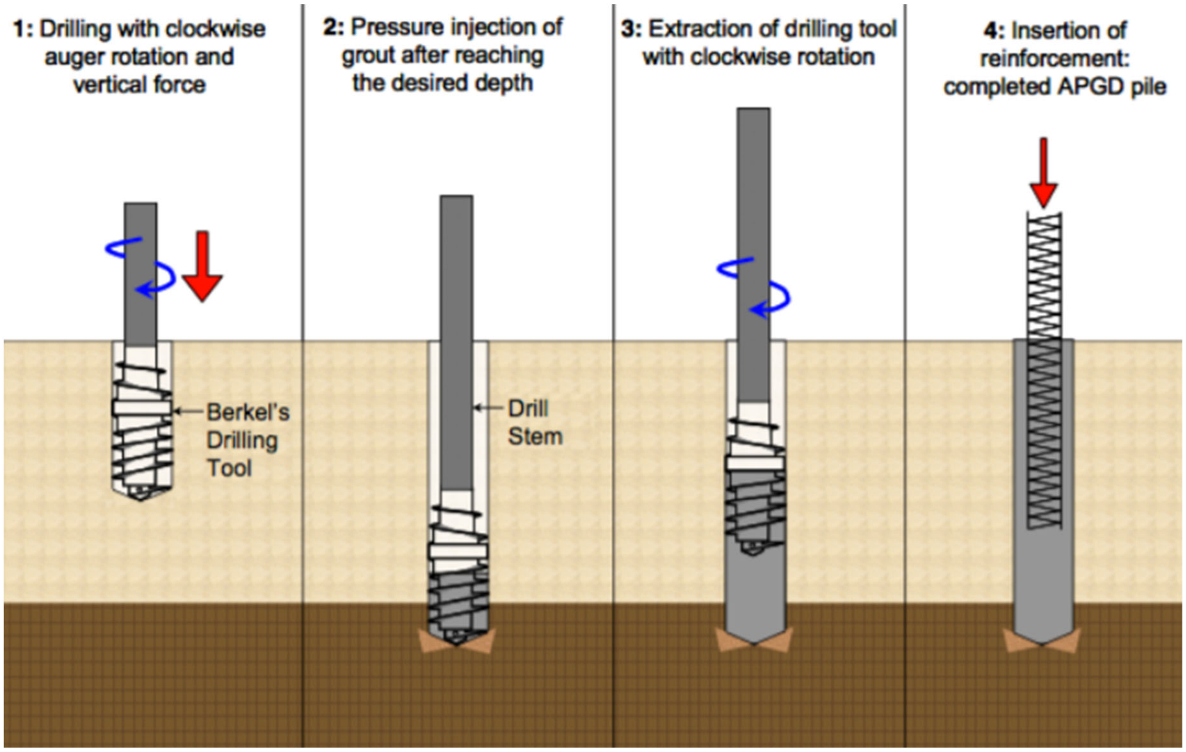
8.6.1 Plans depicting the existing foundation system were provided to us. Existing footing dimensions and embedment into alluvium were not verified by Geocon.

8.6.2 Based on the observed foundation notes and prior geotechnical reports prepared for the site, the existing foundations consist of 70 foot, 14-inch square piles designed with 200 kips axial compression capacity. Additional capacity may be feasible; however, increasing the capacity beyond the maximum allowable capacity may induce additional settlement of the existing foundations.

8.6.3 The project structural engineer should evaluate the existing foundations, existing building loads and proposed building loads. Where excess capacity remains, the existing foundations may be utilized for support of the proposed improvements. However, adding heavier loads to existing foundations could induce settlements on the existing foundations which could be detrimental to existing structural connections. Once existing and proposed load configurations become available, they should be provided to Geocon for additional settlement analyses. The structural engineer should evaluate the anticipated load configuration and resulting settlements, and determine the necessity for new foundations. Recommendations for new foundations are provided in the following section. The project structural engineer should verify the suitability and reinforcement design for all existing and new footings.

8.7 Auger-Cast Displacement Piles

8.7.1 Auger-cast pressure grouted displacement (APGD) piles are installed by advancing a hollow-stem auger with a diameter equivalent to that of the pile to the desired pile tip elevation. The specialized hollow-stem auger bit displaces the penetrated soils laterally away from the auger as it is advanced, creating increased pile capacity and minimizing the amount of soil spoils. Once the desired pile tip elevation is achieved, grout is pumped under pressure from the tip of the auger as it is withdrawn and then the pile reinforcing steel is placed in the grout.



8.7.2 The Client should be aware that APGD piles are typically designed and installed by a specialty geotechnical contractor. The recommendations presented herein for the design of APGD piles may be used for preliminary design purposes.

8.7.3 APGD piles should derive support in the competent alluvial soils found at or below a depth of 30 feet from the existing ground surface. For preliminary design purposes, 24-, 30-, and 36-inch diameter APGD piles have been assumed, and preliminary pile capacities are provided in the following table.

AUGER-CAST GROUTED DISPLACEMENT PILE CAPACITIES

Embedment below Ground Surface (feet)	24-Inch Diameter Pile Capacity (kips)	30-Inch Diameter Pile Capacity (kips)	36-Inch Diameter Pile Capacity (kips)
30 feet into competent bearing material	300	550	950

8.7.4 Single pile uplift capacity can be taken as 50 percent of the allowable downward capacity.

8.7.5 The axial capacity of the APGD piles should be verified by the design-build contractor and confirmed based upon pile load testing. Geocon should review, and if necessary, can assist the design-build contractor in developing a suitable testing program.

8.7.6 It is recommended that at least two pre-production piles or one percent of the production pile quantity be constructed, and load tested to at least 200 percent of the design load. In addition, Thermal Integrity Profiling will be required for all pre-production piles, and one pre-production pile should be exhumed for inspection as required by the building official.

8.7.7 During pile load testing, a representative of Geocon must be present to observe pile installation and testing procedures. The information obtained from the pile load testing program should be used to verify the suitability of the preliminary design parameters, or to modify pile design and installation criteria prior to construction of production piles.

8.7.8 Proof testing of production piles should also be performed by the design-build contractor and verified by the Geotechnical Engineer. It is recommended that at least 5 percent of production piles be constructed, and load tested to at least 160 percent of the design load. In addition, Thermal Integrity Profiling will be required for 10 percent of the production piles. The testing program and acceptance criteria should be configured to satisfy the requirements of the building official.

8.7.9 APGD pile construction should be performed under continuous observation of the Geotechnical Engineer (a representative of Geocon) to observe that soil conditions do not differ from those anticipated and to observe that construction of the APGD piles is performed in accordance with the project plans and specifications. Measurement of drilling torque and grout volume will be recorded to document the installation of the APGD piles, and grout samples will be collected to verify strength of materials.

8.7.10 If piles are spaced at least at least 3 diameters on center, no reduction in axial capacity is considered necessary for group effects. If pile spacing is closer than three pile diameters, an evaluation for group effects including appropriate reductions should be incorporated into the pile design based on pile dimension, spacing, and the direction of loading.

8.7.11 For increased resistance to differential foundation movement and lateral drift, the pile tops should be interconnected in two horizontal directions with grade beams or tied with a structural slab. The project structural engineer should provide slab and grade beam design, reinforcement and spacing dependent on anticipated loading. However, for grade beams we recommend a minimum embedment depth below lowest adjacent pad grade of 24 inches and a minimum width of 12 inches. In addition, minimum reinforcement should consist of four No. 5 steel reinforcing bars; two placed near the top of the grade beam and two near the bottom.

8.7.12 Pile settlement is expected to be less than 1 inch. The majority of settlement should occur on initial application of loading during construction. Differential settlement is not expected to exceed ¾ inch between adjacent piles or pile caps, and the mat foundation spanning across the piles should further reduce or possibly eliminate the effects of potential differential settlements in adjoining structural members.

8.8 Lateral Design

8.8.1 Resistance to lateral loading may be provided by friction acting at the base of foundations, slabs and by passive earth pressure. An allowable coefficient of friction of 0.30 may be used with the dead load forces in the undisturbed alluvial soils or properly compacted engineered fill.

8.8.2 The passive earth pressure may be computed as an equivalent fluid having a density of 110 pcf with a maximum earth pressure of 1,100 pcf (values have been reduced for buoyancy). When combining passive and friction for lateral resistance, the passive component should be reduced by one-third. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

8.8.3 Ultimate lateral capacities for ¼ inch deflection of fixed and free-head drilled cast-in place piles are presented in the table below. No factors of safety have been applied to the lateral load values calculated to induce ¼-inch lateral deflection. Lateral capacities provided are for 24-, 30-, and 36-inch diameter drilled cast-in-place concrete piles, penetrating the earth materials encountered during the course of this investigation. Assumed as part of these lateral capacity calculations are a concrete modulus of elasticity of at least 3,000,000 psi.

LATERAL LOAD CAPACITIES OF DRILLED CAST-IN-PLACE PILES

FIXED HEAD (NO HEAD ROTATION)

PILE NUMBER	PILE DIAMETER (INCHES)	Lateral Load Capacity "P" (KIPS)	Maximum Positive Moment "Mp" (LAT FORCE =P)	Maximum Negative Moment "Mp" (LAT FORCE =P)	Depth to Max Pos. Moment (Feet)	Depth to Zero Moment (Feet)	Depth to Inflection Point (Feet)	MINIMUM PILE LENGTH FOR APPLICABILITY OF LATERAL DESIGN DATA (FEET)
1	24	**43**	**1.4** P	**-5.1** P	**12**	**25**	**6.4**	**25**
2	30	**61**	**1.7** P	**-6.1** P	**15**	**30**	**7.6**	**30**
3	36	**81**	**1.9** P	**-7.1** P	**17**	**35**	**8.8**	**35**

FREE HEAD (HINGED)

PILE NUMBER	PILE DIAMETER (INCHES)	Lateral Load Capacity "P" (KIPS)	Maximum Moment "Mp" (LAT FORCE =P)	Depth to Zero Moment (Feet)	Depth to Maximum Moment (Feet)
1	24	**17**	**4.3** P	**23**	**7**
2	30	**25**	**5.2** P	**27**	**9**
3	36	**33**	**6.0** P	**31**	**10**

Lateral capacities are based on 1/4-inch deflection.
Moment magnitudes are presented as a function of the applied lateral load “P”.
"P" is entered in units of kips and the moment magnitude will be in units of kip-feet.
The maximum negative moment is at the rigid, pile to pile cap or grade beam connection at the top of the pile.

8.9 Exterior Concrete Slabs-on-Grade

8.9.1 Exterior slabs, not subject to traffic loads, should be at least 4 inches thick and reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions, positioned near the slab midpoint. Prior to construction of slabs, the upper 12 inches of subgrade should be moistened to approximately optimum moisture content and properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition). Crack control joints should be spaced at intervals not greater than 10 feet and should be constructed using saw-cuts or other methods as soon as practical following concrete placement. Crack control joints should extend a minimum depth of 1/4 the slab thickness. The project structural engineer should design construction joints as necessary.

8.9.2 The moisture content of the slab subgrade should be maintained and sprinkled as necessary to maintain a moist condition as would be expected in any concrete placement.

8.9.3 The recommendations of this report are intended to reduce the potential for cracking of slabs due to settlement. However, even with the incorporation of the recommendations presented herein, foundations, stucco walls, and slabs-on-grade may exhibit some cracking due to minor soil movement and/or concrete shrinkage. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, proper concrete placement and curing, and by the placement of crack control joints at periodic intervals, in particular, where re-entrant slab corners occur.

8.10 Retaining Wall Design

8.10.1 The recommendations presented below are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 30 feet. In the event that walls significantly higher than 30 feet are planned, Geocon should be contacted for additional recommendations.

8.10.2 Retaining walls with a level backfill surface that are not restrained at the top should be designed utilizing a triangular distribution of pressure (active pressure). Restrained walls are those that are not allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall in feet) at the top of the wall. Where walls are restrained from movement at the top, walls may be designed utilizing a triangular distribution of pressure (at-rest pressure). The table below presents recommended pressures to be used in retaining wall design, assuming that proper drainage will be maintained. Calculation of the recommended retaining wall pressures are provided as Figure 5.

RETAINING WALL WITH LEVEL BACKFILL SURFACE

HEIGHT OF RETAINING WALL (Feet)	ACTIVE PRESSURE EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot)	AT-REST PRESSURE EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot)
Up to 30	37	57

8.10.3 The wall pressures provided above assume that the proposed retaining walls will support relatively undisturbed alluvial soils. If sloping techniques are to be utilized for construction of proposed walls, which would result in a wedge of engineered fill behind the retaining walls, revised earth pressures may be required. This should be evaluated once the use of sloping measures is established and once the geotechnical characteristics of the engineered backfill soils can be further evaluated.

8.10.4 The wall pressures provided above assume that the retaining wall will be properly drained preventing the buildup of hydrostatic pressure. If retaining wall drainage is not implemented, the equivalent fluid pressure to be used in design of undrained walls is 90 pcf. The value includes hydrostatic pressures plus buoyant lateral earth pressures.

8.10.5 Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures and should be designed for each condition as the project progresses. Surcharges may be evaluated using Section 8.21 of this report. Once the design becomes more finalized, an addendum letter can be prepared revising recommendations and addressing specific surcharge conditions throughout the project, if necessary.

8.10.6 In addition to the recommended earth pressure, the upper 10 feet of the wall adjacent to the street or driveway areas should be designed to resist a uniform lateral pressure of 100 psf, acting as a result of an assumed 300 psf surcharge behind the wall due to normal street traffic. If the traffic is kept back at least 10 feet from the wall, the traffic surcharge may be neglected.

8.10.7 Seismic lateral forces should be incorporated into the design as necessary, and recommendations for seismic lateral forces are presented below.

8.11 Dynamic (Seismic) Lateral Forces

8.11.1 The structural engineer should determine the seismic design category for the project in accordance with Section 1613 of the CBC. If the project possesses a seismic design category of D, E, or F, proposed retaining walls in excess of 6 feet in height should be designed with seismic lateral pressure (Section 1803.5.12 of the 2019 CBC).

8.11.2 A seismic load of 10 pcf should be used for design of walls that support more than 6 feet of backfill in accordance with Section 1803.5.12 of the 2019 CBC. The seismic load is applied as an equivalent fluid pressure along the height of the wall and the calculated loads result in a maximum load exerted at the base of the wall and zero at the top of the wall. This seismic load should be applied in addition to the active earth pressure. The earth pressure is based on half of two-thirds of PGA_M calculated from ASCE 7-10 Section 11.8.3.

8.12 Retaining Wall Drainage

8.12.1 Retaining walls should be provided with a drainage system. At the base of the drain system, a subdrain covered with a minimum of 12 inches of gravel should be installed, and a compacted fill blanket or other seal placed at the surface (see Figure 6). The clean bottom and subdrain pipe, behind a retaining wall, should be observed by the Geotechnical Engineer (a representative of Geocon), prior to placement of gravel or compacting backfill.

8.12.2 As an alternative, a plastic drainage composite such as Miradrain or equivalent may be installed in continuous, 4-foot wide columns along the entire back face of the wall, at 8 feet on center. The top of these drainage composite columns should terminate approximately 18 inches below the ground surface, where either hardscape or a minimum of 18 inches of relatively cohesive material should be placed as a cap (see Figure 7). These vertical columns of drainage material would then be connected at the bottom of the wall to a collection panel or a 1-cubic-foot rock pocket drained by a 4-inch subdrain pipe.

8.12.3 Subdrainage pipes at the base of the retaining wall drainage system should outlet to an acceptable location via controlled drainage structures. Drainage should not be allowed to flow uncontrolled over descending slopes.

8.12.4 Moisture affecting below grade walls is one of the most common post-construction complaints. Poorly applied or omitted waterproofing can lead to efflorescence or standing water. Particular care should be taken in the design and installation of waterproofing to avoid moisture problems, or actual water seepage into the structure through any normal shrinkage cracks which may develop in the concrete walls, floor slab, foundations and/or construction joints. The design and inspection of the waterproofing is not the responsibility of the geotechnical engineer. A waterproofing consultant should be retained in order to recommend a product or method, which would provide protection to subterranean walls, floor slabs and foundations.

8.13 Elevator Pit Design

8.13.1 The elevator pit slab and retaining wall should be designed by the project structural engineer. Elevator pit walls may be designed in accordance with the recommendations in the *Foundation Design* and *Retaining Wall Design* sections of this report (see Sections 8.7 through 8.10).

8.13.2 Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent foundations and should be designed for each condition as the project progresses.

8.13.3 If retaining wall drainage is to be provided, the drainage system should be designed in accordance with the *Retaining Wall Drainage* section of this report (see Section 8.12).

8.13.4 It is suggested that the exterior walls and slab be waterproofed to prevent excessive moisture inside of the elevator pit. Waterproofing design and installation is not the responsibility of the geotechnical engineer.

8.14 Elevator Piston

8.14.1 If a plunger-type elevator piston is installed for this project, a deep drilled excavation will be required. It is important to verify that the drilled excavation is not situated immediately adjacent to a foundation or shoring pile, or the drilled excavation could compromise the existing foundation or pile support, especially if the drilling is performed subsequent to the foundation or pile construction.

8.14.2 Casing will be required since caving is expected in the drilled excavation and the contractor should be prepared to use casing and should have it readily available at the commencement of drilling activities. The contractor should also be prepared to mitigate buoyant forces during installation of the piston casing. Continuous observation of the drilling and installation of the elevator piston by the Geotechnical Engineer (a representative of Geocon West, Inc.) is required.

8.14.3 The annular space between the piston casing and drilled excavation wall should be filled with a minimum of 1½-sack slurry pumped from the bottom up. As an alternative, pea gravel may be utilized. The use of soil to backfill the annular space is not acceptable.

8.15 Temporary Excavations

8.15.1 Excavations up to 30 feet in height may be required for the excavation and construction of the proposed towers. The excavations are expected to expose artificial fill and alluvial soils, which may be subject to caving where granular soils are exposed. Vertical excavations up to 5 feet in height may be attempted where not surcharged by adjacent traffic or structures.

8.15.2 Vertical excavations greater than 5 feet will require sloping and/or shoring measures in order to provide a stable excavation. Where sufficient space is available, temporary unsurcharged embankments up to 12 feet in height can be sloped back at a uniform 1:1 slope gradient or flatter. A uniform slope does not have a vertical portion. Where space is limited, shoring measures will be required. *Shoring* recommendations are provided in the following section.

8.15.3 Where temporary construction slopes are utilized, the top of the slope should be barricaded to prevent vehicles and storage loads at the top of the slope within a horizontal distance equal to the height of the slope. If the temporary slopes are to be maintained during the rainy season, berms are suggested along the tops of the slopes where necessary to prevent runoff water from entering the excavation and eroding the slope faces. Geocon personnel should inspect the soils exposed in the cut slopes during excavation so that modifications of the slopes can be made if variations in the soil conditions occur. All excavations should be stabilized within 30 days of initial excavation.

8.16 Shoring – Soldier Pile Design and Installation & Underpinning

8.16.1 The following information on the design and installation of shoring is preliminary. Review of the final shoring plans and specifications should be made by this office prior to bidding or negotiating with a shoring contractor.

8.16.2 One method of shoring would consist of steel soldier piles, placed in drilled holes and backfilled with concrete. Where maximum excavation heights are less than 12 feet the soldier piles are typically designed as cantilevers. Where excavations exceed 12 feet or are surcharged, soldier piles may require lateral bracing utilizing drilled tie-back anchors or raker braces to maintain an economical steel beam size and prevent excessive deflection. The size of the steel beam, the need for lateral bracing, and the acceptable shoring deflection should be determined by the project shoring engineer.

8.16.3 The design embedment of the shoring pile toes must be maintained during excavation activities. The toes of the perimeter shoring piles should be deepened to take into account any required excavations necessary for grading, foundation excavations, and/or adjacent drainage systems.

8.16.4 All piles utilized for shoring can also be incorporated into a permanent retaining wall system (shotcrete wall) and should be designed in accordance with the earth pressure provided in the *Retaining Walls* section of this report (see Section 8.10).

8.16.5 Drilled cast-in-place soldier piles should be placed no closer than 3 diameters on center. The minimum diameter of the piles is 18 inches. Structural concrete should be used for the soldier piles below the excavation; lean-mix concrete may be employed above that level. As an alternative, lean-mix concrete may be used throughout the pile where the reinforcing consists of a wideflange section. The slurry must be of sufficient strength to impart the lateral bearing pressure developed by the wideflange section to the soil. For design purposes, an allowable passive value for the soils below the bottom plane of excavation may be assumed to be 110 psf per foot (value has been reduced for buoyant forces). Where piles are installed by vibration techniques, the passive pressure may be assumed to mobilize across a width equal to the two times the dimension of the beam flange. The allowable passive value may be doubled for isolated piles, spaced a minimum of three times the pile diameter. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed alluvium.

8.16.6 Groundwater was encountered during exploration at a depth of approximately 31 feet below the ground surface; therefore, the contractor should be prepared for groundwater during pile installation. Piles placed below the water level require the use of a tremie to place the concrete into the bottom of the hole. A tremie should consist of a rigid, water-tight tube having a diameter of not less than 6 inches with a hopper at the top. The tube should be equipped with a device that will close the discharge end and prevent water from entering the tube while it is being charged with concrete. The tremie should be supported so as to permit free movement of the discharge end over the entire top surface of the work and to permit rapid lowering when necessary to retard or stop the flow of concrete. The discharge end should be closed at the start of the work to prevent water entering the tube and should be entirely sealed at all times, except when the concrete is being placed. The tremie tube should be kept full of concrete. The flow should be continuous until the work is completed, and the resulting concrete seal should be monolithic and homogeneous. The tip of the tremie tube should always be kept about 5 feet below the surface of the concrete and definite steps and safeguards should be taken to ensure that the tip of the tremie tube is never raised above the surface of the concrete.

8.16.7 A special concrete mix should be used for concrete to be placed below water. The design should provide for concrete with an unconfined compressive strength psi of 1,000 psi over the initial job specification. An admixture that reduces the problem of segregation of paste/aggregates and dilution of paste should be included. The slump should be commensurate to any research report for the admixture, provided that it should also be the minimum for a reasonable consistency for placing when water is present.

8.16.8 Casing may be required if caving may occur in the granular soils, and the contractor should have casing available prior to commencement of pile excavation. If casing is used, extreme care should be employed so that the pile is not pulled apart as the casing is withdrawn. At no time should the distance between the surface of the concrete and the bottom of the casing be less than five feet. Continuous observation of the drilling and pouring of the piles by the Geotechnical Engineer (a representative of Geocon West, Inc.), is required.

8.16.9 The frictional resistance between the soldier piles and retained soil may be used to resist the vertical component of the anchor load. The coefficient of friction may be taken as 0.3 based on uniform contact between the steel beam and lean-mix concrete and retained earth. The portion of soldier piles below the plane of excavation may also be employed to resist the downward loads. The downward capacity may be determined using a frictional resistance of 700 psf (value has been reduced for buoyant forces).

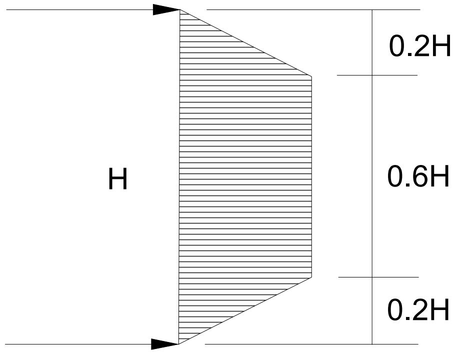
8.16.10 Due to the nature of the site soils, it is expected that continuous lagging between soldier piles will be required. However, it is recommended that the exposed soils be observed by the Geotechnical Engineer (a representative of Geocon West, Inc.), to verify the presence of any competent, cohesive soils and the areas where lagging may be omitted.

8.16.11 The time between lagging excavation and lagging placement should be as short as possible soldier piles should be designed for the full-anticipated pressures. Due to arching in the soils, the pressure on the lagging will be less. It is recommended that the lagging be designed for the full design pressure but be limited to a maximum of 400 psf.

8.16.12 For the design of unbraced shoring, it is recommended that an equivalent fluid pressure be utilized for design. A trapezoidal distribution of lateral earth pressure may be used where shoring will be restrained by bracing or tiebacks. The recommended active and trapezoidal pressure are provided in the following table. A diagram depicting the trapezoidal pressure distribution of lateral earth pressure is provided below the table. Calculation of the recommended shoring wall pressure as provided as Figure 8.

HEIGHT OF SHORING (FEET)	**EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (ACTIVE PRESSURE)**	**EQUIVALENT FLUID PRESSURE (Pounds Per Square Foot per Foot) Trapezoidal (Where H is the height of the shoring in feet)**
Up to 32	29	18H

Trapezoidal Distribution of Pressure



8.16.13 Where a combination of sloped embankment and shoring is utilized, the pressure will be greater and must be determined for each combination. Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic, or adjacent structures and must be determined for each combination.

8.16.14 Where a combination of sloped embankment and shoring is utilized, the pressure will be greater and must be determined for each combination. Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic, or adjacent structures and must be determined for each combination. The surcharge pressure should be evaluated in accordance with the recommendations in Section 8.21 of this report.

8.16.15 In addition to the recommended earth pressure, the upper ten feet of the shoring adjacent to the street or driveway areas should be designed to resist a uniform lateral pressure of 100 psf, acting as a result of an assumed 300 psf surcharge behind the shoring due to normal street traffic. If the traffic is kept back at least ten feet from the shoring, the traffic surcharge may be neglected.

8.16.16 It is difficult to accurately predict the amount of deflection of a shored embankment. It should be realized that some deflection will occur. It is recommended that the deflection be minimized to prevent damage to existing structures and adjacent improvements. Where public right-of-ways are present or adjacent offsite structures do not surcharge the shoring excavation, the shoring deflection should be limited to less than 1 inch at the top of the shored embankment. Where offsite structures are within the shoring surcharge area it is recommended that the beam deflection be limited to less than ½ inch at the elevation of the adjacent offsite foundation, and no deflection at all if deflections will damage existing structures. The allowable deflection is dependent on many factors, such as the presence of structures and utilities near the top of the embankment, and will be assessed and designed by the project shoring engineer.

8.16.17 Because of the depth of the excavation, some means of monitoring the performance of the shoring system is suggested. The monitoring should consist of periodic surveying of the lateral and vertical locations of the tops of all soldier piles and the lateral movement along the entire lengths of selected soldier piles.

8.16.18 Due to the depth of the excavation and proximity to adjacent structures, it is suggested that prior to excavation the existing improvements be inspected to document the present condition. For documentation purposes, photographs should be taken of preconstruction distress conditions and level surveys of adjacent grade and pavement should be considered. During excavation activities, the adjacent structures and pavement should be periodically inspected for signs of distress. In the event that distress or settlement is noted, an investigation should be performed and corrective measures taken so that continued or worsened distress or settlement is mitigated. Documentation and monitoring of the offsite structures and improvements is not the responsibility of the geotechnical engineer.

8.17 Temporary Tie-Back Anchors

8.17.1 Temporary tie-back anchors may be used with the solider pile wall system to resist lateral loads. Post-grouted friction anchors are recommended. For design purposes, it may be assumed that the active wedge adjacent to the shoring is defined by a plane drawn 35 degrees with the vertical through the bottom plane of the excavation. Friction anchors should extend a minimum of 20 feet beyond the potentially active wedge and to greater lengths if necessary to develop the desired capacities. The locations and depths of all offsite utilities should be thoroughly checked and incorporated into the drilling angle design for the tie-back anchors.

8.17.2 The locations and depths of all offsite utilities should be thoroughly checked and incorporated into the drilling angle design for the tie-back anchors.

8.17.3 The capacities of the anchors should be determined by testing of the initial anchors as outlined in a following section. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads. Anchors should be placed at least 6 feet on center to be considered isolated. Based on the height of the proposed excavation, two rows of anchors may be required. For preliminary design purposes, it is estimated that drilled friction anchors constructed without utilizing post-grouting techniques will develop average skin frictions (*reduced for buoyancy) as follows:

- 7 feet below the top of the excavation – 1,100 pounds per square foot
- 15 feet below the top of the excavation – 900 pounds per square foot*
- 25 feet below the top of the excavation – 1,100 pounds per square foot*

  *capacity reduced for buoyant forces

8.17.4 Depending on the techniques utilized, and the experience of the contractor performing the installation, a maximum allowable friction capacity of 3.7 kips per linear foot for post-grouted anchors (for a minimum 20-foot length beyond the active wedge) may be assumed for design purposes. Only the frictional resistance developed beyond the active wedge should be utilized in resisting lateral loads.

8.18 Anchor Installation

8.18.1 Tied-back anchors are typically installed between 20 and 40 degrees below the horizontal; however, occasionally alternative angles are necessary to avoid existing improvements and utilities. The locations and depths of all offsite utilities should be thoroughly checked prior to design and installation of the tie-back anchors. Caving of the anchor shafts, particularly within sand and gravel deposits or seepage zones, should be anticipated during installation and provisions should be implemented in order to minimize such caving. It is suggested that hollow-stem auger drilling equipment be used to install the anchors. The anchor shafts should be filled with concrete by pumping from the tip out, and the concrete should extend from the tip of the anchor to the active wedge. In order to minimize the chances of caving, it is recommended 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 flush with the face of the excavation. The sand backfill should be placed by pumping; the sand may contain a small amount of cement to facilitate pumping.

8.19 Anchor Testing

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

8.19.2 At least ten percent of the anchors should be selected for "quick" 200 percent tests and two additional anchors should be selected for 24-hour 200 percent tests. The purpose of the 200 percent tests is to verify the friction value assumed in design. The anchors should be tested to develop twice the assumed friction value. These tests should be performed prior to installation of additional tiebacks. 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.

8.19.3 The total deflection during the 24-hour 200 percent test should not exceed 12 inches. During the 24-hour tests, the anchor deflection should not exceed 0.75 inches measured after the 200 percent test load is applied.

8.19.4 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 200 percent quick tests should not exceed 12 inches; the deflection after the 200 percent load has been applied should not exceed 0.25 inch during the 30-minute period.

8.19.5 After a satisfactory test, each anchor should be locked-off at the design load. This should be verified by rechecking the load in the anchor. The load should be within 10 percent of the design load. The installation and testing of the anchors should be observed by a representative of this firm.

8.20 Internal Bracing

8.20.1 Rakers may be utilized to brace the soldier piles in lieu of tieback anchors. The raker bracing could be supported laterally by temporary concrete footings (deadmen) or by the permanent, interior footings. For design of such temporary footings or deadmen embedding into bedrock, poured with the bearing surface normal to rakers inclined at 45 degrees, a bearing value of 2,000 psf may be used, provided the shallowest point of the footing is at least one foot below the lowest adjacent grade.

8.20.2 The client should be aware that the utilization of rakers could significantly impact the construction schedule do to their intrusion into the construction site and potential interference with equipment. In addition, the raker footing plan should be checked by the project structural engineer to verify if there are any conflicts with the proposed structural foundations, and resolve any issues prior to commencement of construction activities.

8.21 Surcharge from Adjacent Structures and Improvements

8.21.1 Additional pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures and should be designed for each condition as the project progresses.

8.21.2 It is recommended that line-load surcharges from adjacent wall footings, use horizontal pressures generated from NAV-FAC DM 7.2. The governing equations are:

$$For\ {}^{x}/_{H} \leq 0.4$$

$$\sigma_H(z) = \frac{0.20 \times \left(\frac{z}{H}\right)}{\left[0.16 + \left(\frac{z}{H}\right)^2\right]^2} \times \frac{Q_L}{H}$$

and

$$For\ {}^{x}/_{H} > 0.4$$

$$\sigma_H(z) = \frac{1.28 \times \left(\frac{x}{H}\right)^2 \times \left(\frac{z}{H}\right)}{\left[\left(\frac{x}{H}\right)^2 + \left(\frac{z}{H}\right)^2\right]^2} \times \frac{Q_L}{H}$$

where x is the distance from the face of the excavation or wall to the vertical line-load, H is the distance from the bottom of the footing to the bottom of excavation or wall, z is the depth at which the horizontal pressure is desired, Q_L is the vertical line-load and $\sigma_H(z)$ is the horizontal pressure at depth z.

8.21.3 It is recommended that vertical point-loads, from construction equipment outriggers or adjacent building columns use horizontal pressures generated from NAV-FAC DM 7.2. The governing equations are:

$$For\ {}^{x}\!/_{H} \leq 0.4$$

$$\sigma_H(z) = \frac{0.28 \times \left(\frac{z}{H}\right)^2}{\left[0.16 + \left(\frac{z}{H}\right)^2\right]^3} \times \frac{Q_P}{H^2}$$

and

$$For\ {}^{x}\!/_{H} > 0.4$$

$$\sigma_H(z) = \frac{1.77 \times \left(\frac{x}{H}\right)^2 \times \left(\frac{z}{H}\right)^2}{\left[\left(\frac{x}{H}\right)^2 + \left(\frac{z}{H}\right)^2\right]^3} \times \frac{Q_P}{H^2}$$

then

$$\sigma'_H\,(z) = \sigma_H(z)cos^2(1.1\theta)$$

where x is the distance from the face of the excavation/wall to the vertical point-load, H is distance from the outrigger/bottom of column footing to the bottom of excavation, z is the depth at which the horizontal pressure is desired, Q_P is the vertical point-load, $\sigma_H(z)$ is the horizontal pressure at depth z, θ is the angle between a line perpendicular to the excavation/wall and a line from the point-load to location on the excavation/wall where the surcharge is being evaluated, and $\sigma_H(z)$ is the horizontal pressure at depth z.

8.21.4 In addition to the recommended earth pressure, the upper 10 feet of the shoring adjacent to the street or driveway areas should be designed to resist a uniform lateral pressure of 100 psf, acting as a result of an assumed 300 psf surcharge behind the shoring due to normal street traffic. If the traffic is kept back at least 10 feet from the shoring, the traffic surcharge may be neglected.

8.22 Surface Drainage

8.22.1 Proper surface drainage is critical to the future performance of the project. Uncontrolled infiltration of irrigation excess and storm runoff into the soils can adversely affect the performance of the planned improvements. Saturation of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change in the original designed engineering properties. Proper drainage should be maintained at all times.

8.22.2 All site drainage should be collected and controlled in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2019 CBC 1804.4 or other applicable standards. In addition, drainage should not be allowed to flow uncontrolled over any descending slope. Discharge from downspouts, roof drains and scuppers are not recommended onto unprotected soils within 5 feet of the building perimeter. Planters which are located adjacent to foundations should be sealed to prevent moisture intrusion into the soils providing foundation support. Landscape irrigation is not recommended within 5 feet of the building perimeter footings except when enclosed in protected planters.

8.22.3 Positive site drainage should be provided away from structures, pavement, and the tops of slopes to swales or other controlled drainage structures.

8.22.4 Landscaping planters immediately adjacent to paved areas are not recommended due to the potential for surface or irrigation water to infiltrate the pavement's subgrade and base course. Either a subdrain, which collects excess irrigation water and transmits it to drainage structures, or impervious above-grade planter boxes should be used. In addition, where landscaping is planned adjacent to the pavement, it is recommended that consideration be given to providing a cutoff wall along the edge of the pavement that extends at least 12 inches below the base material.

8.23 Plan Review

8.23.1 Grading, foundation, and shoring plans should be reviewed by the Geotechnical Engineer (a representative of Geocon West, Inc.), prior to finalization to verify that the plans have been prepared in substantial conformance with the recommendations of this report and to provide additional analyses or recommendations.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. The recommendations of this report pertain only to the site investigated and are based upon the assumption that the soil conditions do not deviate from those disclosed in the investigation. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that anticipated herein, Geocon West, Inc. should be notified so that supplemental recommendations can be given. The evaluation or identification of the potential presence of hazardous or corrosive materials was not part of the scope of services provided by Geocon West, Inc.

2. This report is issued with the understanding that it is the responsibility of the owner, or of his representative, to ensure that the information and recommendations contained herein are brought to the attention of the architect and engineer for the project and incorporated into the plans, and the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

3. The findings of this report are valid as of the date of this report. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of three years.

4. The firm that performed the geotechnical investigation for the project should be retained to provide testing and observation services during construction to provide continuity of geotechnical interpretation and to check that the recommendations presented for geotechnical aspects of site development are incorporated during site grading, construction of improvements, and excavation of foundations. If another geotechnical firm is selected to perform the testing and observation services during construction operations, that firm should prepare a letter indicating their intent to assume the responsibilities of project geotechnical engineer of record. A copy of the letter should be provided to the regulatory agency for their records. In addition, that firm should provide revised recommendations concerning the geotechnical aspects of the proposed development, or a written acknowledgement of their concurrence with the recommendations presented in our report. They should also perform additional analyses deemed necessary to assume the role of Geotechnical Engineer of Record.

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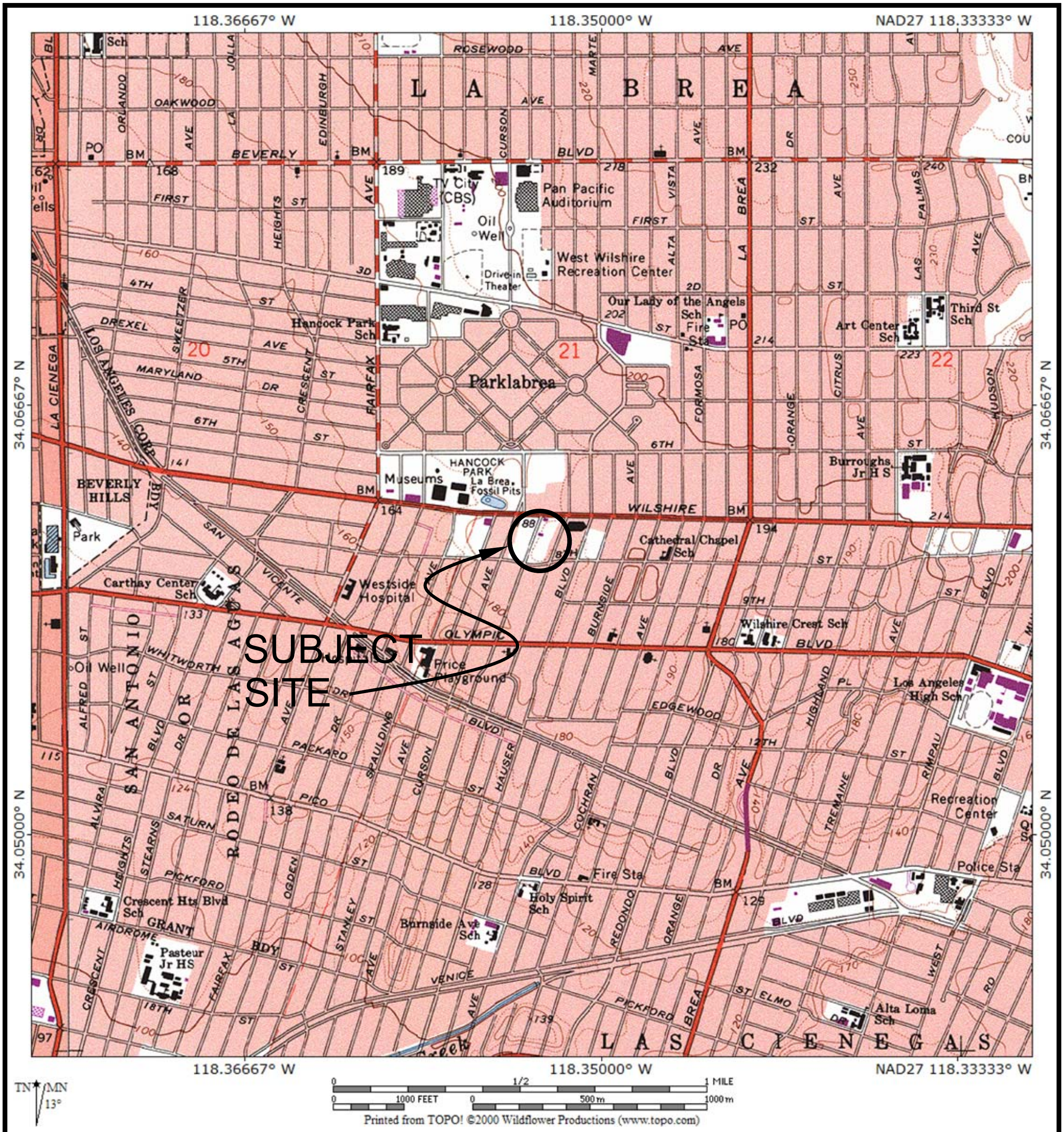

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REFERENCE:U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, HOLLYWOOD, CA QUADRANGLE

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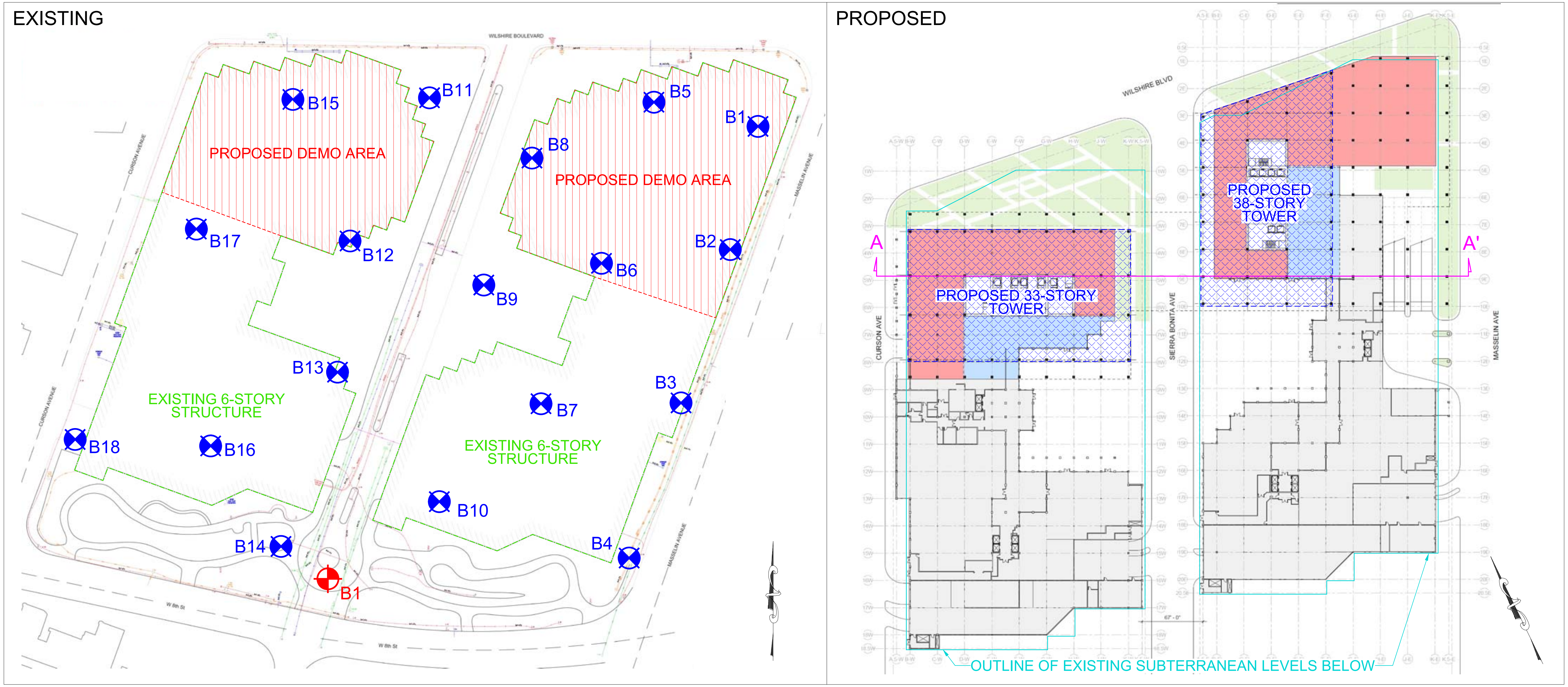
DRAFTED BY: MDS | CHECKED BY: SFK

VICINITY MAP

5700 & 5750 W. WILSHIRE BLVD.
LOS ANGELES, CALIFORNIA

MARCH 2020 | PROJECT NO. W1038-06-01A | FIG. 1



LEGEND

⊕ B3	Approximate Location of Boring (Geocon, 2020)
⊗ B18	Approximate Location of Boring (LeRoy Crandall, 1984)

0 120' 240'

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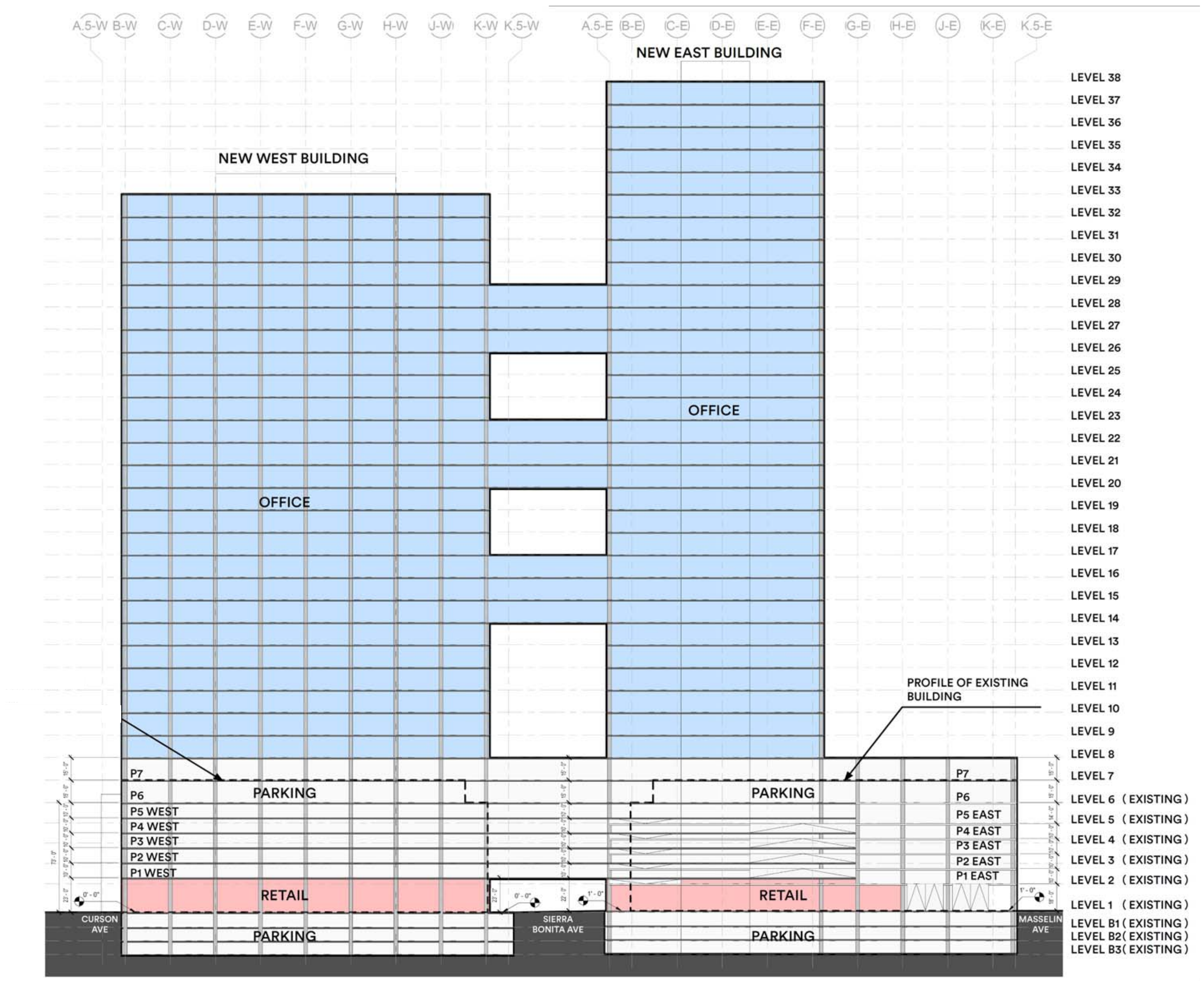
SITE PLAN

5700 & 5750 W. WILSHIRE BLVD.
LOS ANGELES, CALIFORNIA

MARCH 2020 PROJECT NO. W1038-06-01A FIG. 2A



SECTION A-A'

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CROSS SECTION

5700 & 5750 W. WILSHIRE BLVD.
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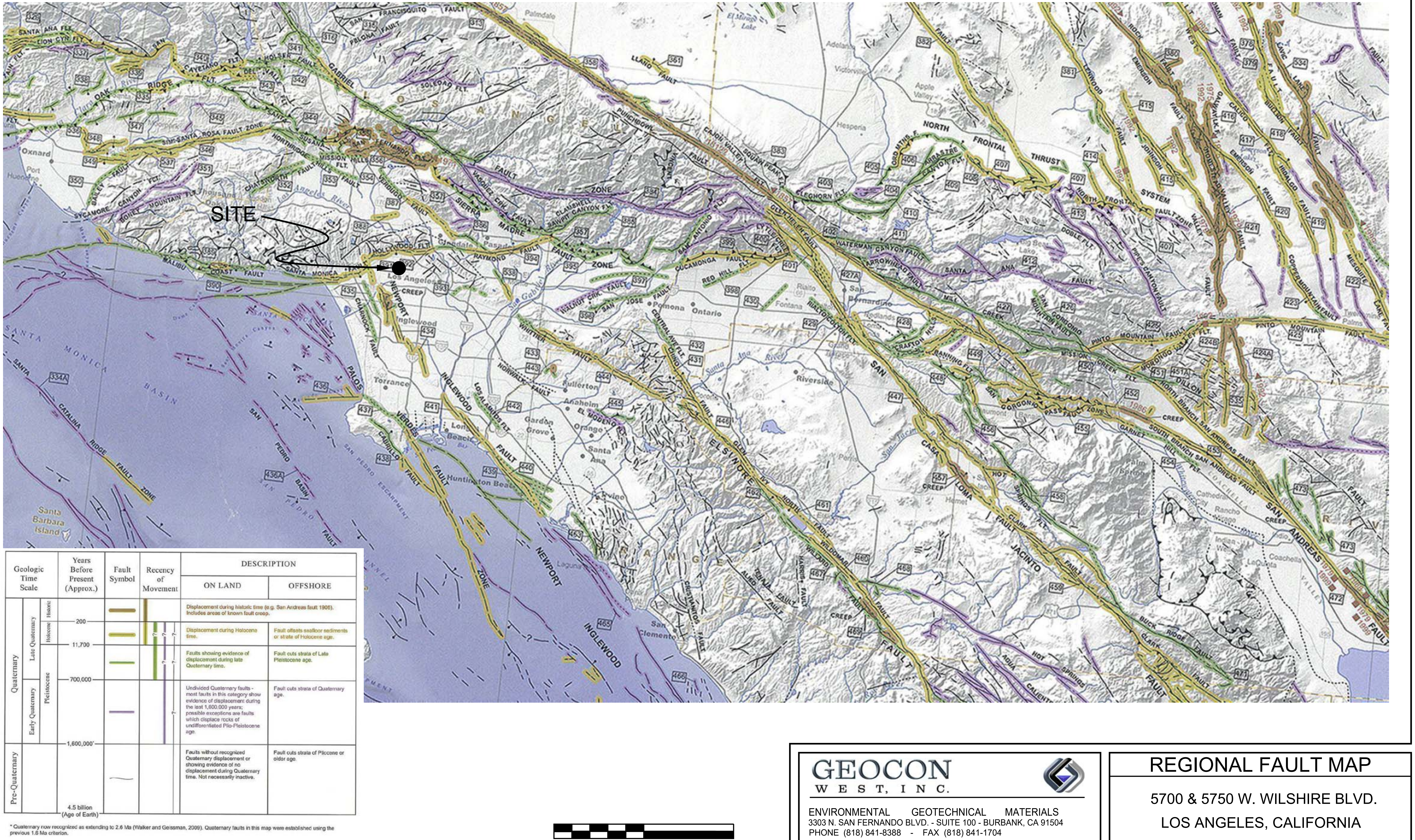
MARCH 2020	PROJECT NO. W1038-06-01A	FIG. 2B

Reference: Jennings, C.W. and Bryant, W. A., 2010, Fault Activity Map of California, California Geological Survey Geologic Data Map No. 6.



Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION ON LAND	DESCRIPTION OFFSHORE
Quaternary	Late Quaternary	Historic		▬		Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
		Holocene	200	▬	? ? ?	Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
		Pleistocene	11,700	▬	? ?	Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
	Early Quaternary		700,000	▬	?	Undivided Quaternary faults - most faults in this category show evidence of displacement during the last 1,600,000 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age.	Fault cuts strata of Quaternary age.
Pre-Quaternary			1,600,000* 4.5 billion (Age of Earth)	~		Faults without recognized Quaternary displacement or showing evidence of no displacement during Quaternary time. Not necessarily inactive.	Fault cuts strata of Pliocene or older age.

* Quaternary now recognized as extending to 2.6 Ma (Walker and Geissman, 2009). Quaternary faults in this map were established using the previous 1.6 Ma criterion.

0 12 24 Miles

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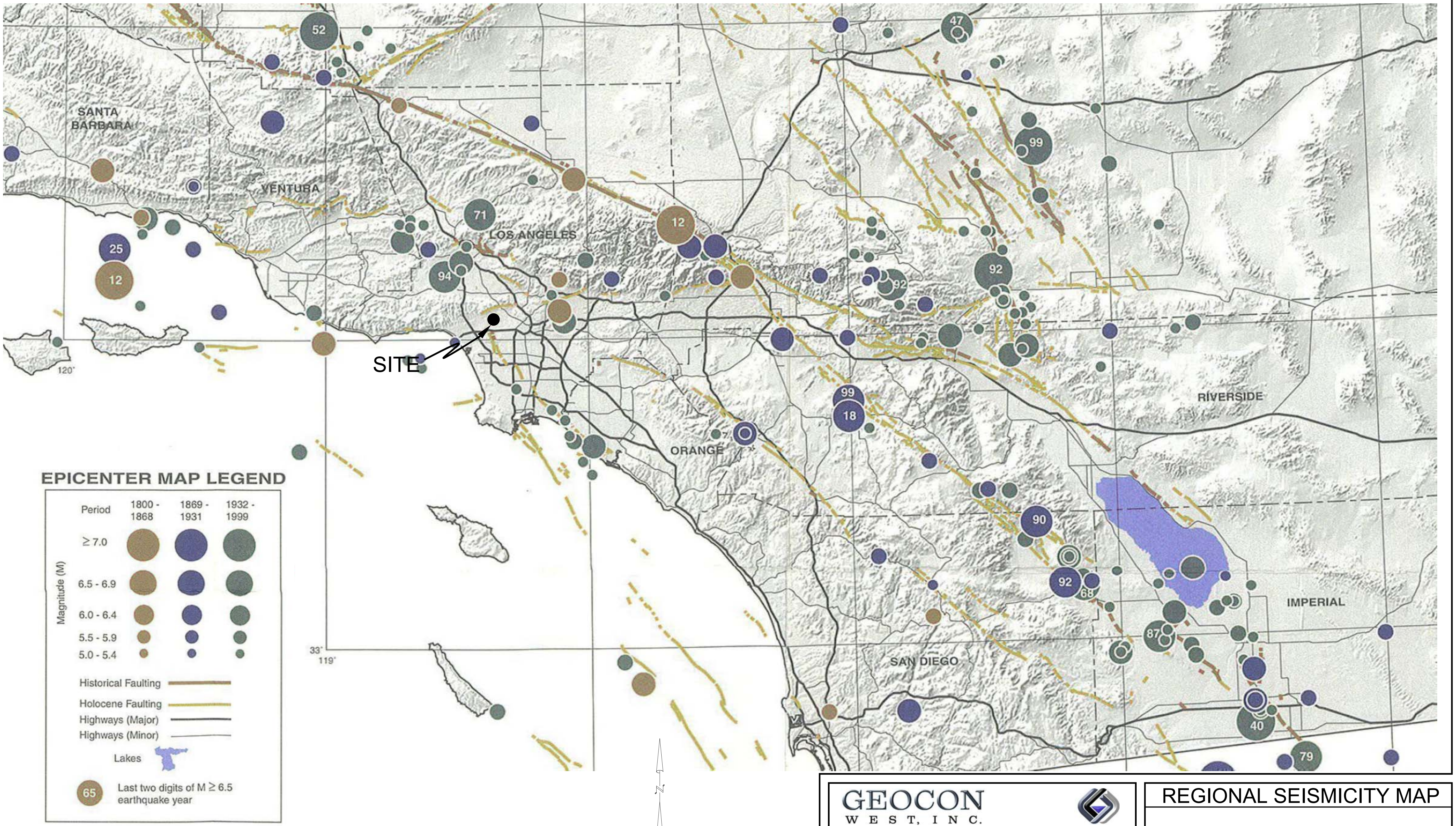
REGIONAL FAULT MAP

5700 & 5750 W. WILSHIRE BLVD.
LOS ANGELES, CALIFORNIA

MARCH 2020	PROJECT NO. W1038-06-01A	FIG. 3



Reference: Toppozada, T., Branum, D., Petersen, M., Hallstrom, C., Cramer, C., and Reichle, M., 2000, Epicenters and Areas Damaged by M≥5 California Earthquakes, 1800 - 1999, California Geological Survey, Map Sheet 49.

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REGIONAL SEISMICITY MAP

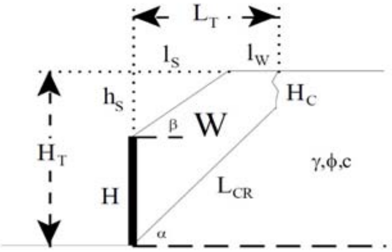
5700 & 5750 W. WILSHIRE BLVD.
LOS ANGLES, CALIFORNIA

MARCH 2020	PROJECT NO. W1038-06-01A	FIG. 4

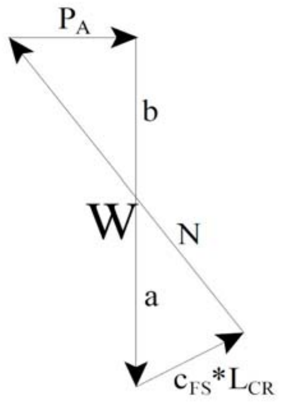
Retaining Wall Design with Transitioned Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	30.00	feet
Slope Angle of Backfill	(b)	0.0	degrees
Height of Slope above Wall	(h_s)	0.0	feet
Horizontal Length of Slope	(l_s)	0.0	feet
Total Height (Wall + Slope)	(H_T)	30.0	feet
Unit Weight of Retained Soils	(g)	125.0	pcf
Friction Angle of Retained Soils	(f)	33.0	degrees
Cohesion of Retained Soils	(c)	320.0	psf
Factor of Safety	(FS)	1.50	
Factored Parameters:	(f_{FS})	23.4	degrees
	(c_{FS})	213.3	psf



Failure Angle (a) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) feet^2	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
45	6.0	432	53985.5	33.9	18043.5	35942.0	14223.5
46	5.9	418	52240.9	33.5	17096.2	35144.6	14622.4
47	5.7	404	50534.8	33.2	16228.4	34306.4	14981.2
48	5.6	391	48867.3	32.8	15431.3	33436.0	15301.4
49	5.5	378	47237.8	32.4	14697.6	32540.2	15583.9
50	5.4	365	45645.4	32.1	14020.6	31624.7	15829.8
51	5.4	353	44089.0	31.7	13394.8	30694.2	16040.0
52	5.3	341	42567.4	31.3	12814.9	29752.5	16215.1
53	5.3	329	41079.3	31.0	12276.6	28802.7	16355.8
54	5.2	317	39623.1	30.6	11775.7	27847.4	16462.6
55	5.2	306	38197.6	30.3	11308.9	26888.7	16535.8
56	5.2	294	36801.2	29.9	10872.9	25928.4	16575.7
57	5.2	283	35432.6	29.6	10464.8	24967.8	16582.5
58	5.2	273	34090.4	29.2	10082.2	24008.2	16556.2
59	5.2	262	32773.1	28.9	9722.7	23050.4	16496.6
60	5.3	252	31479.5	28.6	9384.3	22095.2	16403.6
61	5.3	242	30208.3	28.2	9065.0	21143.3	16276.9
62	5.3	232	28958.0	27.9	8763.0	20195.0	16115.9
63	5.4	222	27727.6	27.6	8476.8	19250.8	15920.2
64	5.5	212	26515.8	27.3	8204.9	18310.9	15689.0
65	5.6	203	25321.4	26.9	7945.8	17375.7	15421.6
66	5.7	193	24143.2	26.6	7698.1	16445.2	15117.0
67	5.8	184	22980.1	26.3	7460.5	15519.6	14774.1
68	6.0	175	21830.9	25.9	7231.8	14599.1	14391.8
69	6.1	166	20694.4	25.6	7010.7	13683.7	13968.6
70	6.3	157	19569.5	25.2	6795.8	12773.6	13503.2



Design Equations (Vector Analysis):

$a = c_{FS}*L_{CR}*\sin(90+f_{FS})/\sin(a-f_{FS})$

$b = W-a$

$P_A = b*\tan(a-f_{FS})$

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\ max}$ 16582.5 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall) At-Rest= γ*(1-sin(ϕ))

$EFP = 2*P_A/H^2$

EFP 36.9 pcf 56.9 pcf

Design Wall for an Equivalent Fluid Pressure: **37 pcf** **57 pcf**

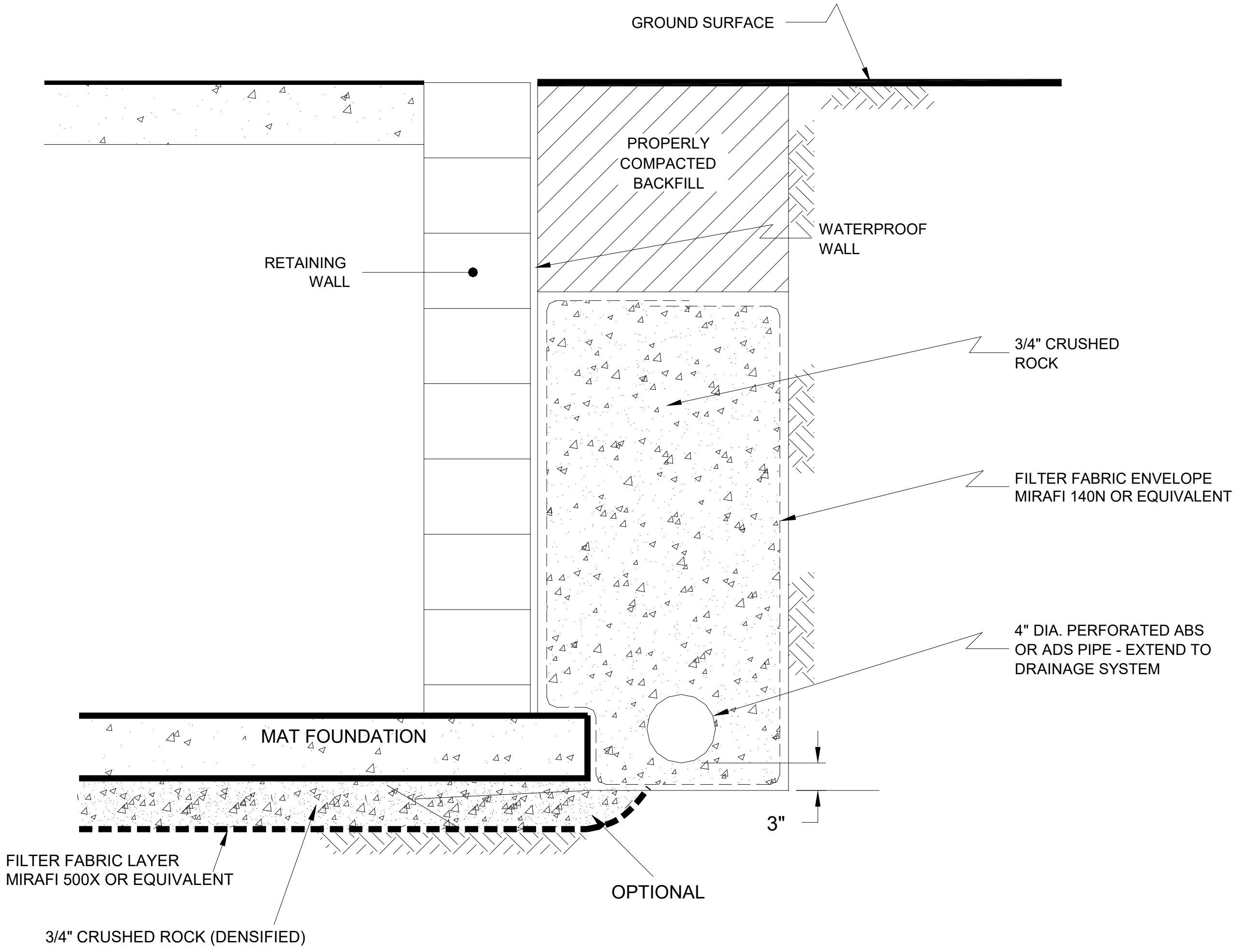
GEOCON
W E S T, I N C.

ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

RETAINING WALL PRESSURE CALCULATION

5700 & 5750 W. WILSHIRE BLVD.
LOS ANGELES, CALIFORNIA

DRAFTED BY: PZ	CHECKED BY: NDB	MARCH 2020	PROJECT NO. W1038-06-01A	FIG. 5



NO SCALE

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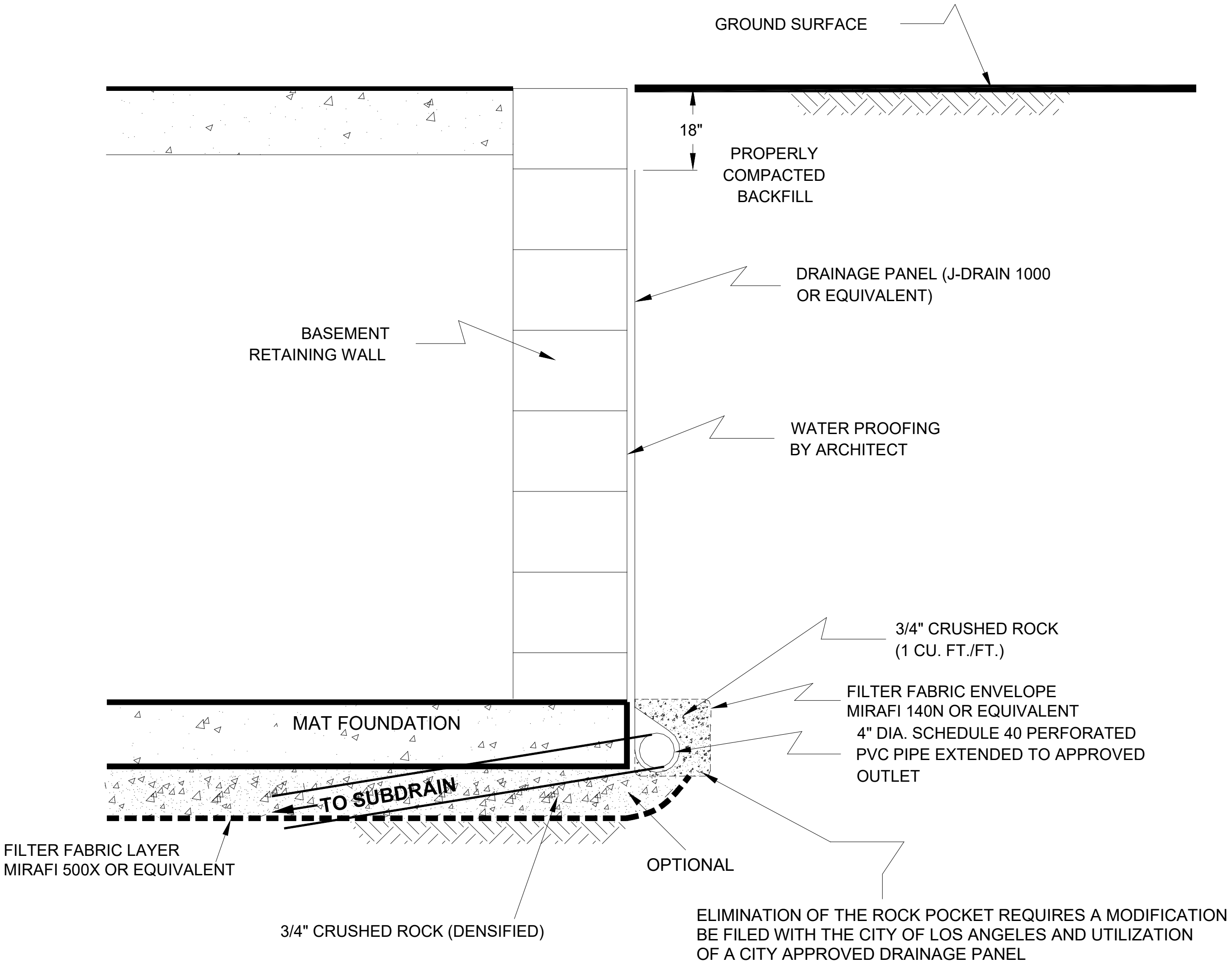
RETAINING WALL DRAIN DETAIL

5700 & 5750 W. WILSHIRE BLVD.
LOS ANGELES, CALIFORNIA

DRAFTED BY: PZ	CHECKED BY: NDB	MARCH 2020	PROJECT NO. W1038-06-01A	FIG. 6



NOTE: TOP OF DRAINAGE PANEL NOT MORE THAN
18 INCHES FROM GROUND SURFACE

NO SCALE

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RETAINING WALL DRAIN DETAIL

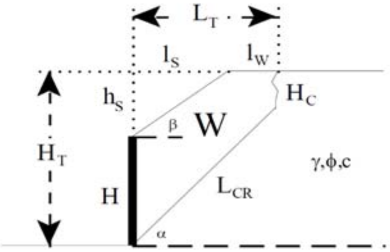
5700 & 5750 W. WILSHIRE BLVD.
LOS ANGELES, CALIFORNIA

DRAFTED BY: PZ	CHECKED BY: NDB	MARCH 2020	PROJECT NO. W1038-06-01A	FIG. 7

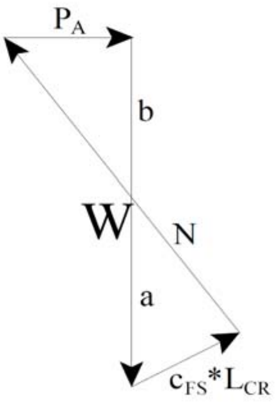
Shoring Design with Transitioned Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	32.00	feet
Slope Angle of Backfill	(b)	0.0	degrees
Height of Slope above Shoring	(h_s)	0.0	feet
Horizontal Length of Slope	(l_s)	0.0	feet
Total Height (Shoring + Slope)	(H_T)	32.0	feet
Unit Weight of Retained Soils	(g)	125.0	pcf
Friction Angle of Retained Soils	(f)	33.0	degrees
Cohesion of Retained Soils	(c)	320.0	psf
Factor of Safety	(FS)	1.25	
Factored Parameters:	(f_{FS})	27.5	degrees
	(c_{FS})	256.0	psf



Failure Angle (a) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
45	8.5	476	59457.9	33.2	25015.4	34442.5	10890.7
46	8.2	462	57721.0	33.1	23604.9	34116.1	11446.2
47	8.0	448	55983.9	32.9	22314.1	33669.8	11954.1
48	7.7	434	54255.8	32.6	21131.1	33124.7	12415.8
49	7.5	420	52543.4	32.4	20045.0	32498.4	12832.2
50	7.4	407	50851.0	32.1	19046.0	31805.0	13204.6
51	7.2	393	49181.6	31.9	18125.3	31056.3	13533.9
52	7.1	380	47536.9	31.6	17275.0	30261.9	13821.1
53	7.0	367	45918.1	31.3	16488.2	29429.9	14066.9
54	6.9	355	44325.5	31.0	15758.6	28566.9	14272.2
55	6.9	342	42759.1	30.7	15080.6	27678.5	14437.4
56	6.8	330	41218.7	30.4	14449.3	26769.4	14563.0
57	6.8	318	39703.7	30.1	13860.3	25843.4	14649.5
58	6.7	306	38213.4	29.8	13309.5	24903.9	14697.0
59	6.7	294	36746.9	29.5	12793.3	23953.6	14705.8
60	6.8	282	35303.3	29.1	12308.5	22994.8	14675.8
61	6.8	271	33881.6	28.8	11852.1	22029.6	14607.0
62	6.8	260	32480.8	28.5	11421.4	21059.4	14499.2
63	6.9	249	31099.8	28.2	11013.9	20085.8	14351.9
64	7.0	238	29737.4	27.9	10627.4	19110.0	14164.9
65	7.1	227	28392.5	27.5	10259.5	18133.0	13937.6
66	7.2	217	27064.0	27.2	9908.3	17155.7	13669.2
67	7.3	206	25750.7	26.8	9571.8	16178.9	13359.1
68	7.5	196	24451.3	26.5	9248.1	15203.2	13006.3
69	7.6	185	23164.6	26.1	8935.1	14229.5	12610.0
70	7.9	175	21889.4	25.7	8631.1	13258.3	12169.0



Design Equations (Vector Analysis):

$a = c_{FS}*L_{CR}*\sin(90+f_{FS})/\sin(a-f_{FS})$

$b = W-a$

$P_A = b*\tan(a-f_{FS})$

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A, max}$ 14705.8 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 28.7 pcf

Design Shoring for an Equivalent Fluid Pressure: 29 pcf

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SHORING PRESSURE CALCULATION

5700 & 5750 W. WILSHIRE BLVD.
LOS ANGELES, CALIFORNIA

MARCH 2020 | PROJECT NO. W1038-06-01A | FIG. 8

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

The site was explored on February 14 and 15, 2020 by excavating one 4⅞-inch diameter boring to a depth of approximately 150 feet below the existing ground surface using a truck-mounted mud-rotary drilling machine. Representative and relatively undisturbed samples were obtained by driving a 3-inch O. D. California Modified Sampler into the “undisturbed” soil mass with blows from a 140-pound auto-hammer falling 30 inches. The California Modified Sampler was equipped with 1-inch high by 2⅜-inch diameter brass sampler rings to facilitate soil removal and testing. Standard Penetration Tests (SPTs) were performed. A geophysical survey consisting of down-hole suspension PS logging was performed in the boring as a part of the site exploration.

The soil conditions encountered in the borings were visually examined, classified and logged in general accordance with the Unified Soil Classification System (USCS). The logs of the boring are presented on Figure A1. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained. The logs also include our interpretation of the conditions between sampling intervals. Therefore, the logs contain both observed and interpreted data. We determined the lines designating the interface between soil materials on the logs using visual observations, penetration rates, excavation characteristics and other factors. The transition between materials may be abrupt or gradual. Where applicable, the logs were revised based on subsequent laboratory testing. The location of the boring is shown on Figure 2A.

BORING 1

ELEV. (MSL.) **--** DATE COMPLETED **2/14/2020**

EQUIPMENT **MUD ROTARY** BY: **JMH**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL** Sandy Clay, firm, moist, bluish gray.			
2								
4					Clayey Sand, poorly graded, loose, moist, bluish gray, fine- to medium-grained.			
6					Sand, poorly graded, loose, moist, bluish gray, fine- to medium-grained.			
8					**OLDER ALLUVIUM** Silt, stiff, slightly moist, bluish gray.			
10	B1@10'					24	103.0	25.9
12				ML				
14								
	B1@15'				- dark bluish gray with reddish brown mottles, trace fine-grained sand	26	107.6	24.1
16								
18								
					Sand, dense, slighlty moist, black, tar.			
20	B1@20'					65	107.3	12.4
22								
24				SP				
	B1@25'				- fine- to medium-grained	79	121.4	8.1
26								
28								

Figure A1,
Log of Boring 1, Page 1 of 6

W1038-06-01A BORING LOGS.GPJ

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
☒ ... DISTURBED OR BAG SAMPLE
◪ ... CHUNK SAMPLE
▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. W1038-06-01A

BORING 1

ELEV. (MSL.) -- DATE COMPLETED 2/14/2020

EQUIPMENT **MUD ROTARY** BY: **JMH**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B1@30'		▼	SP		83	124.9	3.1
					Silt, stiff, slightly moist, dark bluish gray with black mottles, tar.			
32								
	B1@32.5'					29	87.2	25.9
34								
	B1@35'				- hard, dark blackish gray, decrease in tar	68	96.0	29.0
36				ML				
38	B1@37.5'				- black, increase in tar	76	109.2	10.7
40	B1@40'				- moist, bluish gray with dark gray mottles	45	97.9	25.1
					Sand, poorly graded, dense, moist, black, fine-grained, tar.			
42								
	B1@42.5'					80	113.5	9.4
44								
	B1@45'					70	116.5	5.7
46								
48	B1@47.5'				- no recovery	95	120.7	14.8
50	B1@49.5'			SP	- fine- to medium-grained, trace fine gravel	100	107.2	3.9
52								
54								
	B1@54.5'				- very dense	56		
56								
58								

W1038-06-01A BORING LOGS.GPJ

Figure A1,
Log of Boring 1, Page 2 of 6

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
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PROJECT NO. W1038-06-01A

BORING 1

ELEV. (MSL.) -- DATE COMPLETED 2/14/2020

EQUIPMENT MUD ROTARY BY: JMH

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
60	B1@59.5'				- fine-grained	100	111.4	3.9
62								
64								
	B1@65' ■					88		
66								
68								
70	B1@69.5' ■			SP		100	114.8	4.2
72								
74								
	B1@75' ■				- trace gravel	65		
76								
78								
80	B1@79.5' ■					100		
82								
84								
					Sandy Silt, hard, moist, black, tar.			
	B1@85' ■					59		
86				ML				
88								
				ML	Silt, hard, moist, blackish brown, trace tar.			

Figure A1,
Log of Boring 1, Page 3 of 6

W1038-06-01A BORING LOGS.GPJ

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
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PROJECT NO. W1038-06-01A

BORING 1

ELEV. (MSL.) **--** DATE COMPLETED **2/14/2020**

EQUIPMENT **MUD ROTARY** BY: **JMH**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
90	B1@89.5'					100	92.5	22.8
92								
94								
	B1@95' ◧				- stiff	32		
96				ML				
98								
100	B1@100' ■				- hard, slightly moist	100	81.1	30.0
102								
104								
	B1@104.5' ■				Silty Sand with Gravel, poorly graded, very dense, wet, blackish brown, fine- to medium-grained, some tar.	50 (5")		
106								
108								
110	B1@110' ■			SM	- dense, saturated	55	74.2	28.2
112								
114								
	B1@114.5' ☐				- no recovery	50 (6")		
116								
					Silt, hard, wet, blackish brown, trace tar.			
118				ML				

W1038-06-01A BORING LOGS.GPJ

Figure A1,
Log of Boring 1, Page 4 of 6

SAMPLE SYMBOLS
☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
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GEOCON

PROJECT NO. W1038-06-01A

BORING 1

ELEV. (MSL.) -- DATE COMPLETED **2/14/2020**

EQUIPMENT **MUD ROTARY** BY: **JMH**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
120	B1@120'					50 (5")	91.3	27.0
122								
				ML				
124								
	B1@125'				- trace gravel	50 (5")		
126								
					Silt with Gravel, hard, wet, blackish brown, trace tar, fine angular gravel.			
128								
130	B1@129.5			ML		50 (5")	108.2	14.2
132					Silt, hard, wet, blackish brown, trace tar.			
134								
136								
138								
140	B1@139.5					50 (6")	88.9	23.2
				ML				
142								
144								
146								
148								

W1038-06-01A BORING LOGS.GPJ

Figure A1,
Log of Boring 1, Page 5 of 6

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
☒ ... DISTURBED OR BAG SAMPLE
◪ ... CHUNK SAMPLE
▼ ... WATER TABLE OR SEEPAGE

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GEOCON

BORING 1

ELEV. (MSL.) **--** DATE COMPLETED **2/14/2020**

EQUIPMENT **MUD ROTARY** BY: **JMH**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
	B1@149.5				Total depth of boring: 150 feet Fill to 8 feet. Groundwater enountered at 31 feet. Backfilled with grout and patched. *Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.	50 (5")	93.9	20.6

Figure A1,
Log of Boring 1, Page 6 of 6

W1038-06-01A BORING LOGS.GPJ

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	▩ ... DISTURBED OR BAG SAMPLE	◪ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

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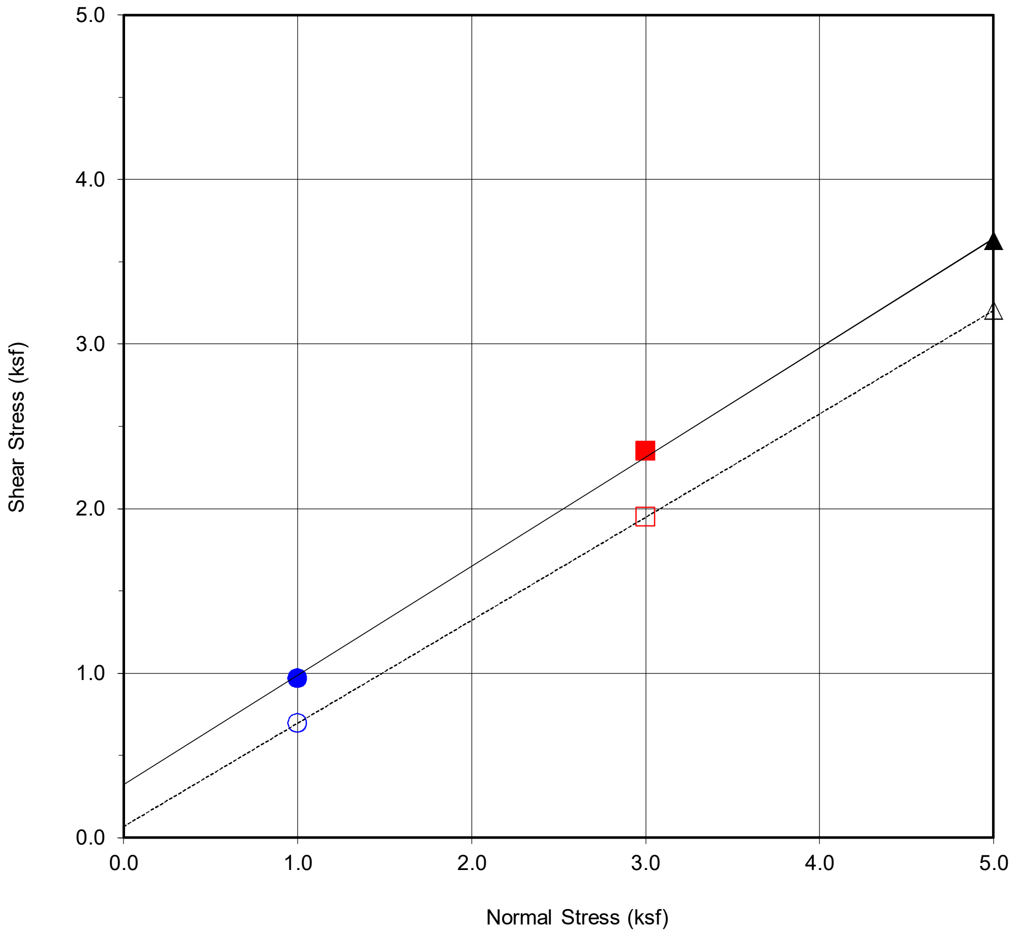
APPENDIX B

APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the "American Society for Testing and Materials (ASTM)", or other suggested procedures. Selected samples were tested for direct shear strength, consolidation and expansion characteristics, maximum dry density, corrosivity, in-place dry density and moisture content. The results of the laboratory tests are summarized in Figures B1 through B17. The in-place dry density and moisture content of the samples tested are presented on the boring logs, Appendix A.



Boring No.	**B1**
Sample No.	**B1@10'**
Depth (ft)	**10**
Sample Type:	Ring

Soil Identification:

Bluish Gray Silt (ML)

Strength Parameters

	C (psf)	ϕ (°)
Peak	321	33.6
Ultimate	68	32.1

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.97	■ 2.35	▲ 3.63
Shear Stress @ End of Test (ksf)	○ 0.70	□ 1.95	△ 3.20
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	33.2	25.9	25.9
Initial Dry Density (pcf)	98.7	103.4	103.8
Initial Degree of Saturation (%)	126.4	110.8	112.2
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	27.5	24.5	24.0

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

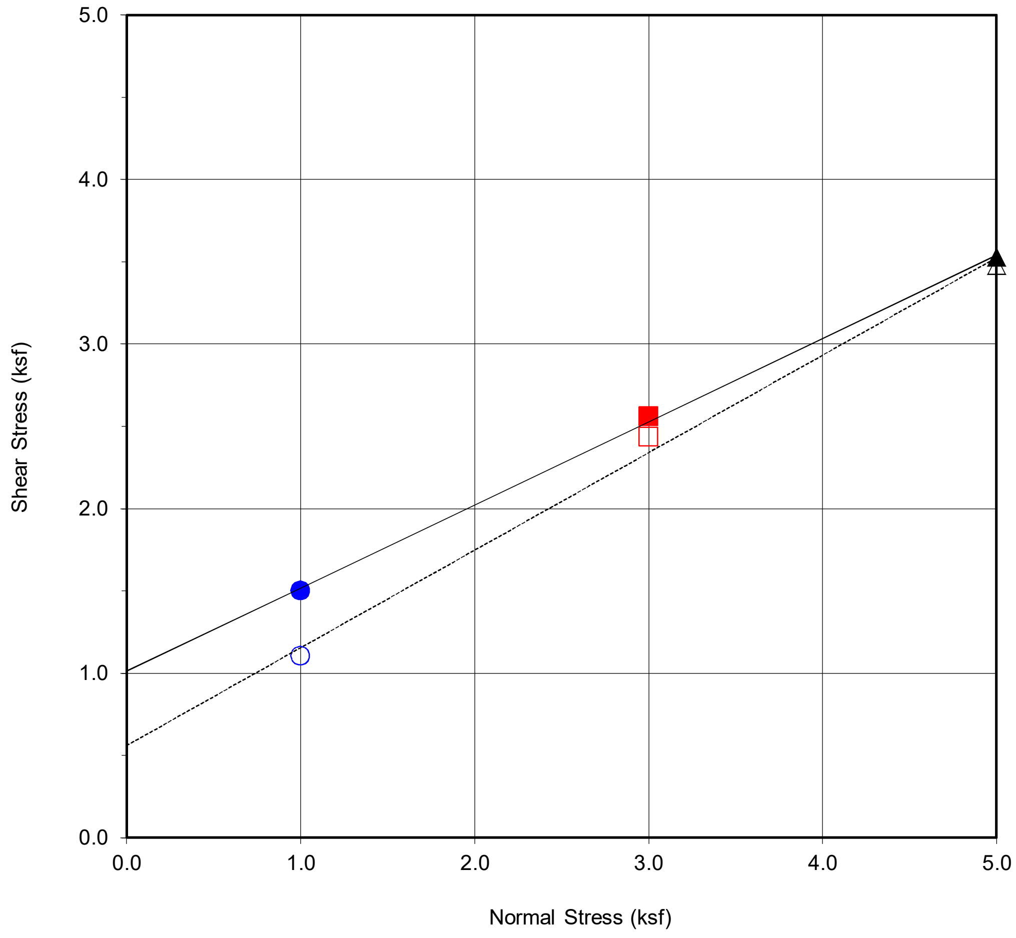
Checked by: PZ

Project No.: W1038-06-01A

5700 & 5750 W. Wilshire Blvd.
Los Angeles, California

March 2020

Figure B1



Boring No.	**B1**
Sample No.	**B1@20'**
Depth (ft)	**20**
Sample Type:	Ring

Soil Identification:		
Black Poorly Graded Sand (SP)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	1011	26.8
Ultimate	560	30.7

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.50	■ 2.56	▲ 3.53
Shear Stress @ End of Test (ksf)	○ 1.11	□ 2.43	△ 3.48
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	11.4	12.4	12.3
Initial Dry Density (pcf)	109.2	109.4	109.9
Initial Degree of Saturation (%)	56.5	62.2	62.5
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	14.3	14.2	14.0

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

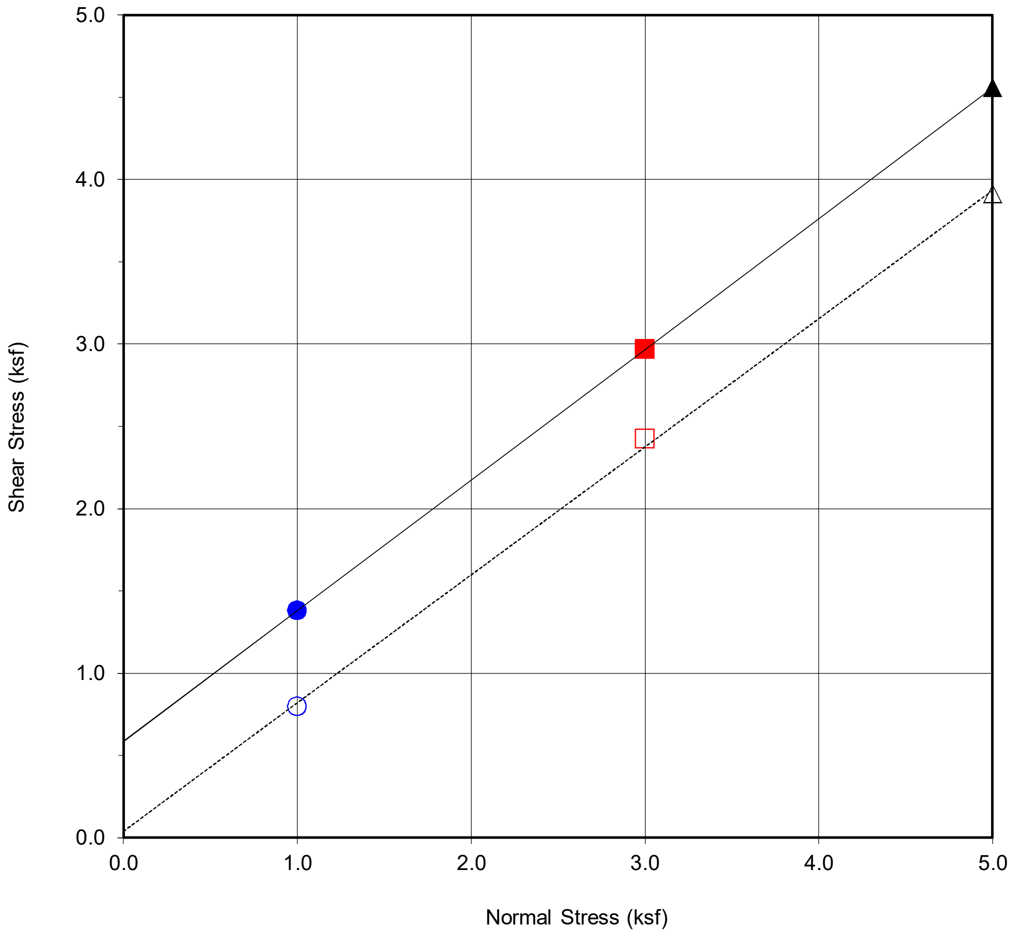
Checked by: PZ

Project No.: W1038-06-01A

5700 & 5750 W. Wilshire Blvd.
Los Angeles, California

March 2020

Figure B2



Boring No.	**B1**
Sample No.	**B1@30'**
Depth (ft)	**30**
Sample Type:	Ring

Soil Identification:

Black Poorly Graded Sand (SP)

Strength Parameters

	C (psf)	ϕ (°)
Peak	583	38.5
Ultimate	39	37.9

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.38	■ 2.97	▲ 4.56
Shear Stress @ End of Test (ksf)	○ 0.80	□ 2.42	△ 3.91
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	12.2	12.4	12.4
Initial Dry Density (pcf)	107.4	106.9	104.6
Initial Degree of Saturation (%)	57.7	58.2	54.9
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	14.8	14.1	14.1

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

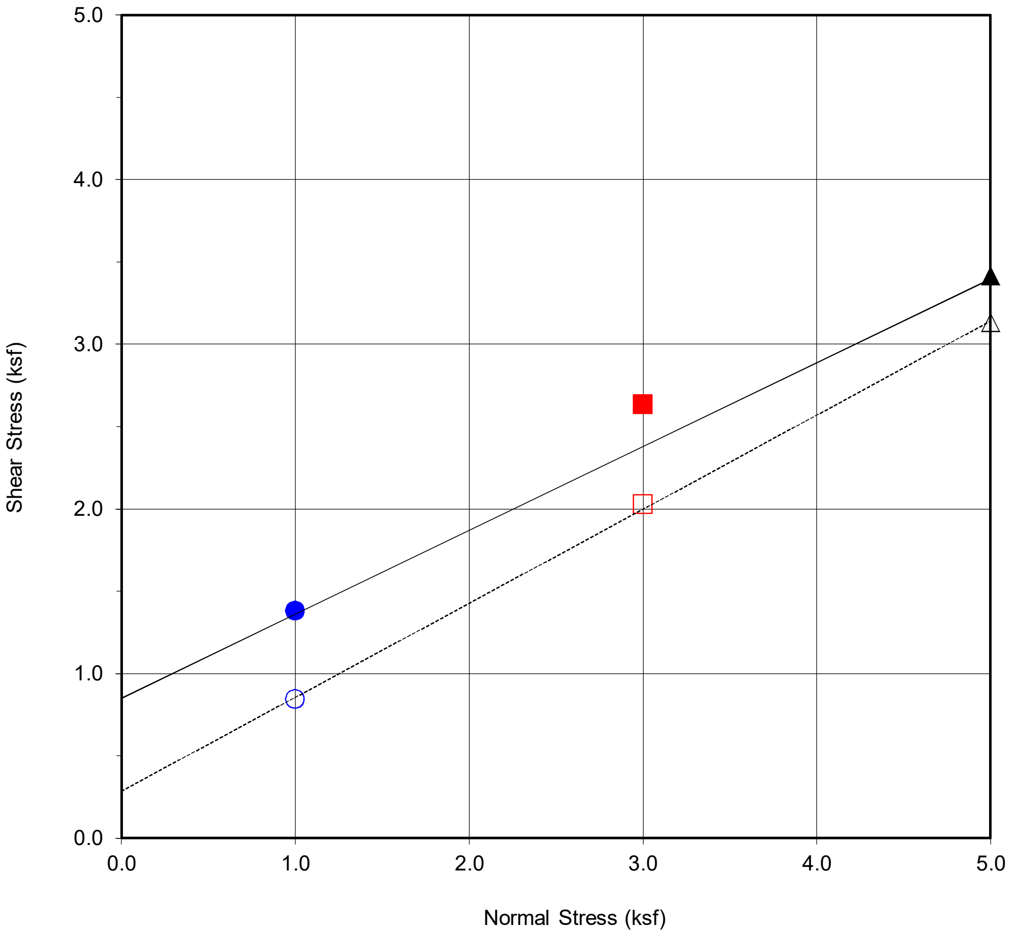
Checked by: PZ

Project No.: W1038-06-01A

5700 & 5750 W. Wilshire Blvd.
Los Angeles, California

March 2020

Figure B3



Boring No.	**B1**
Sample No.	**B1@35**
Depth (ft)	**35**
Sample Type:	Ring

Soil Identification:		
Gray Silt (ML)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	850	27.0
Ultimate	285	29.7

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.38	■ 2.63	▲ 3.42
Shear Stress @ End of Test (ksf)	○ 0.84	□ 2.02	△ 3.13
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	30.4	33.1	32.7
Initial Dry Density (pcf)	89.3	88.9	85.8
Initial Degree of Saturation (%)	92.3	99.7	91.4
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	32.8	33.1	31.5

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

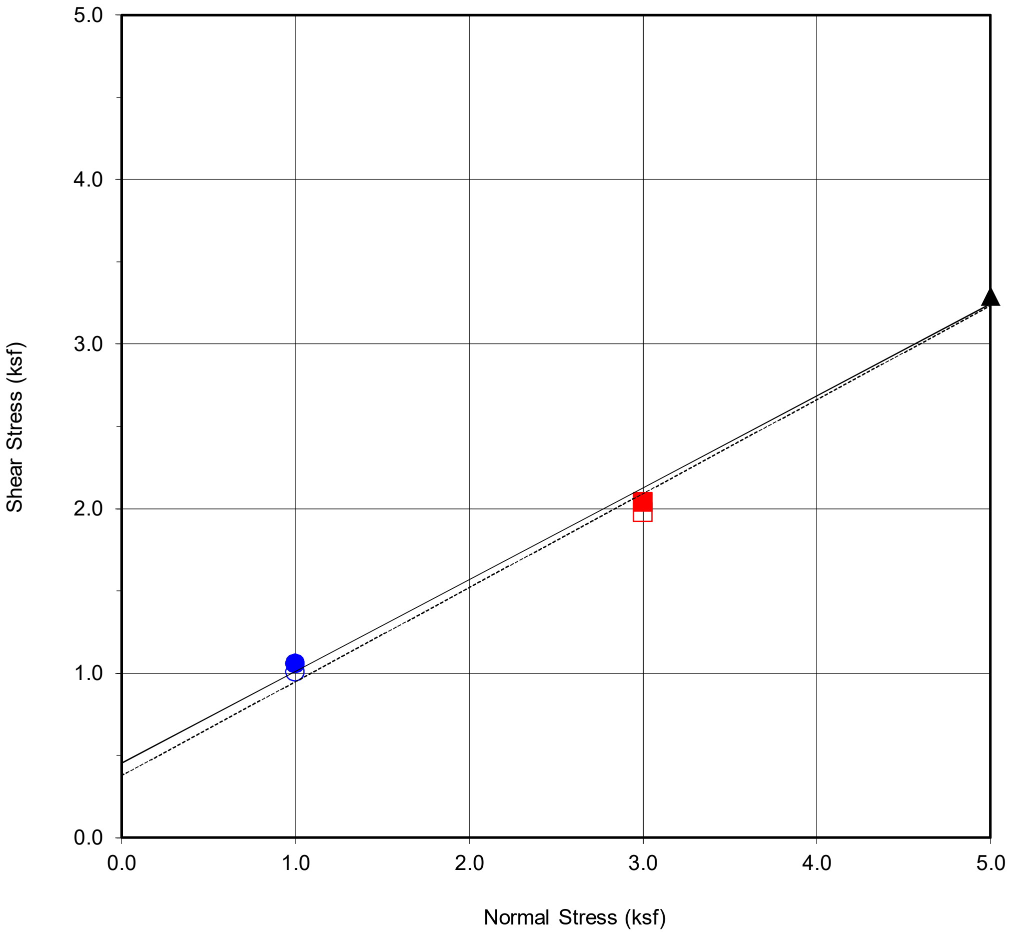
Checked by: PZ

Project No.: W1038-06-01A

5700 & 5750 W. Wilshire Blvd.
Los Angeles, California

March 2020

Figure B4



Boring No.	**B1**
Sample No.	**B1@40'**
Depth (ft)	**40**
Sample Type:	Ring

Soil Identification:		
Gray Silt (ML)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	450	29.2
Ultimate	377	29.7

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.06	■ 2.04	▲ 3.29
Shear Stress @ End of Test (ksf)	○ 1.01	□ 1.97	△ 3.29
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	18.9	18.1	18.2
Initial Dry Density (pcf)	104.6	106.5	110.2
Initial Degree of Saturation (%)	83.4	84.0	92.6
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	21.5	18.9	18.4

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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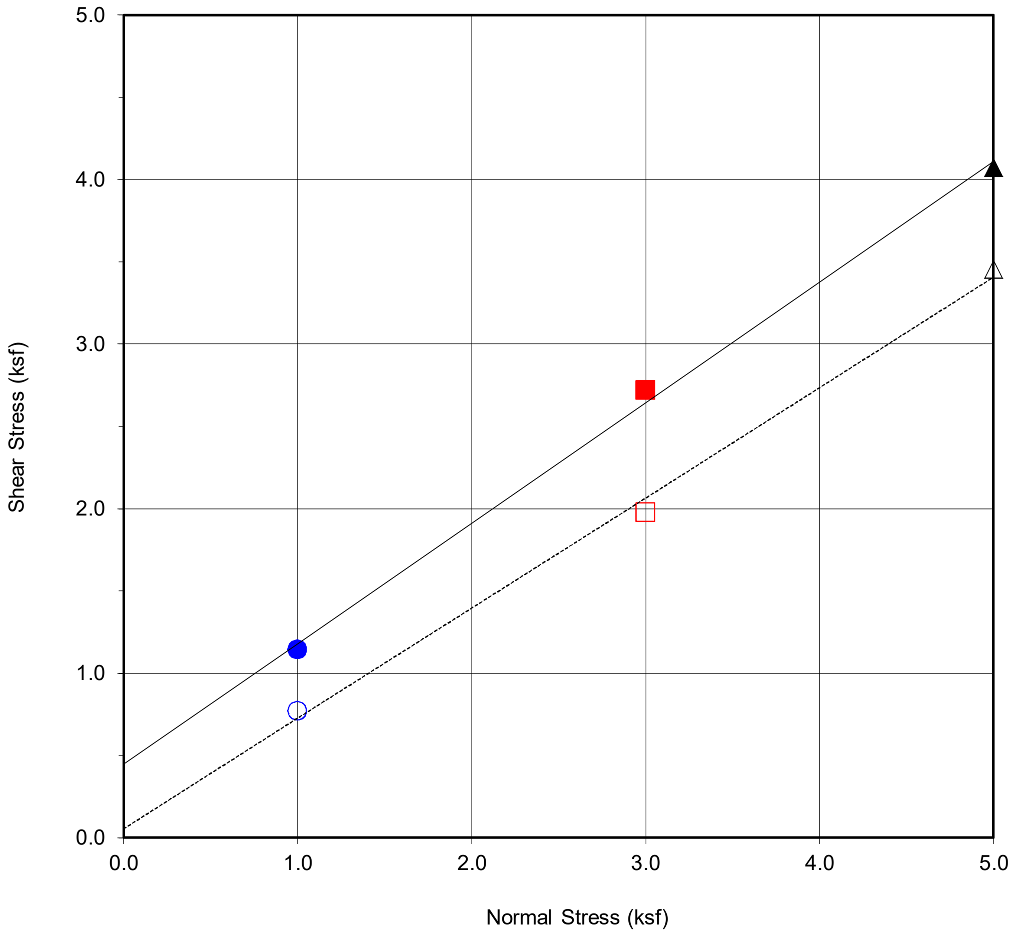
March 2020

Figure B5



Boring No.	**B1**
Sample No.	**B1@45'**
Depth (ft)	**45**
Sample Type:	Ring

Soil Identification:

Black Poorly Graded Sand (SP)

Strength Parameters

	C (psf)	ϕ (°)
Peak	446	36.2
Ultimate	54	33.8

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.14	■ 2.72	▲ 4.07
Shear Stress @ End of Test (ksf)	○ 0.77	□ 1.97	△ 3.45
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	8.3	9.0	8.6
Initial Dry Density (pcf)	108.6	109.0	109.3
Initial Degree of Saturation (%)	40.5	44.3	43.0
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	9.4	9.8	9.2

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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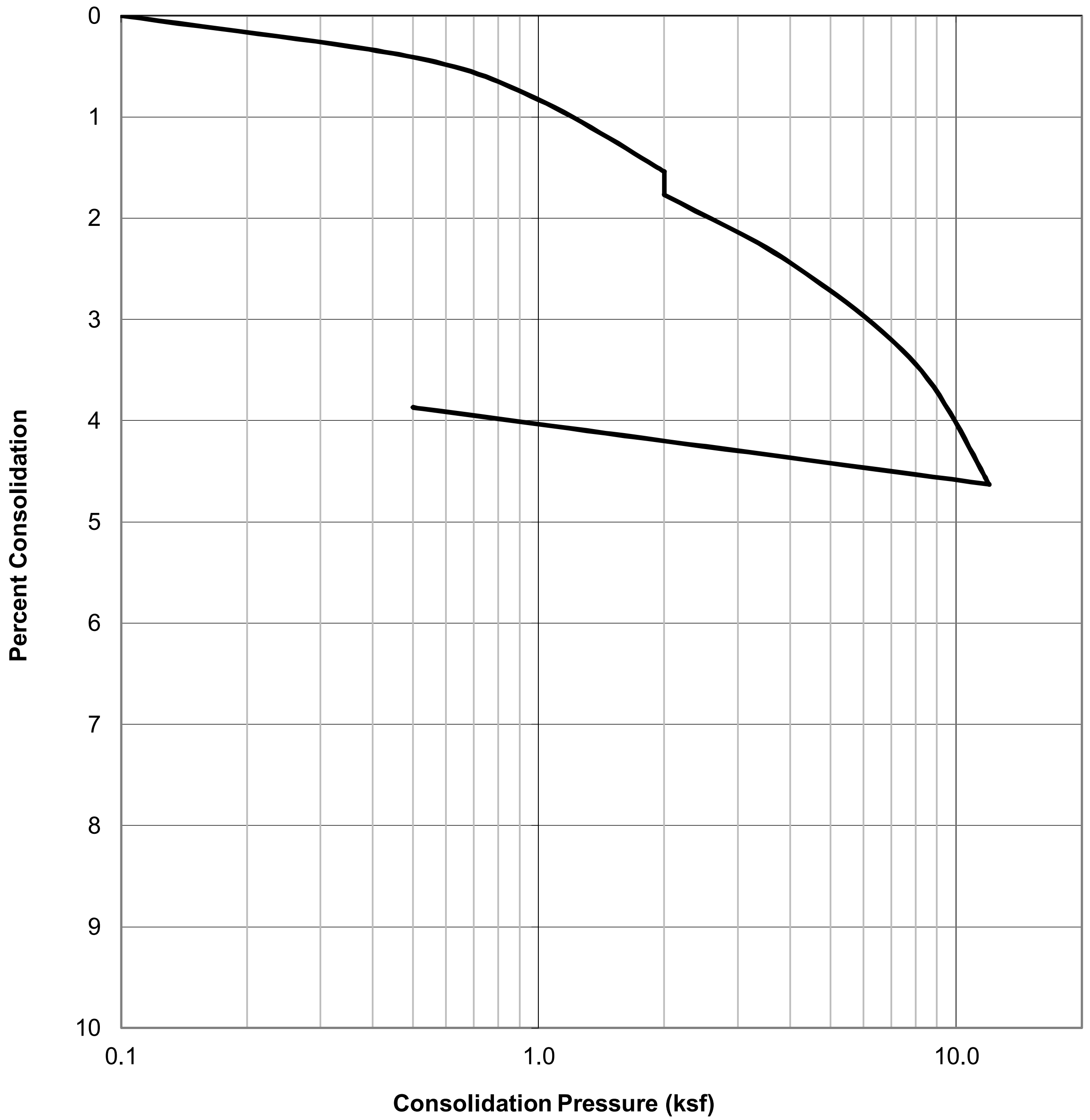
Figure B6

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@30	Black Poorly Graded Sand (SP)	103.3	11.7	12.3

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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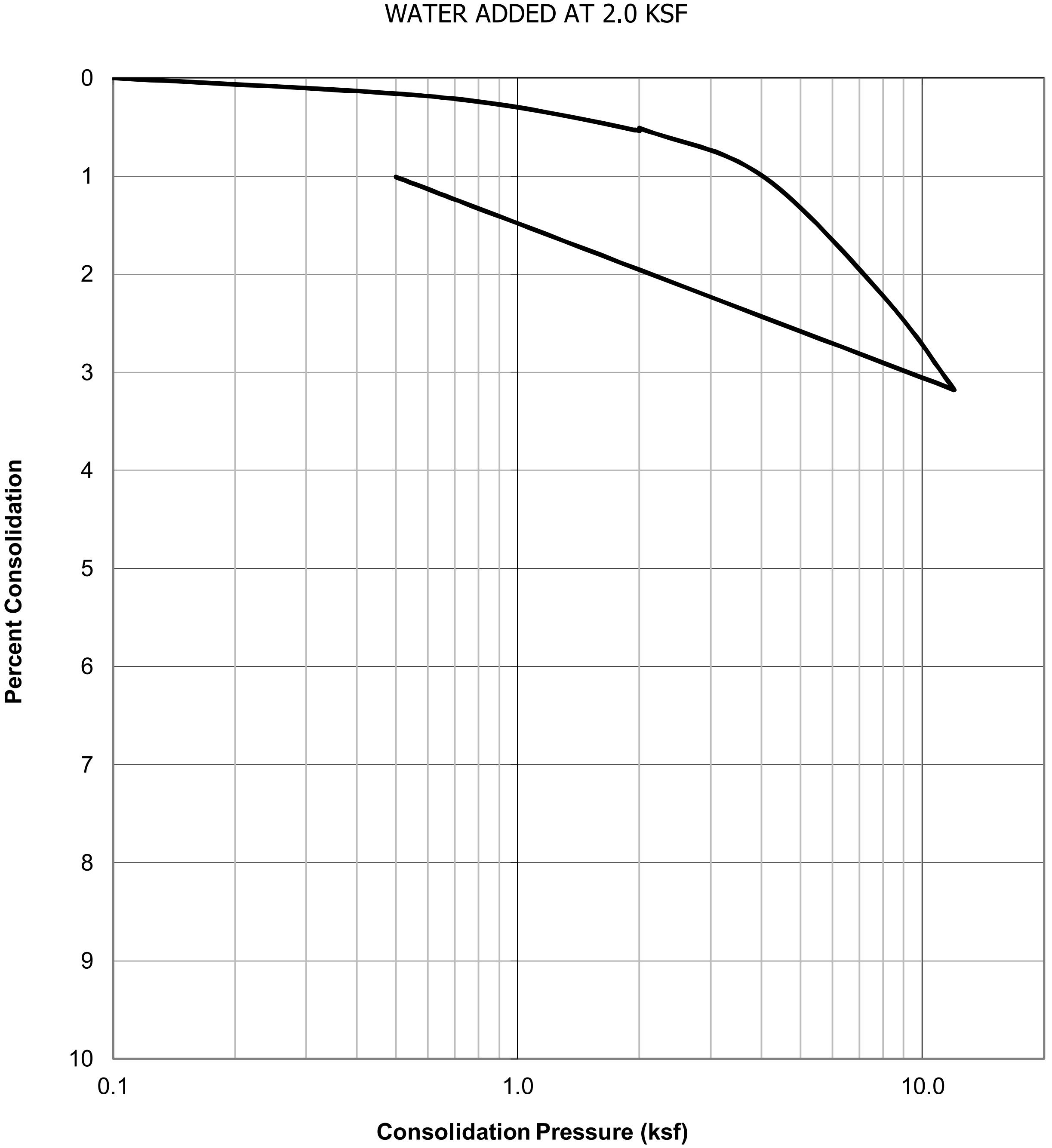
Figure B7



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@35	Gray Silt (ML)	90.5	29.0	29.8

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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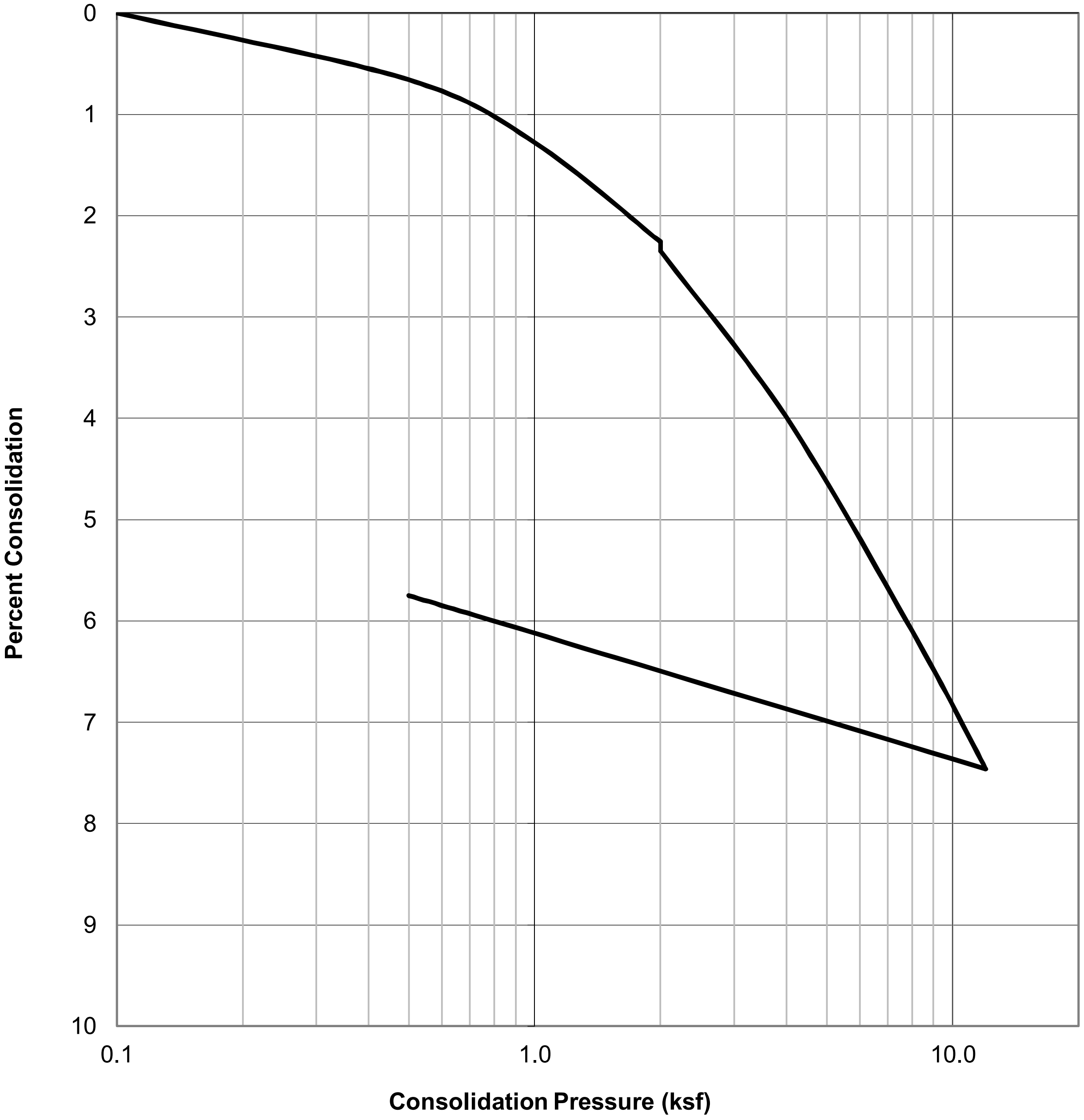
Figure B8

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@37.5	Black Silt (ML)	104.9	10.7	10.8

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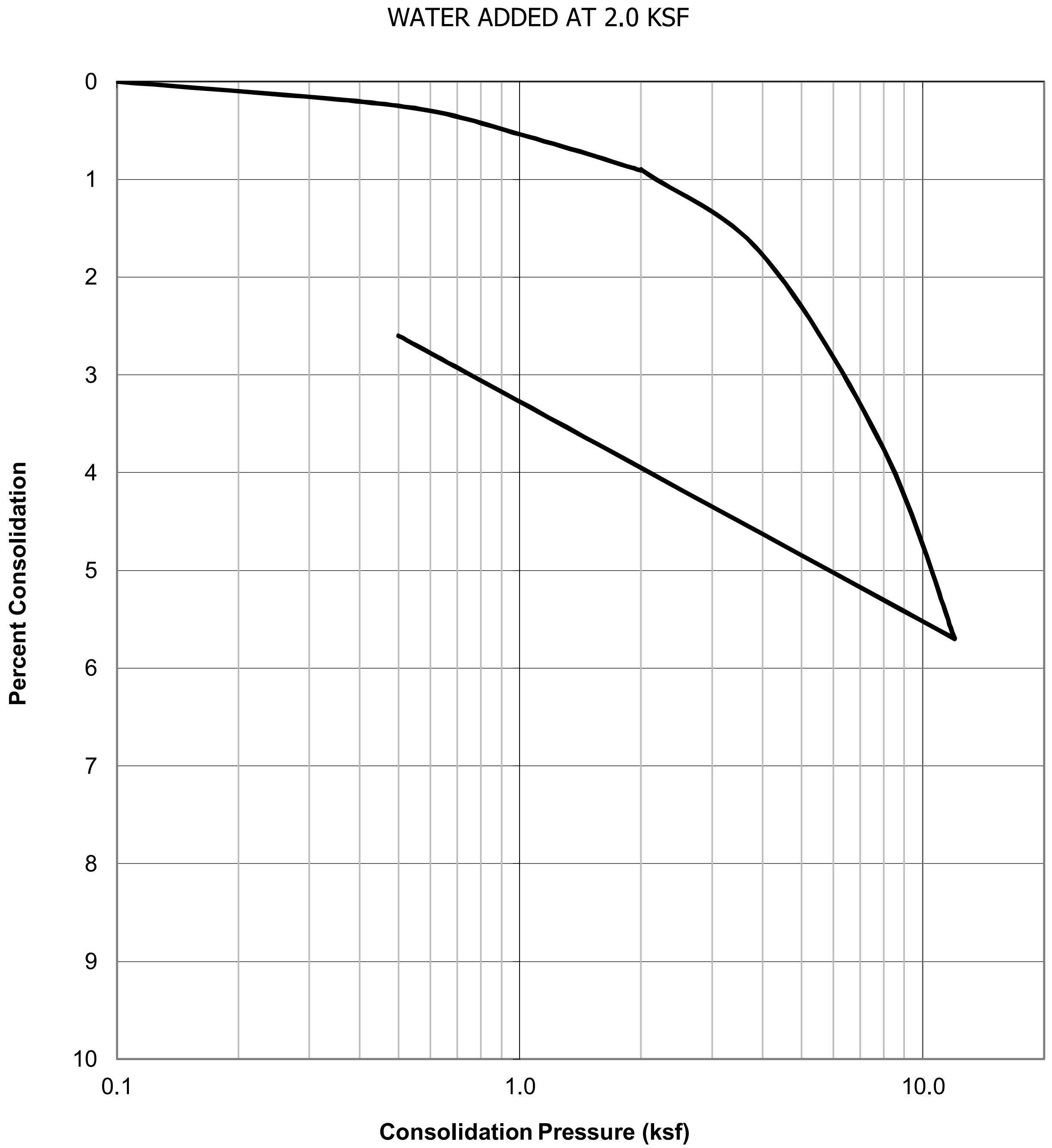

SAMPLE ID.	**SOIL TYPE**	**DRY DENSITY (PCF)**	**INITIAL MOISTURE (%)**	**FINAL MOISTURE (%)**
B1@40	Gray Silt (ML)	89.5	25.1	25.7

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CONSOLIDATION TEST RESULTS

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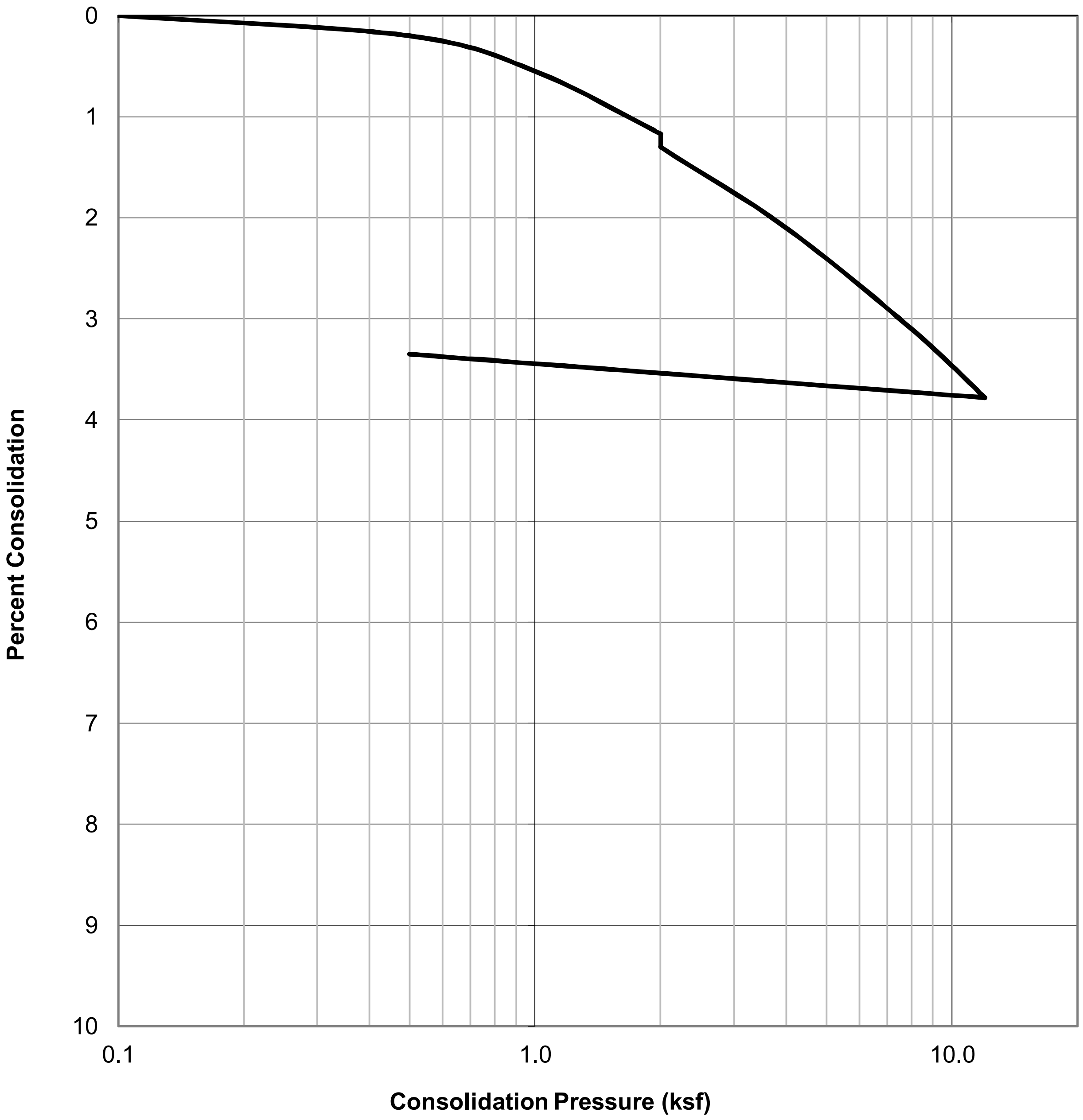
Figure B10

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@42.5	Black Poorly Graded Sand (SP)	114.9	9.4	6.8

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CONSOLIDATION TEST RESULTS

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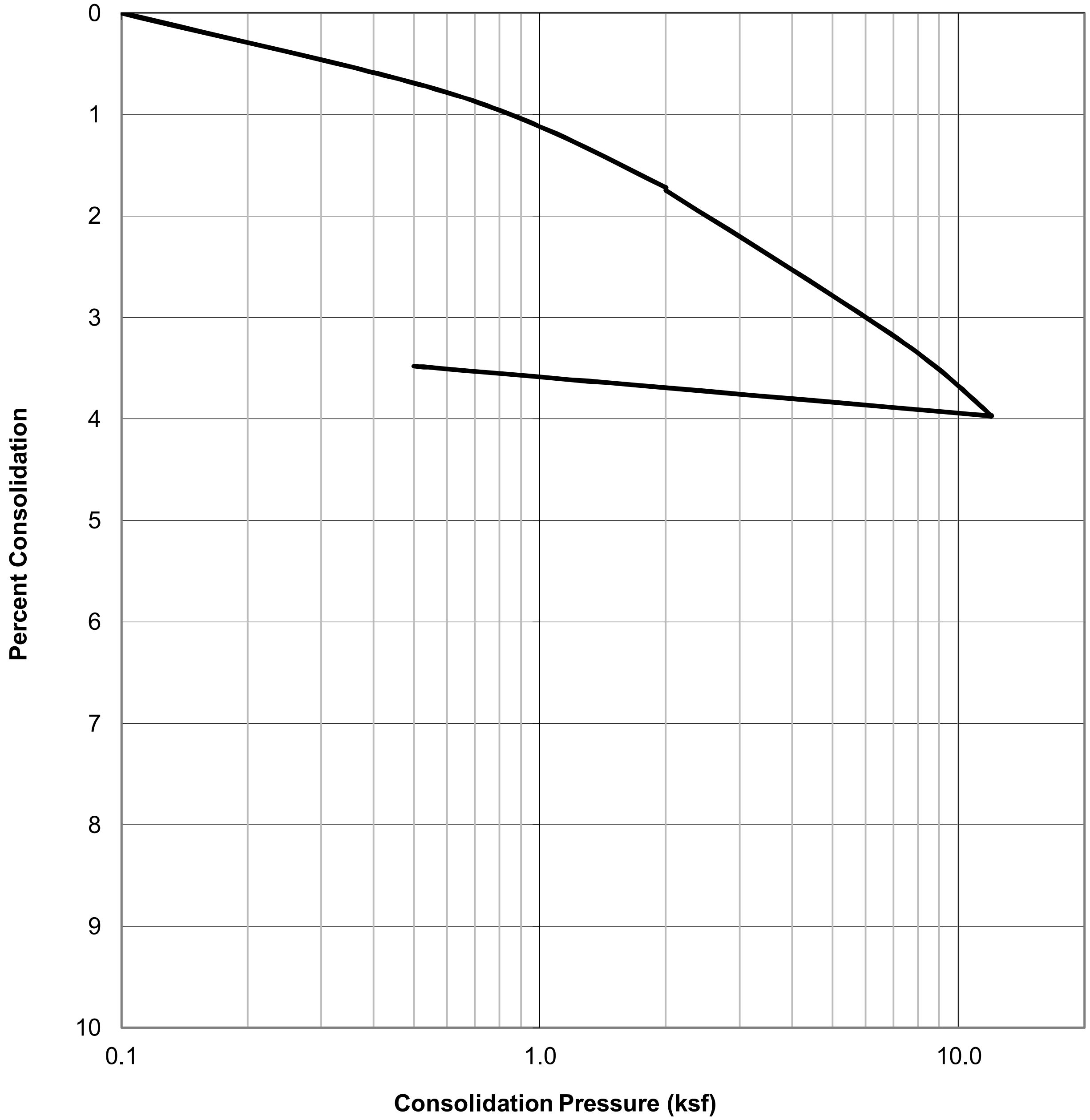
Figure B11

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@45	Black Poorly Graded Sand (SP)	111.9	5.7	5.4

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CONSOLIDATION TEST RESULTS

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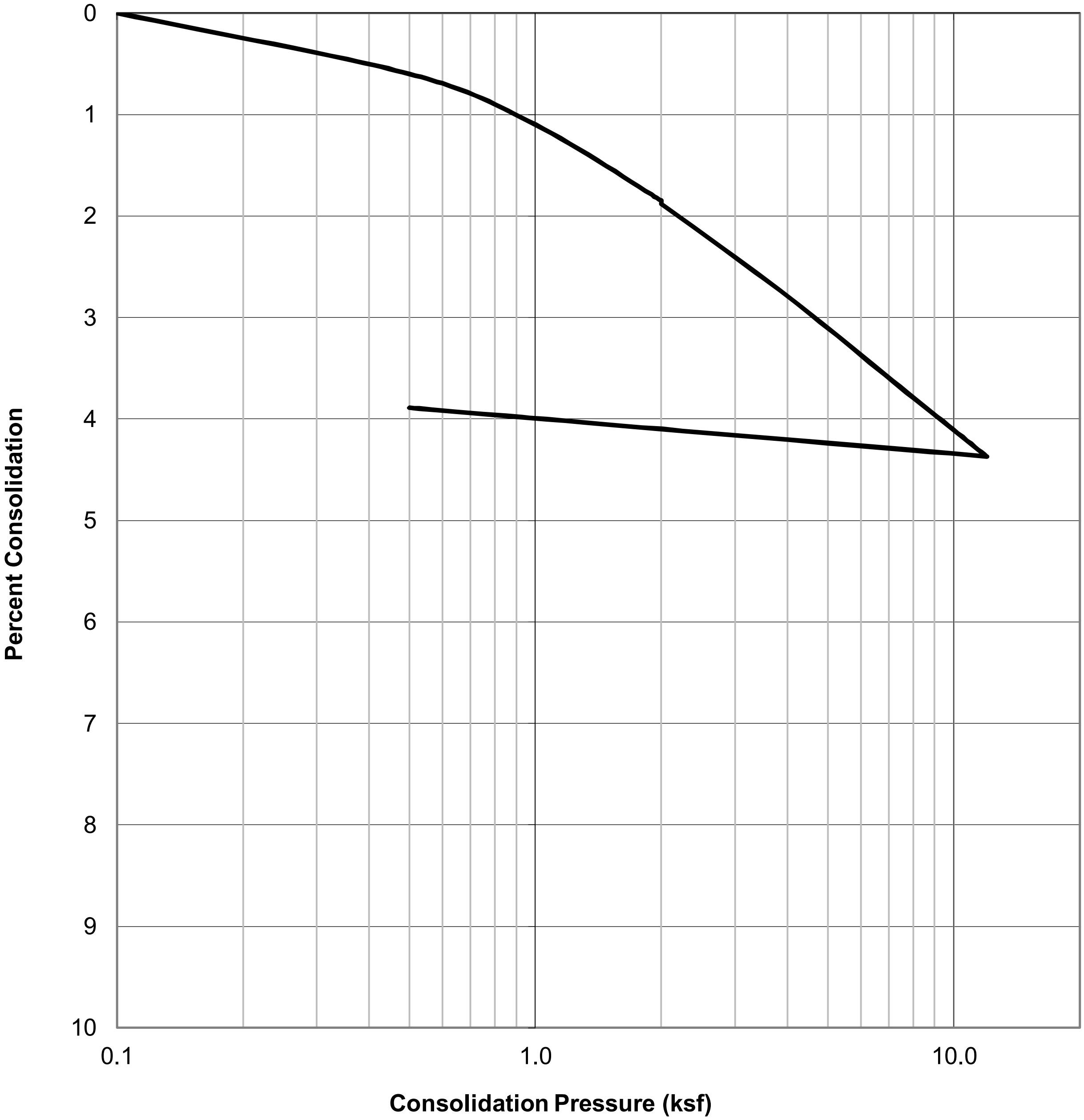
Figure B12

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@49.5	Black Poorly Graded Sand (SP)	112.1	3.9	3.6

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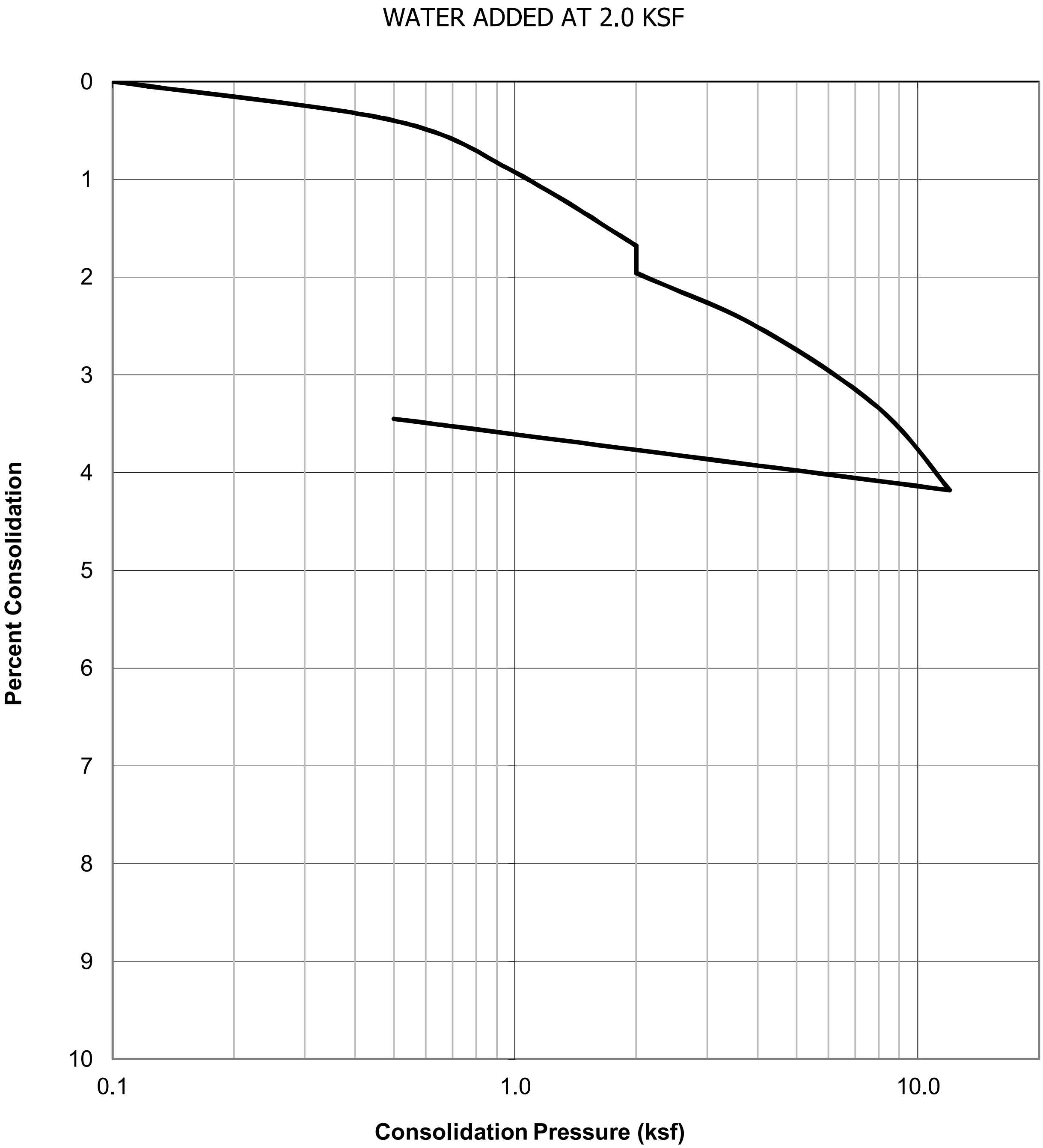

SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@59.5	Black Poorly Graded Sand (SP)	112.1	3.9	3.6

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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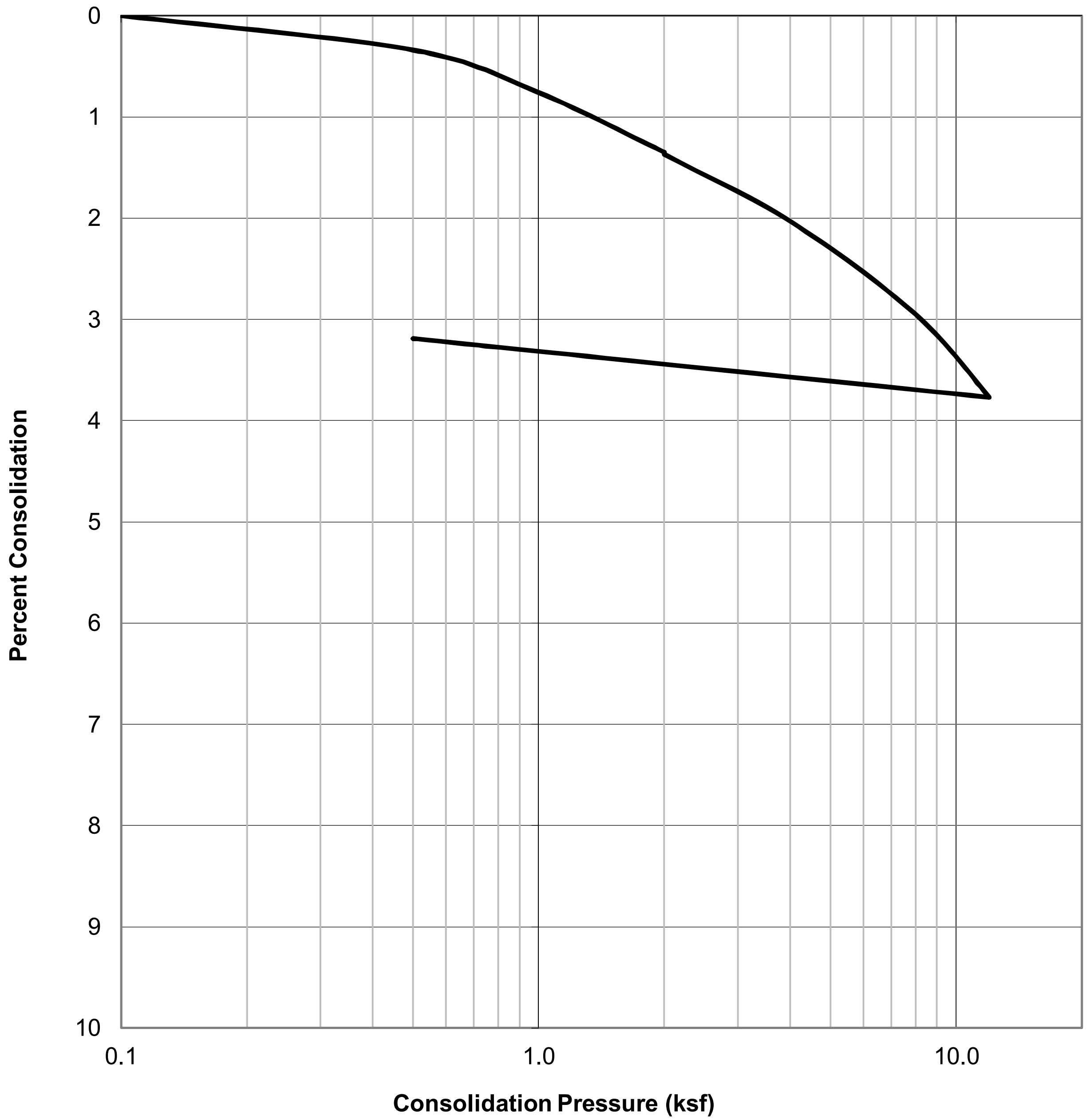
Figure B14

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@69.5	Black Poorly Graded Sand (SP)	109.8	4.2	4.0

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CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: PZ

Project No.: W1038-06-01A

5700 & 5750 W. Wilshire Blvd.
Los Angeles, California

March 2020

Figure B15

SUMMARY OF LABORATORY EXPANSION INDEX TEST RESULTS
ASTM D 4829-11

Sample No.	Moisture Content (%)		Dry Density (pcf)	Expansion Index	*UBC Classification	**CBC Classification
	Before	After				
B1 @ 32.5'	24.9	26.2	92.8	18	Very Low	Non-Expansive

* Reference: 1997 Uniform Building Code, Table 18-I-B.

** Reference: 2019 California Building Code, Section 1803.5.3

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LABORATORY TEST RESULTS

5700 & 5750 W. WILSHIRE BLVD.
LOS ANGELES, CALIFORNIA

DRAFTED BY: PZ
CHECKED BY: NDB

MARCH 2020
PROJECT NO. W1038-06-01A
FIG. B16

SUMMARY OF LABORATORY POTENTIAL OF HYDROGEN (pH) AND RESISTIVITY TEST RESULTS
CALIFORNIA TEST NO. 643

Sample No.	**pH**	**Resistivity (ohm centimeters)**
B1 @ 32.5	8.0	1000 (Severely Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS
EPA NO. 325.3

Sample No.	Chloride Ion Content (%)
B1@32.5'	0.051

SUMMARY OF LABORATORY WATER SOLUBLE SULFATE TEST RESULTS
CALIFORNIA TEST NO. 417

Sample No.	**Water Soluble Sulfate (% SQ_4)**	**Sulfate Exposure***
B1@32.5	0.028	S0

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CORROSIVITY TEST RESULTS

Checked by: PZ

Project No.: W1038-06-01A

5700 & 5750 W. Wilshire Blvd.
Los Angeles, California

March 2020

APPENDIX C

APPENDIX C

PRIOR REPORTS

3832

REPORT OF FOUNDATION INVESTIGATION
PROPOSED OFFICE STRUCTURES AND TOWNHOUSES
MUSEUM SQUARE

WILSHIRE BOULEVARD BETWEEN
CURSON STREET AND MASSELIN AVENUE
LOS ANGELES, CALIFORNIA
FOR
OSCHIN AND SNYDER
(OUR JOB NO. A-84021)

[redacted]

LeROY CRANDALL AND ASSOCIATES consulting geotechnical engineers, 711 n. alvarado st., los angeles, ca. 90026 (213) 413-3500, telex [illegible]

3832

March 12, 1984

TRACT: 5798
LOT: 1-8; 44-52; 85-93
(127-135); 290-293
JOB ADDRESS: 715-761 SIERRA BONITA AVE

Oschin and Snyder
5757 Wilshire Boulevard
Los Angeles, California 90036

(Our Job No. A-84021)

Attention: Mr. Russell Kubovec

Gentlemen:

Our "Report of Foundation Investigation, Proposed Office Structures and Townhouses, Museum Square, Wilshire Boulevard between Curson Street and Masselin Avenue, for Oschin and Snyder" is herewith submitted.

The scope of the investigation was planned in collaboration with Mr. Russell Kubovec. We were advised of the anticipated features of the proposed development by Mr. Kubovec and by Mr. Carl McLarand of Carl McLarand Associates, Inc.

The soils beneath the site consist of stiff clay and dense asphalt-impregnated sands. Gases are being given off by the asphaltic sands. Water was measured in three of our borings at depths of eight to nine feet.

The proposed buildings may be supported on spread-type foundations. Either individual or combined spread footings or a mat foundation may be used. Because of the water and asphalt conditions, either a hydrostatic design will be required, or a permanent subdrain system will be needed to prevent the development of hydrostatic pressures on the lower floors and walls. Also, gases are being given off by the asphaltic materials, so that a gas barrier or a suitable venting system should be provided to prevent gas from entering the basements. Maintenance of a permanent subdrain system within the asphaltic materials would present continuing operational difficulties. If a mat-type foundation is used, the foundation could be designed to resist the hydrostatic pressures and to provide an effective barrier to gases, thus eliminating the need for a subdrain system and venting system.

Because of the viscosity characteristics of the asphalt impregnated sands, subterranean construction extending into these deposits will be more difficult than normal. However, if proper attention is given to construction methods and shoring installation procedures, we do not see any insurmountable problems in the subterranean construction.

Recommendations for foundation and basement wall design, for excavating and shoring, and for floor slab support are presented in the report. The recommendations should be reviewed at such time as the structural features of the buildings have been definitely established. Also, one additional boring is to be drilled after the site has been cleared; a supplementary report will be submitted after completion of the additional boring and laboratory tests.

Respectfully submitted,

LeROY CRANDALL AND ASSOCIATES

by James L. Van Beveren
James L. Van Beveren, R.C.E. 17785
Project Engineer

by
J. D. Kirkgard, R.C.E. 10441
Executive Vice President

JK-VB/pa
(4 copies submitted)

cc: (2) Carl McLarand Associates, Inc.
Attn: Mr. Carl McLarand

REPORT OF FOUNDATION INVESTIGATION
PROPOSED OFFICE STRUCTURES AND TOWNHOUSES
MUSEUM SQUARE
WILSHIRE BOULEVARD BETWEEN
CURSON STREET AND MASSELIN AVENUE
LOS ANGELES, CALIFORNIA
FOR
OSCHIN AND SNYDER

SCOPE

This report presents the results of a foundation investigation performed for the subject proposed office structures and townhouses. The locations of the proposed buildings and our exploration borings are shown on Plate 1, Plot Plan. Due to existing structures, one of the planned borings could not be drilled at this time. The boring will be drilled after the existing structures have been removed, and the results will be submitted in a supplementary report.

Data obtained from our concurrent investigations for two nearby Museum Square condominium developments (our Job Nos. A-84011 and A-84024) and a prior investigation for the nearby Museum Square parking structure (our Job No. A-82168) were utilized to the extent possible in this investigation.

This investigation was authorized to determine the static physical characteristics of the soils at selected locations and to provide recommendations for foundation design, for excavating, shoring, and walls below grade, and floor slab support for the proposed buildings. The results of the field explorations and laboratory tests, which form the basis of our recommendations, are presented in the attached Appendix. The results of corrosion studies, performed for us by M. J. Schiff & Associates, are also presented in the Appendix.

We are currently performing geologic-seismic studies in order to provide the necessary geotechnical information to meet current Los Angeles City E.I.R. requirements. The results of these studies will be submitted shortly.

Our professional services have been performed using that degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical engineers practicing in this or similar localities. No other warranty, expressed or implied, is made as to the professional advice included in this report. This report has been prepared for Oschin and Snyder and their design consultants to be used solely in the design of the proposed buildings. The report has not been prepared for use by other parties, and may not contain sufficient information for purposes of other parties or other uses.

STRUCTURAL CONSIDERATIONS

Based on present plans, the project will consist of two buildings, which will be five to six stories in height constructed over two to three parking levels. The buildings are shown in plan on Plate 1. The structural features of the proposed buildings have not been definitely established at this time. We understand that the buildings will be of either reinforced concrete or steel frame construction. When considering reinforced concrete construction, column loads will range from 1,000 to 1,700 kips. When considering steel frame construction, column loads will range from 800 to 1,300 kips.

As mentioned above, two to three parking levels are planned. The parking levels will extend some 20 or 30 feet below the existing grade.

SITE CONDITIONS

Part of the site is occupied by existing buildings, with appurtenant paved and planted areas. Another part of the site is paved and used for parking. Elevations of the existing grade at selected locations are shown on Plate 1.

SOIL CONDITIONS

Existing fill soils, two and three feet in thickness, were encountered in 2 of the 18 borings. Deeper fill deposits may be encountered between borings. In any event, the existing fill soils should be removed automatically by the planned excavation.

The natural soils beneath the site consist of clay to depths of 20 to 25 feet, underlain by a thin asphalt sand layer. A second clay layer was encountered between the depths of 30 to 40 feet. A major asphalt sand deposit was encountered below depths of 35 to 45 feet. All of the natural deposits are firm and stiff or dense. Sections illustrating the general soil conditions are presented on Plates 2-A and 2-B, Subsurface Sections; the locations of the sections are indicated on

Plate 1. More detailed information on the soil characteristics is presented on the boring logs in the Appendix. The sections were prepared by interpolation from the logs of the exploration borings, and are therefore accurate only at the borings. However, the sections are believed to describe the subsurface conditions at intermediate locations with sufficient accuracy for the purposes of this report.

Water was measured in three of the borings at depths of 8 to 9 feet below the existing grade. The water level could rise somewhat in the future.

RECOMMENDATIONS

GENERAL

Because of existing structures, one of the planned borings could not be drilled at this time. A supplementary report will be submitted after the boring has been drilled. As previously discussed, the structural features of the proposed buildings have not been definitely established. The following recommendations should be reviewed when the structural features are established and foundation load information is available. This should be done prior to final design of foundations.

The deeper portions of the excavation will extend through the upper clay layer and into the upper, thin layer of asphaltic sand. The natural soils at and below the planned excavated level (20 to 30 feet below the existing grade) are firm and dense, and the proposed buildings may be supported on spread footings. Either individual or combined spread footings or a mat foundation may be used.

Because of the shallow water level and asphalt in the deeper soils, either a hydrostatic design will be required, or a subdrain system will be required beneath the lower subterranean floor to prevent the development of external hydrostatic pressures on the lower floor and walls. Also, gases are being given off by the asphaltic materials, so that either a gas barrier or a suitable venting system will be required to prevent gas from entering the basement. Operation of a permanent subdrain system within the asphaltic materials would present operational difficulties. If a mat-type foundation is used, the foundation could be designed to resist the hydrostatic pressures and to provide an effective barrier to gases, thus eliminating the need for a subdrain system and venting system.

If individual or combined spread footings are used, a heavy reinforced slab between the footings could be designed to resist the hydrostatic pressure similar to the mat foundation. We believe it would be more practical to design the mat foundation or lower slab and walls below grade for hydrostatic pressure and eliminate the need for a subdrain. Accordingly, design data for a subdrain system are not presented in this report. However, if the present design concept is modified and a subdrain system is desired, we could provide the necessary data for design.

Because of the viscous characteristics of the asphalt impregnated sands, subterranean construction extending into these deposits will be more difficult than normal. However if proper attention is given to construction methods and shoring installation procedures, we do not see any insurmountable problems in such subterranean construction.

FOUNDATIONS

General

As discussed above, either spread footings or a mat foundation may be used for support of the proposed buildings. If a subdrain is not provided, the mat foundation or the lower slab on grade will have to be designed to resist the anticipated hydrostatic pressures.

Spread Footings

Spread footings carried into undisturbed natural soils may be designed to impose a net dead plus live load pressure of 5,000 pounds per square foot. A one-third increase may be used for wind or seismic loads. Since the recommended bearing value is a net value, the weight of concrete in the footings may be taken as 50 pounds per cubic foot and the weight of soil backfill may be neglected when determining the downward load on the footings.

Mat Foundation

The mat foundation may be designed for a net dead plus live load pressure up to 5,000 pounds per square foot. A one-third increase may be used for wind or seismic loads. A coefficient of subgrade reaction of 200 pounds per cubic inch may be used in the design of the mat.

Settlement

The maximum settlement of the proposed structures will occur during construction when significant hydrostatic pressures are not developed. We estimate that the maximum settlement of the buildings, if supported on spread footings or a mat in the manner recommended, will be about three-fourths to one inch. Differential settlement between adjacent columns is not expected to exceed one-fourth inch.

Uplift Resistance

In determining the hydrostatic pressure against the bottom slab or mat foundation, the water level may be assumed to be at a depth of about 5 feet below existing site grade.

Lateral Loads

Lateral loads may be resisted by soil friction and by the passive resistance of the soils. A coefficient of friction of 0.4 may be used between footings or the floor slabs and the supporting soils. If the lower floor slabs are waterproofed and designed for hydrostatic pressure, there would be no lateral resistance of the slabs and the lateral resistance of footings due to friction would be reduced as a result of the uplift pressure. The full frictional resistance between the floor slabs and the supporting soils would be developed if a subdrain is installed beneath the floors.

The passive resistance of the natural soils or properly compacted backfill may be assumed to be equal to the pressure developed by a fluid with a density of 300 pounds per cubic foot. If a subdrain is not installed, a reduced pressure of 200 pounds per cubic foot should be used. A one-third increase in the passive value may be used for wind or seismic loads. The frictional resistance and the passive resistance of the soils may be combined without reduction in determining the total lateral resistance.

Observation of Foundation Construction

The asphaltic soils will be greatly disturbed by construction traffic, particularly during hot weather. Care must be taken to avoid excessive disturbance of these soils. As discussed later in this report, a layer of gravel may have to be placed over the asphaltic sands to permit construction traffic. Also, footings should be poured as soon after excavation as possible.

All foundation excavations should be observed by personnel of our firm to verify penetration into undisturbed soils. Footings should be deepened if necessary to extend through disturbed materials. Asphalt seepage must be expected; shallow pools of asphalt will have to be removed before concrete is placed directly on these soils.

To reduce the disturbance of the subgrade soils, we suggest that a waste slab be poured as soon as possible. As discussed later in this report, the waterproofing and gas barrier membrane would then be placed on this waste slab. The waste slab is in addition to any gravel layer that may be required over parts of the area to stabilize the subgrade soils.

EXCAVATION AND SLOPES

Excavation approximately 20 or 30 feet deep will be required for the subterranean levels. Where the necessary space is available, temporary unsurcharged excavations may be sloped back at 3/4:1 (horizontal to vertical) in lieu of using shoring. All applicable requirements of the California Construction and General Industry Safety Orders, the Occupational Safety and Health Act of 1970, and the Construction Safety Act should be met.

Where there is not sufficient space for sloped embankments, shoring will be required. As discussed previously, water was encountered at depths as shallow as eight feet. The excavation will extend below the water level and into asphaltic materials. Because of the asphalt condition, it is our opinion that a well system will not be practical for dewatering during construction. We anticipate that a series of shallow trenches could be used to collect the water and asphalt as the excavation proceeds. This opinion for site dewatering. should be confirmed during excavation when the quantity of inflow can be better evaluated. Under this scheme, dewatering will not be done prior to the commencement of the excavation. Accordingly, water and asphalt should be expected in the drilled holes for soldier piles and tie-back anchors, and special techniques will be necessary to install these shoring elements satisfactorily.

The soils at the bottom of the excavation, especially the asphaltic sands, may be soft and spongy and a layer of gravel may be required to stabilize the subgrade soils and provide support to construction equipment.

SHORING

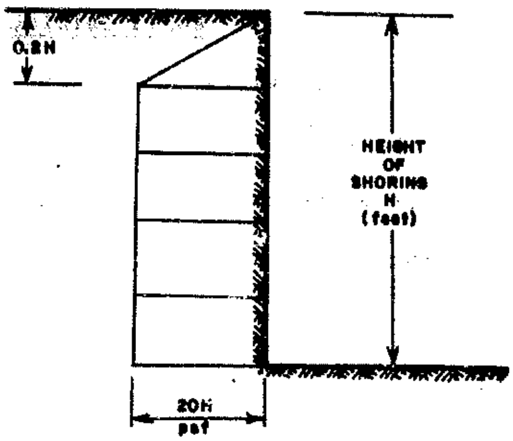
General

Where there is not sufficient space for sloped embankments, shoring will be required. The shoring system could consist of steel soldier piles, placed in drilled holes and backfilled with lean concrete, and tied-back with drilled-in friction anchors. Some difficulty may be experienced in the drilling of the soldier piles and the anchors due to water and asphalt within the soils. Also, caving could be experienced in the drilling of the soldier piles and the anchors through the sand deposits. It may be necessary to utilize casing and/or drilling mud to permit the installation of the soldier piles. The use of special techniques, such as hollow-stem augers, may be found necessary to permit the installation of anchors.

The following information on the design and installation of the shoring is as complete as possible at this time. We can furnish additional required data as the design progresses.

Lateral Pressures

For the design of the tied-back shoring, we recommend the use of a trapezoidal distribution of lateral earth pressure with the maximum pressure equal to 20H in pounds per square foot, where H is the height of the shoring in feet. The recommended pressure distribution, for the case where the grade is level behind the shoring, is illustrated below. (If a combination of sloped embankment and shoring is to be used, the pressure would be greater and must be determined for each combination.)



As stated previously, the upper soils are generally cohesive and are underlain by asphaltic sands. The predominant asphaltic sand deposit is some 30 to 40 feet deep. The active pressure within the cohesive soils is less than that within the underlying asphaltic sand. Accordingly, if the proposed excavation were to extend deeper than about 30 feet, a higher intensity of pressure may be required. We could provide the necessary data if needed.

In addition to the recommended earth pressure, the upper ten feet of shoring adjacent to the streets should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pounds per square foot surcharge behind the shoring due to normal street traffic. If the traffic is kept back at

Missing page 12 of the Report of Foundation Investigation, which detailed lateral pressure recommendations.

A records request regarding the missing page of the report was submitted to the City of Los Angeles Department of Building and Safety (LADBS) by Jelisa Thomas Adams on behalf of the Wilshire Courtyard Redevelopment Project. According to LADBS, page 12 of the report was not scanned to microfilm; therefore, the page could not be acquired.

Anchors

Tie-back anchors may be used to resist lateral loads. Data for design of tie-back anchors and additional shoring design data will be provided when the depth of excavation is established.

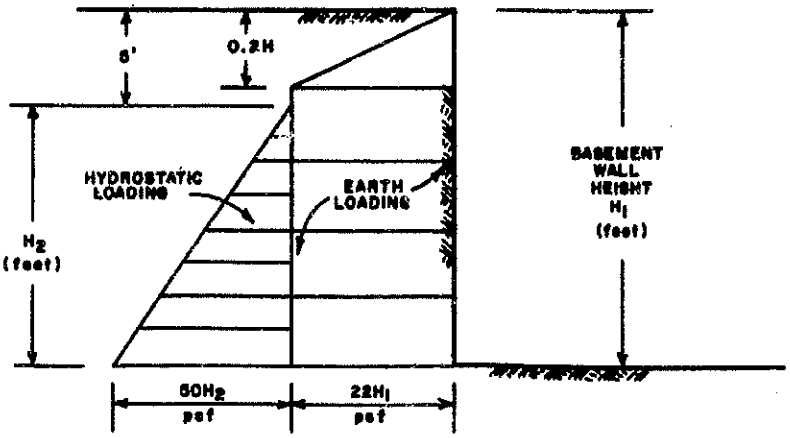
WALLS BELOW GRADE

General

Water was encountered in the borings above the planned basement level. If a subdrain system will not be provided beneath the building, the building walls below grade should be waterproofed and designed to resist the anticipated hydrostatic pressures. (Data for design of a subdrain system can be provided if desired.) Gas emissions from the asphaltic materials must also be prevented from entering the building.

Lateral Pressures

We recommend that the basement walls be designed to resist a trapezoidal distribution of lateral earth pressures plus the hydrostatic pressure. The lateral earth pressure on the permanent basement walls will be similar to that recommended for design of temporary shoring, except the maximum lateral pressure will be 22H in pounds per square foot where H is the height of the basement wall in feet. The recommended pressure distribution is shown below. If a subdrain will not be installed, the walls should be designed to resist the hydrostatic pressure in addition to the earth loading. The effective hydrostatic pressure is also shown below.



Backfill

If any backfill is required, it should be placed in layers and compacted to at least 90% of the maximum density obtainable by the ASTM Designation D1557-70 method of compaction. Flooding should not be permitted. Proper compaction of the backfill will be necessary to reduce settlement of the backfill and consequent settlement of any overlying walks and slabs.

The clay soils will be difficult to compact in confined areas and are expansive. Accordingly, we suggest that the clay soils not be used for backfill and that backfill soils consist of relatively non-expansive soils. The backfill soils should contain sufficient fines so as to be relatively impermeable when compacted.

FLOOR SLAB SUPPORT

Walls below a depth of five feet and the lower floor slabs will have to be waterproofed and designed for the appropriate hydrostatic pressure. As previously discussed, subdrain systems could be installed beneath the buildings to avoid the build-up of hydrostatic pressure on the lower floors and basement walls. If desired to give further consideration to a subdrain system, we could provide the necessary design data.

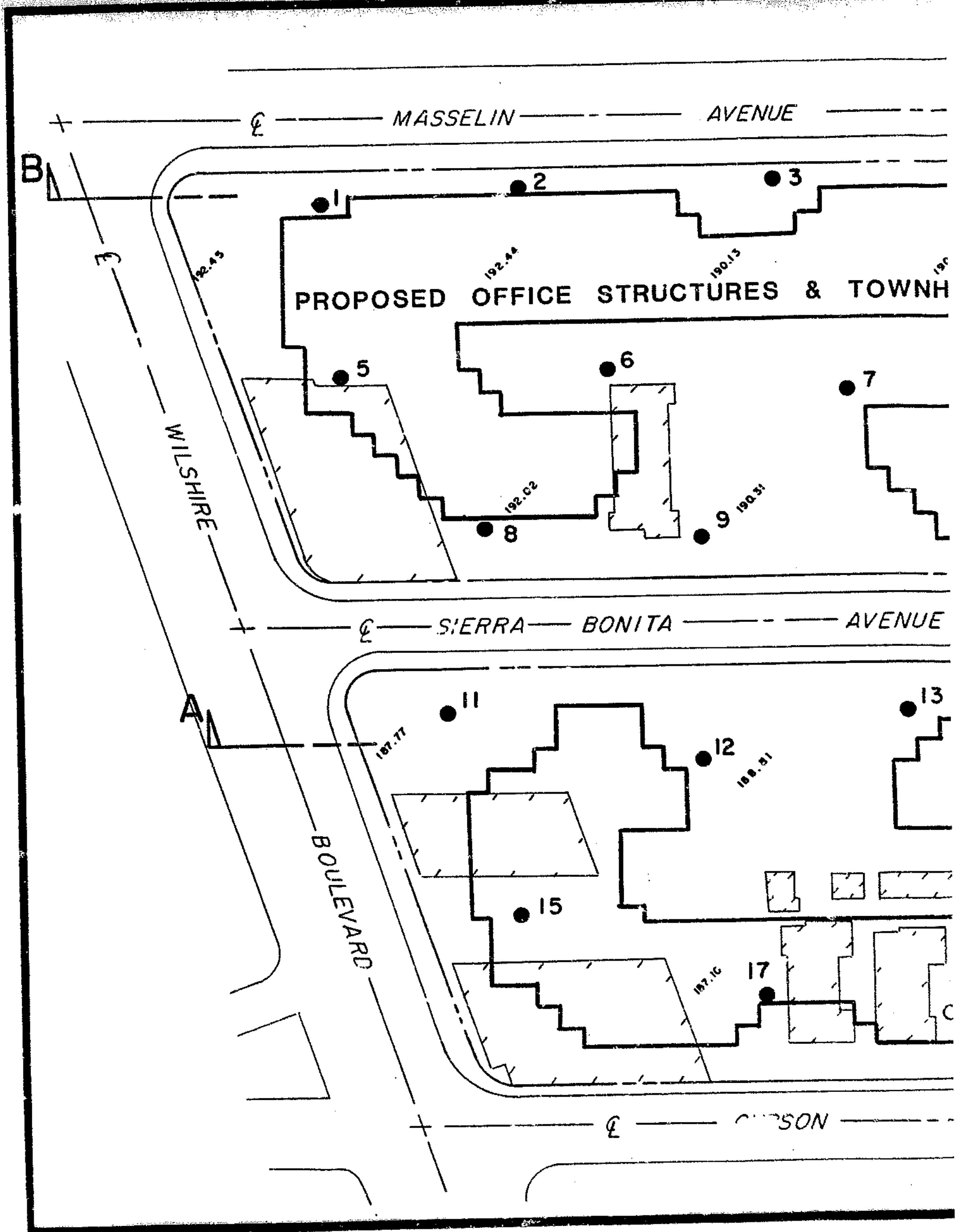
As discussed previously, gases are being given off by the underlying asphaltic soils. A suitable impermeable membrane should be placed beneath the lower floors to prevent gas from entering the buildings. A membrane could be incorporated within the slabs on grade to act as a barrier to gases as well as the necessary waterproofing.

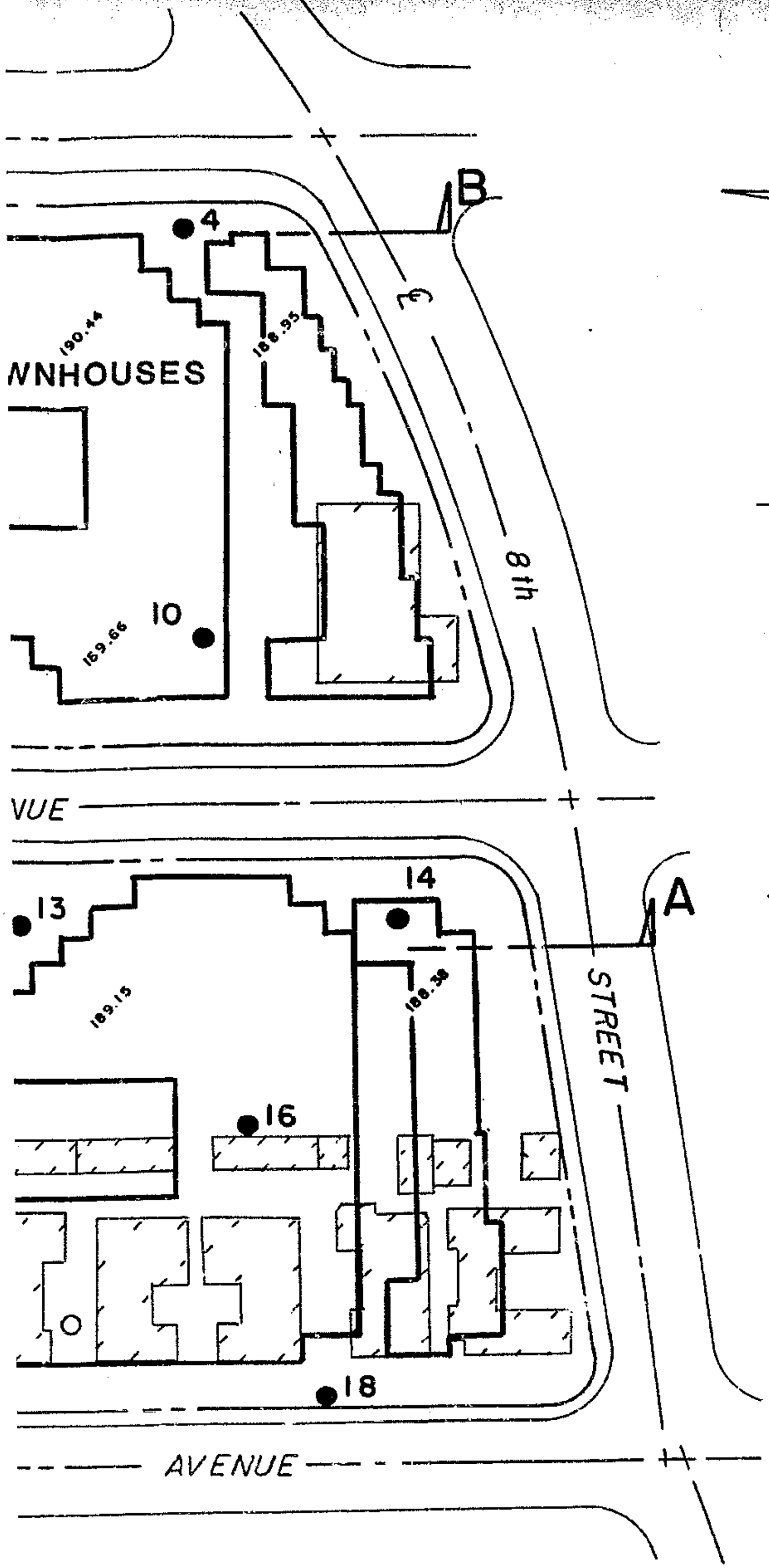
The undisturbed natural soils will offer adequate support to the lower level floor slabs. Any natural soils loosened or over-excavated should be properly compacted; compaction to at least 90% is recommended. As previously stated, a waterproof membrane is recommended beneath the floors.

-oOo-

The following Plates and Appendix are attached and complete this report:

Plate 1 ------------- Plot Plan
Plates 2-A and 2-B--- Subsurface Sections
Appendix ------------ Explorations and Laboratory Tests


℄ MASSELIN AVENUE
B
1
2
3
192.45
192.44
190.13
PROPOSED OFFICE STRUCTURES & TOWNH
5
6
7
WILSHIRE
192.02
8
9
190.51
℄ SIERRA BONITA AVENUE
A
11
12
13
187.77
188.51
BOULEVARD
15
17
187.10
℄ CARSON



REFERENCES:
SURVEY (DATED 1-8-81) BY
COOPER-SHERMAN ENGINEERING CO.
PLAN (UNDATED) BY
CARL MCLARAND ASSOCIATES, INC.

KEY:

18
● BORING LOCATION
○ BORING TO BE DRILLED
BORING NUMBER

188.38 EXISTING GRADE

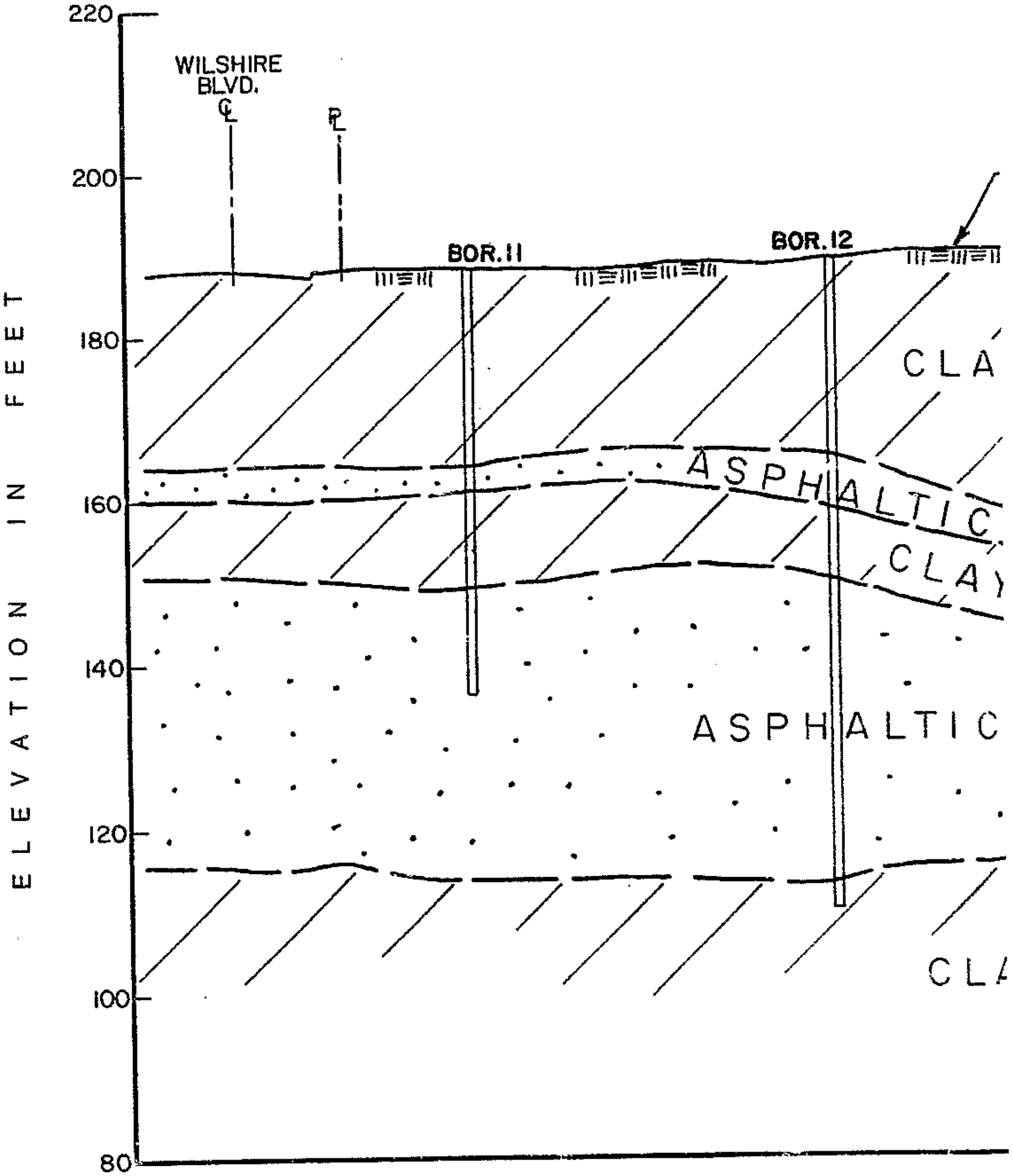
▨ EXISTING STRUCTURE

PLOT PLAN

SCALE 1" = 80'

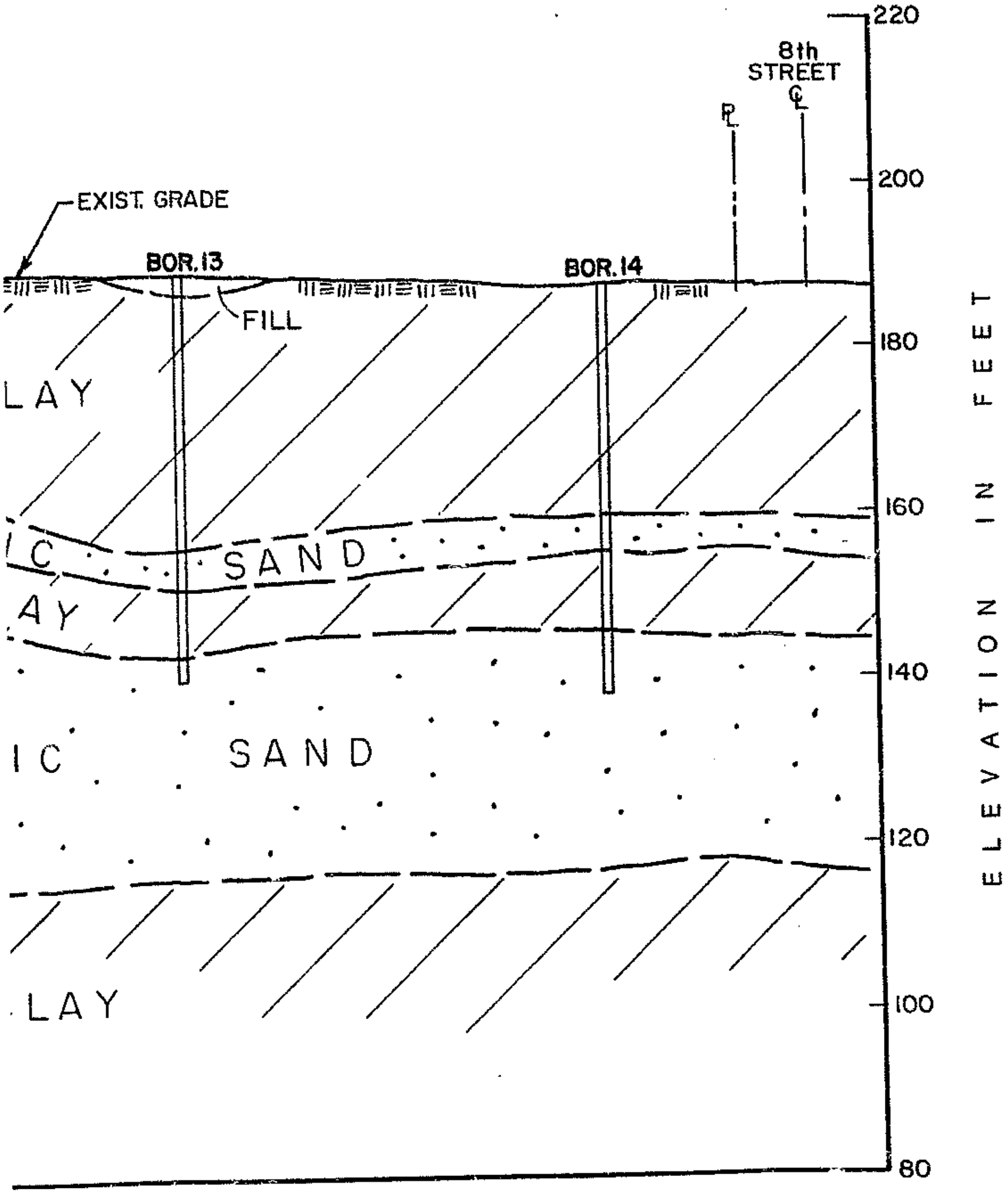
LeROY CRANDALL AND ASSOCIATES

PLATE I



NOTE:

SECTIONS BASED ON SOIL CONDITIONS AT BORING LOCATIONS. SOIL CONDITIONS BETWEEN BORINGS HAVE BEEN INTERPOLATED AND ARE NOT NECESSARILY ACCURATE.

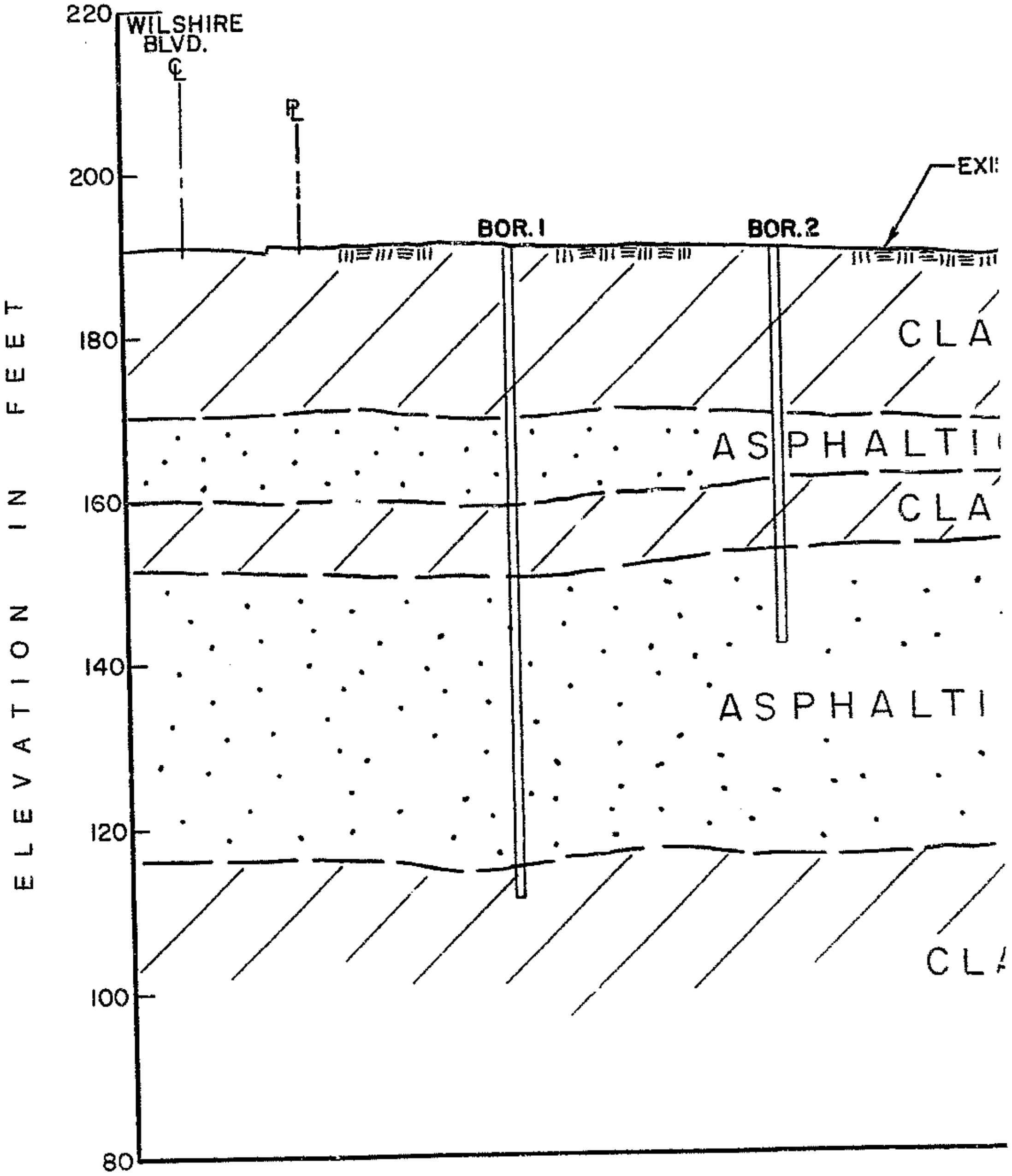


SUBSURFACE SECTION A-A

SCALES: HORIZ. 1" = 80'
VERT. 1" = 20'

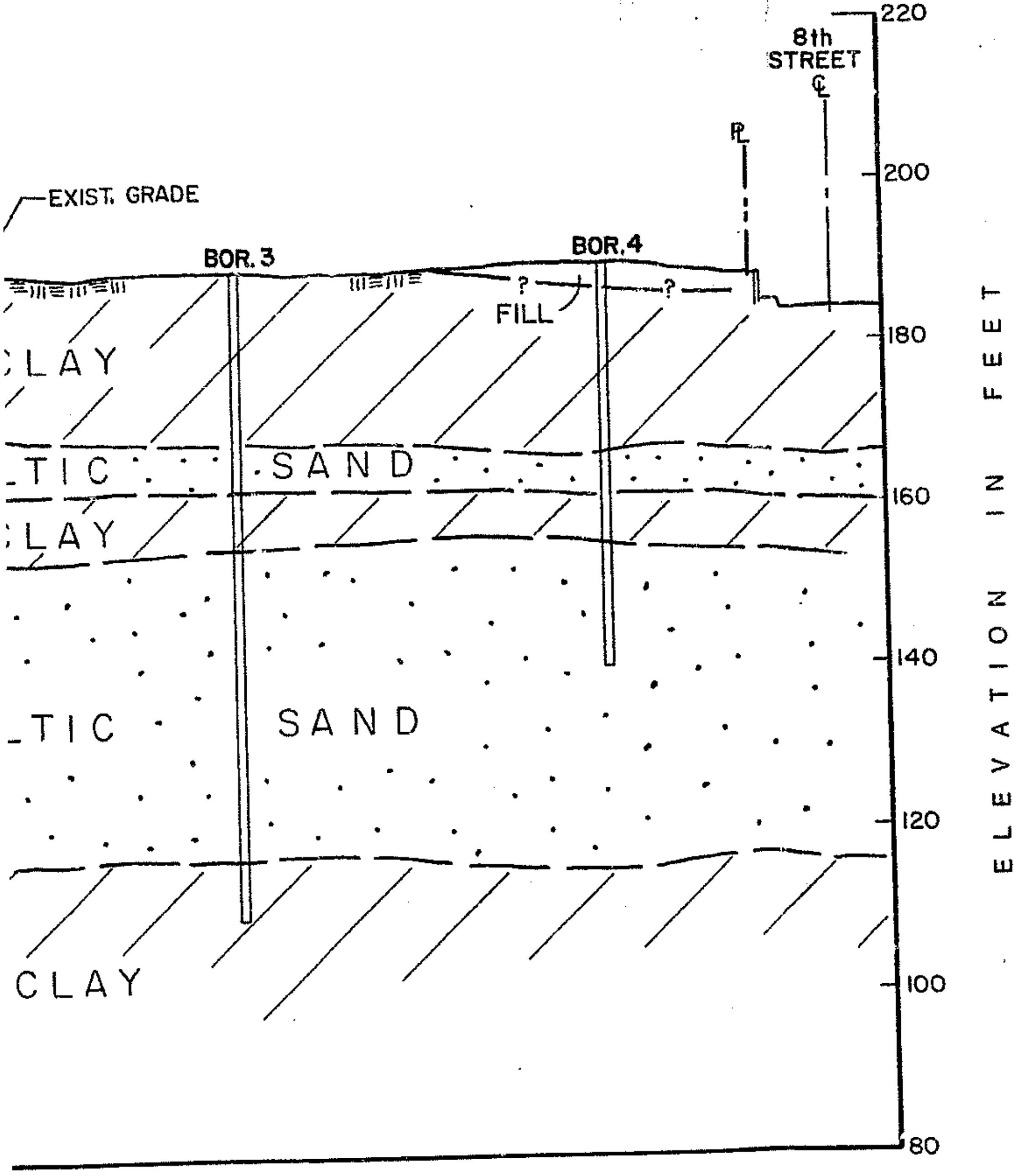
LeROY CRANDALL AND ASSOCIATES

PLATE 2-A



NOTE:

SECTIONS BASED ON SOIL CONDITIONS AT BORING LOCATIONS. SOIL CONDITIONS BETWEEN BORINGS HAVE BEEN INTERPOLATED AND ARE NOT NECESSARILY ACCURATE.



SUBSURFACE SECTION B-B

SCALES: HORIZ. 1" = 80'
VERT. 1" = 20'

LeROY CRAN[illegible] AND ASSOCIATES

PLATE 2-B

APPENDIX

EXPLORATIONS

The soil conditions beneath the site were explored by drilling 18 borings at the locations shown on Plate 1. In addition, data were available from our concurrent and prior nearby investigations.

The borings were drilled to depths of 49 to 81 feet below the existing grade using rotary wash-type drilling equipment. Drilling mud was used with the rotary wash equipment to prevent caving. The mud was removed from the borings following completion of the drilling. To form a seal and to help prevent asphalt and gases from reaching the ground surface at the boring locations, the borings were filled with pea gravel overlain by a concrete plug.

The soils encountered were logged by our field technician, and undisturbed samples were obtained for laboratory inspection and testing. The logs of the borings are presented on Plates A-1.1 through A-1.18; the depths at which undisturbed samples were obtained are indicated to the left of the boring logs. The energy required to drive the sampler twelve inches is indicated on the logs. The soils are classified in accordance with the Unified Soil Classification System described on Plate A-2.

LABORATORY TESTS

The percent of volatile liquids and the in-situ dry density of the undisturbed samples were determined in the laboratory. (The in-situ dry density includes the combined weight of solids and non-volatile liquids.) The samples were oven-dried at a temperature of 105°C to remove moisture and other volatile liquids. For those samples not containing asphalt, the weight lost during drying would be water only, and the volatile liquids and dry density thus determined would be the moisture content and true dry density of the soil (solids). However, oven drying of the asphaltic sands does not remove all of the asphalt from the sample. Accordingly, the oven-dried asphaltic soils will include the weight of solids and any non-volatile liquids, and the dry density thus determined would not indicate the dry density of the solids only. To determine the asphalt content of the asphaltic sands, and thus determine the true dry density, three selected samples were washed with a solvent to dissolve all of the asphalt. The results of the tests are shown to the left of the boring logs.

Direct shear tests were performed on selected undisturbed samples to determine the strength of the soils. The tests were performed at field and increased moisture contents and at various surcharge pressures. The yield-point values determined from the direct shear tests are presented on Plates A-3.1 and A-3.2, Direct Shear Test Data.

Confined consolidation tests were performed on eight undisturbed samples to determine the compressibility of the soils. The samples were tested at field moisture content. The results of the tests are presented on Plates A-4.1 through A-4.4, Consolidation Test Data.

Corrosion studies were performed for us on six samples by M. J. Schiff & Associates. The results are presented on Plates A-5.1 through A-5.3.

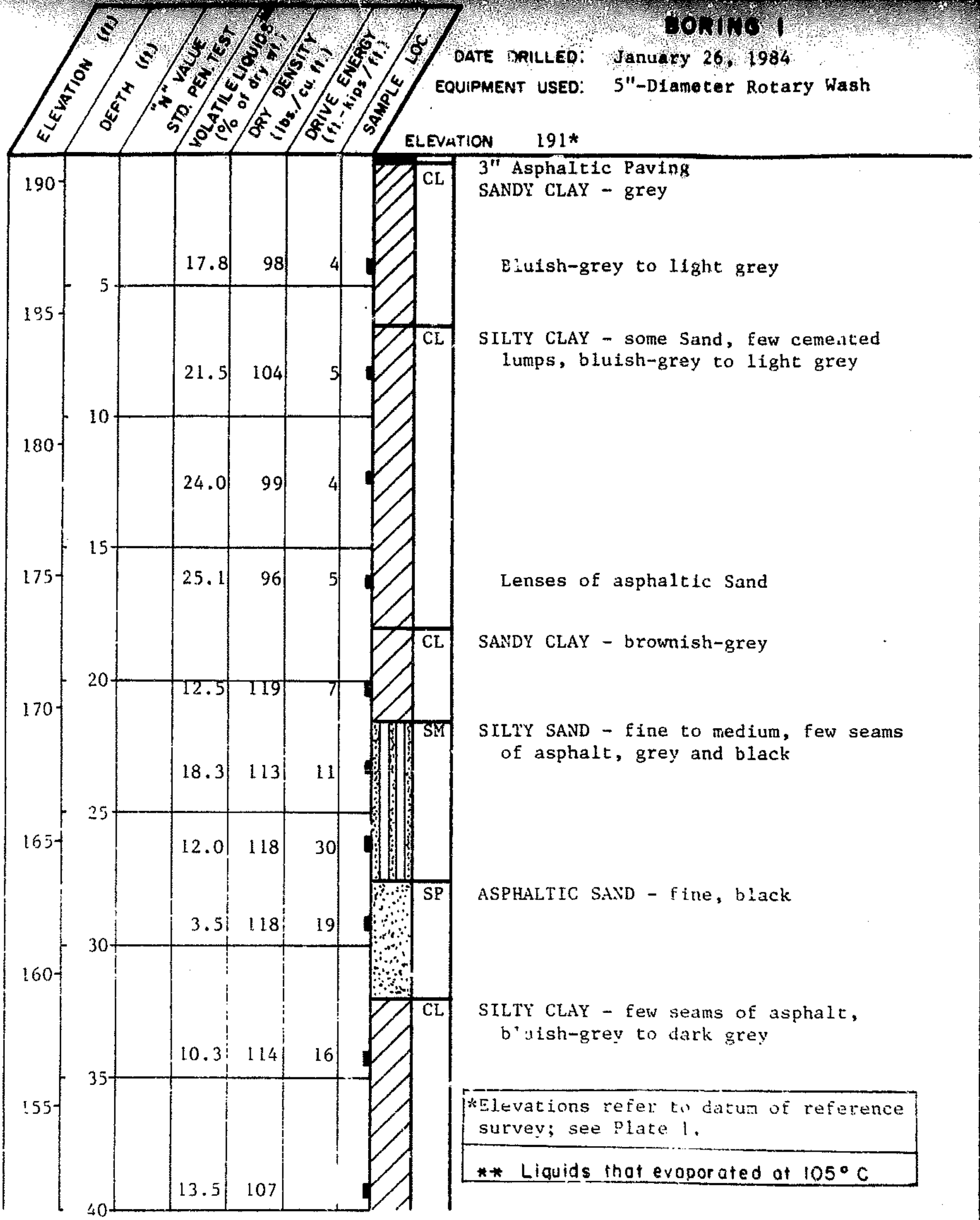
-oOo-

BORING 1

DATE DRILLED: January 26, 1984

EQUIPMENT USED: 5"-Diameter Rotary Wash

ELEVATION 191*



VOLATILE LIQUIDS** (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)
17.8	98	4
21.5	104	5
24.0	99	4
25.1	96	5
12.5	119	7
18.3	113	11
12.0	118	30
3.5	118	19
10.3	114	16
13.5	107	

3" Asphaltic Paving

CL SANDY CLAY - grey

Bluish-grey to light grey

CL SILTY CLAY - some Sand, few cemented lumps, bluish-grey to light grey

Lenses of asphaltic Sand

CL SANDY CLAY - brownish-grey

SM SILTY SAND - fine to medium, few seams of asphalt, grey and black

SP ASPHALTIC SAND - fine, black

CL SILTY CLAY - few seams of asphalt, bluish-grey to dark grey

*Elevations refer to datum of reference survey; see Plate 1.

** Liquids that evaporated at 105° C

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

(CONTINUED ON FOLLOWING PLATE)

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

BORING 1 (CONTINUED

DATE DRILLED: January 26, 1984
EQUIPMENT USED: 5"-Diameter Rotary Wash

ELEVATION (ft.)	DEPTH (ft.)	"N" VALUE STD. PEN. TEST	VOLATILE LIQUIDS (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.		
150								
							SP	ASPHALTIC SAND - fine, black
			6.5	120	15	■		
	45							
145								
			4.0	109	23	■		
	50							
140								
			6.2	106	34	■		
	55							
135								
			5.0	109	23	■		
	60							
130								
			5.6	105	19	■		
	65							
125								
	70							
120								
			6.8	99	23	■		
	75							
115								
							CL	SILTY CLAY - few seams of asphalt, dark grey
			22.0	91	17	■		
	80							

NOTE: Drilling mud used in drilling process.
Water level not established.

LOG OF BORING

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

Form 124 JOB A-84921 DATE 2/13/84 DR. JOHN W.P. CHKD

LeROY CRANDALL AND ASSOCIATES

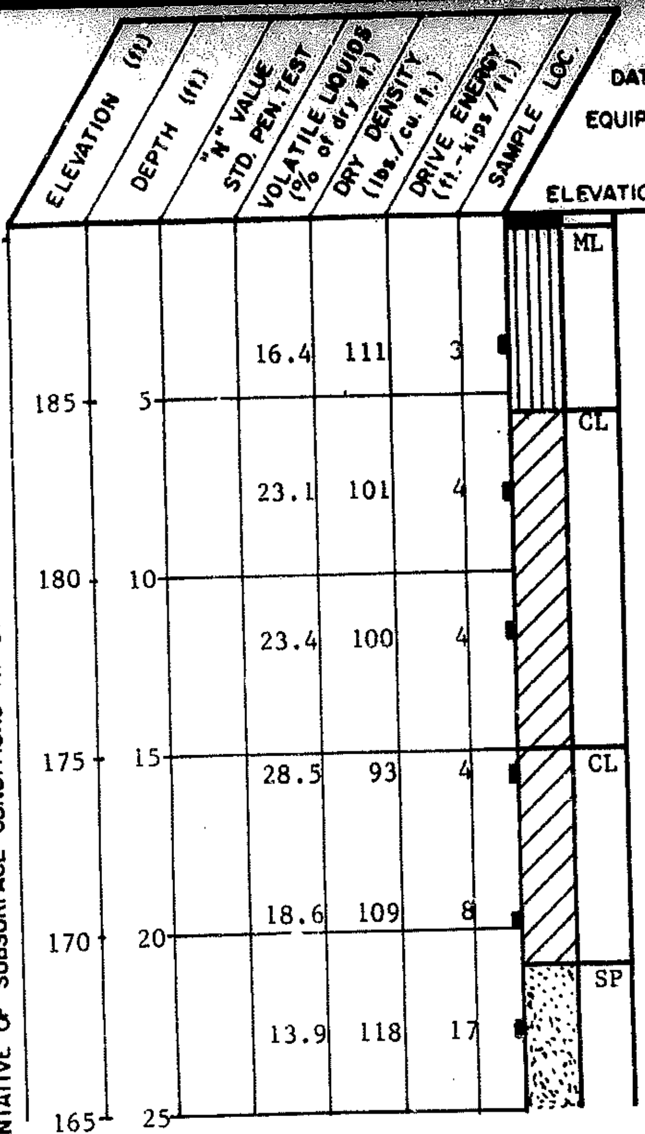
PLATE A-1.1b

BORING 2

DATE DRILLED: January 28, 1984

EQUIPMENT USED: 5"-Diameter Rotary Wash

ELEVATION 190



ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD. PEN. TEST	VOLATILE LIQUIDS (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.		
								3" Asphaltic Paving
							ML	CLAYEY SILT - some Sand, light brown
			16.4	111	3			
185	5						CL	SILTY CLAY - grey
			23.1	101	4			
180	10							
			23.4	100	4			
175	15		28.5	93	4		CL	SANDY CLAY - grey
								Few cemented lumps, brownish-grey
			18.6	109	8			
170	20						SP	ASPHALTIC SAND - fine, some Silt, black
			13.9	118	17			
165	25							

(CONTINUED ON FOLLOWING PLATE)

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

Form 125 JOB # 84021 DATE 2/13/84 DR. JOHN W.P. tk CHKD

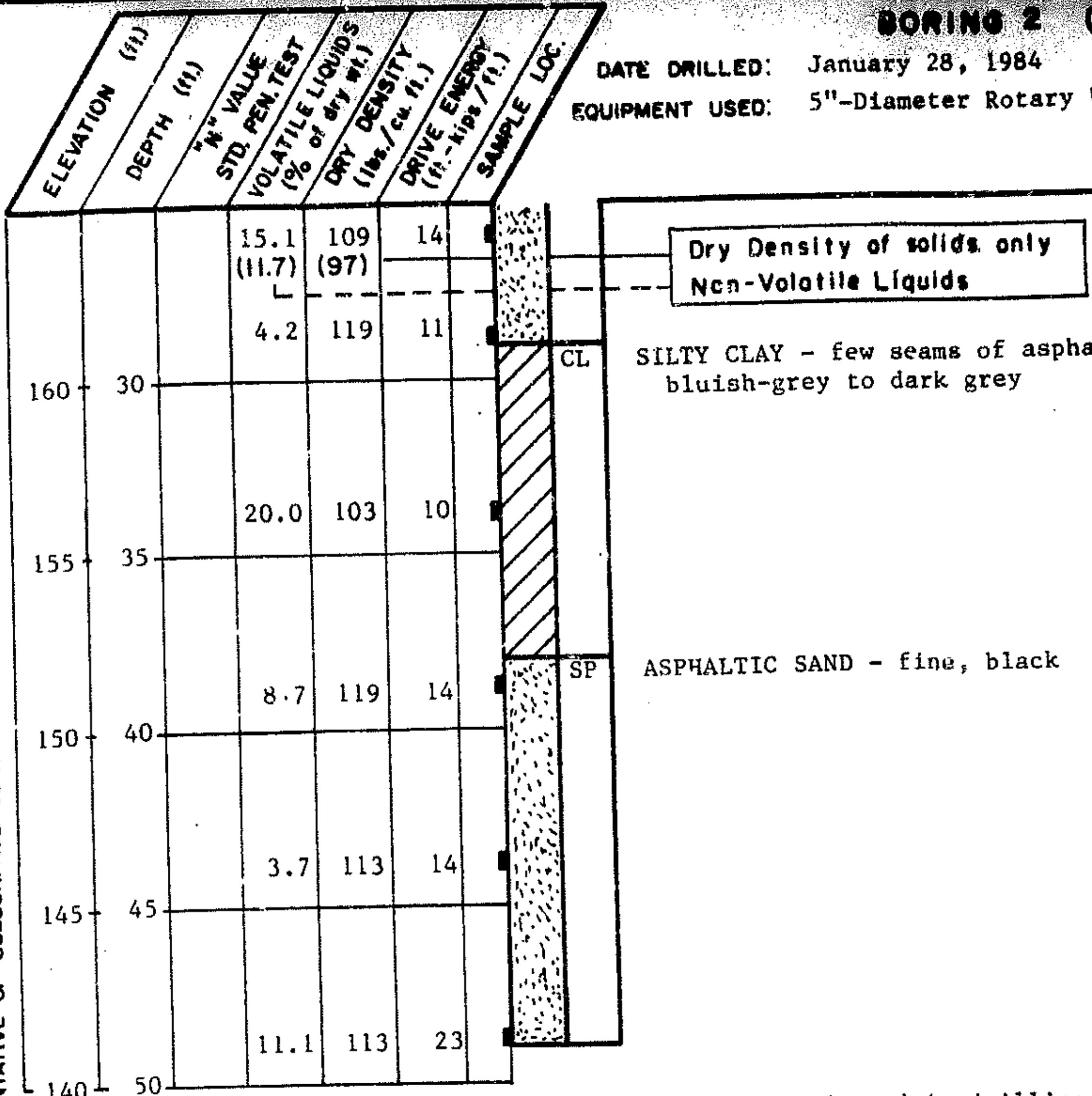
LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

PLATE A-1.2a

BORING 2 (CONTINUED)

DATE DRILLED: January 28, 1984
EQUIPMENT USED: 5"-Diameter Rotary Wash



ELEVATION (ft.)	DEPTH (ft.)	"N" VALUE STD. PEN. TEST	VOLATILE LIQUIDS (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.
			15.1 (11.7)	109 (97)	14	
			4.2	119	11	
			20.0	103	10	
			8.7	119	14	
			3.7	113	14	
			11.1	113	23	

CL SILTY CLAY - few seams of asphalt, bluish-grey to dark grey

SP ASPHALTIC SAND - fine, black

NOTE: Drilling mud used in drilling process. Water level not established.

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

Form 124 JOB A-84021 DATE 2/13/84 DR JOHN W.P. CHKD

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

PLATE A-1.2b

BORING 3

DATE DRILLED: January 30, 1984

EQUIPMENT USED: 5"-Diameter Rotary Wash

ELEVATION 189

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD. PEN. TEST	VOLATILE LIQUIDS (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.
			20.3	105	2	
185	5					
			27.3	98	3	
180	10		39.3	80	2	
			36.6	84	4	
175	15					
			18.0	109	10	
170	20					
			10.5	114	10	
165	25		13.0	112	23	
			17.6	104	8	
160	30					
			16.4	104	11	
155	35					
			6.4	123	13	
150	40					

3" Asphaltic Paving

CL SANDY CLAY - grey

CL SILTY CLAY - bluish-grey

CL SANDY CLAY - few cemented lumps, grey

Brownish-grey

Seams of asphalt

SP ASPHALTIC SAND - fine, black

CL SILTY CLAY - few seams of asphalt, dark grey

Few thin layers of asphaltic Sand

SP ASPHALTIC SAND - fine, black

Some gravel

(CONTINUED ON FOLLOWING PLATE)

LOG OF BORING

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

JOB A-84021 DATE 2/13/84 DR JOHN W.P. CHKD

Form 123

LeROY CRANDALL AND ASSOCIATES

PLATE A-1.3

BORING 3 (CONTINUED)

DATE DRILLED: January 30, 1984
EQUIPMENT USED: 5"-Diameter Rotary Wash

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD. PEN. TEST	VOLATILE LIQUIDS (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.
			5.0	118	23	■
145	45					
			3.9	108	23	■
140	50					
			3.5	111	51	■
135	55					
			6.6	111	44	■
130	60					
			3.0	120	34	■
125	65					
			6.2	105	19	■
120	70					
			10.2	102	19	■
115	75					
			17.8	99	16	■
110	80					
			24.6	89	16	■

SILTY CLAY - few seams of asphalt, dark grey

NOTE: Drilling mud used in drilling process. Water level not established.

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

PLATE A-1.3b

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

FORM 124 JOB A-84[illegible] DATE 2/13/84 DR JOHN W.P. CHKD

BORING 4

DATE DRILLED: January 30, 1984

EQUIPMENT USED: 5"-Diameter Rotary Wash

ELEVATION 190

ELEVATION (ft.)	DEPTH (ft.)	"N" VALUE STD. PEN. TEST	VOLATILE LIQUIDS (% of dry wt.)	DRY DENSITY (lbs./cu.ft.)	DRIVE ENERGY (ft-kips/ft.)	SAMPLE LOC.		
								3" Asphaltic Paving
			17.9	107	1	■	CL SP	FILL - CLAY and SAND - few pieces of brick, brown and grey
							CL	SANDY CLAY - few cemented lumps, bluish-grey
185	5							
			18.2	106	3	■		
							CL	SILTY CLAY - bluish-grey
180	10		28.9	94	3	■		
							CL	SANDY CLAY - few cemented lumps, grey
			23.1	102	7	■		
175	15							
			21.6	106	8	■		Lenses of asphaltic Sand
170	20							
								Seams of asphalt, lenses of asphaltic Sand
			34.3	85	6	■		
165	25						SP	ASPHALTIC SAND - fine, black
			6.2	124	14	■		
160	30						CL	SILTY CLAY - few seams of asphalt, some asphaltic Sand, cemented lumps, bluish-grey

(CONTINUED ON FOLLOWING PLATE)

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

Form 123 JOB A-84021 DATE 2/13/84 DR JOHN W.P. CHKD

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

BORING 4 (CONTINUED)

DATE DRILLED: January 30, 1984

EQUIPMENT USED: 5"-Diameter Rotary Wash

ELEVATION (ft.)	DEPTH (ft.)	"N" VALUE STD. PEN. TEST	VOLATILE LIQUIDS (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.		
			20.3	104	8	■		
			23.4	96	5	■		
155	35						SP	ASPHALTIC SAND - fine, black
			6.5	121	13	■		
			5.9	117	16	■		
150	40							
			2.7	117	26	■		
145	45							
			2.3	117	27	■		
140	50							

NOTE: Drilling mud used in drilling process. Water level not established.

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

PLATE A-1.4b

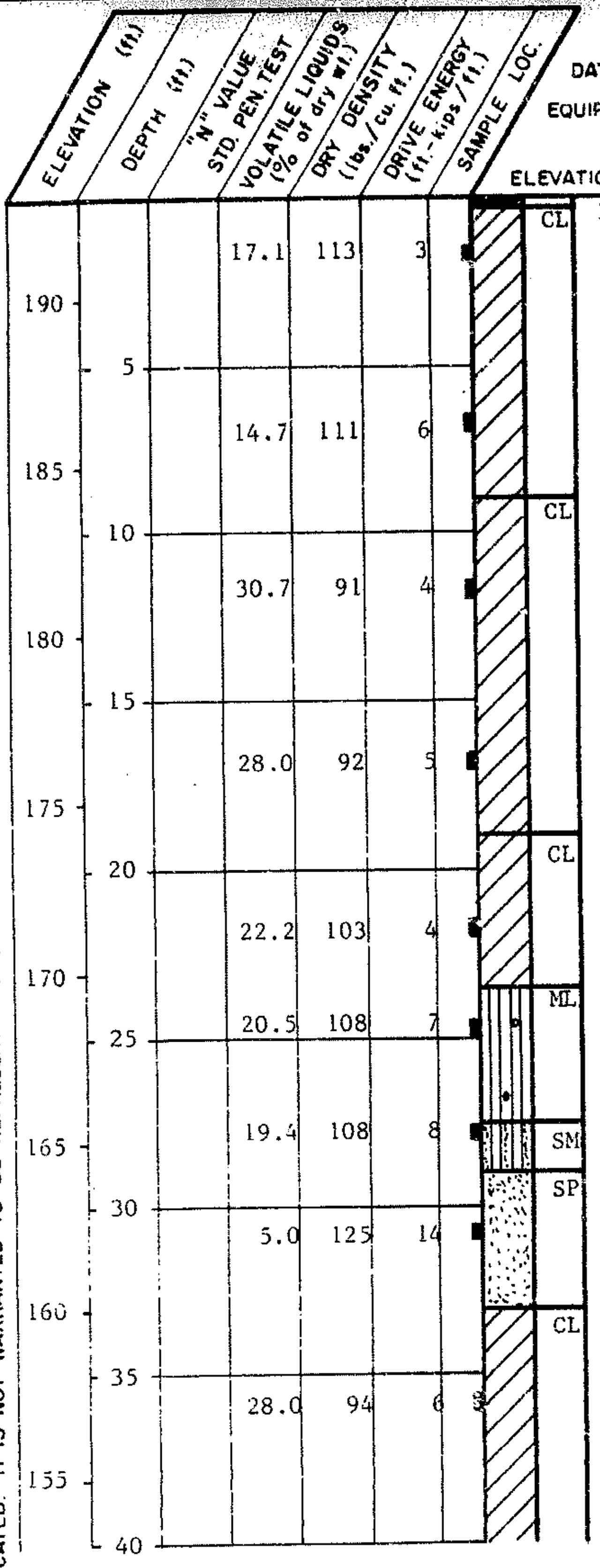
FORM 124 JOB A-84 DATE 2/13/84 DR. W.P. CHKD

BORING 5

DATE DRILLED: January 26 & 27, 1984

EQUIPMENT USED: 5"-Diameter Rotary Wash

ELEVATION 193



"N" VALUE STD. PEN. TEST	VOLATILE LIQUIDS (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)
	17.1	113	3
	14.7	111	6
	30.7	91	4
	28.0	92	5
	22.2	103	4
	20.5	108	7
	19.4	108	8
	5.0	125	14
	28.0	94	6

3" Asphaltic Paving

CL SANDY CLAY - dark grey

CL SILTY CLAY - bluish-grey

Few cemented lumps, bluish-grey to light grey

Few seams of asphalt, some asphaltic Sand, brownish-grey

CL SANDY CLAY - grey

ML SANDY SILT - slightly Clayey, few gravel, bluish-grey

Seams of asphalt

SM SILTY SAND - fine, grey to dark grey

SP ASPHALTIC SAND - fine, black

CL SILTY CLAY - bluish-grey

Seams of asphalt

Many seams of asphalt

(CONTINUED ON FOLLOWING PLATE)

LOG OF BORING

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

JOB B4 DATE 3/84 DR JOHN W.P. CHKD

LeROY CRANDALL AND ASSOCIATES

BORING 5 (CONTINUED)

DATE DRILLED: January 26 & 27, 1984

EQUIPMENT USED: 5"-Diameter Rotary Wash

ELEVATION (ft.)	DEPTH (ft.)	"N" VALUE STD. PEN. TEST	VOLATILE LIQUIDS (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.
150			6.4	114	11	■
145	45		18.5	106	6	■
140	50		3.6 (17.5)	103 (89)	19	■
135	55		2.7	117	21	■
130	60		7.1	104	17	■
125	65		3.2	120	27	■
120	70		9.5	91	23	■
115	75		9.8	98	19	■
	80		22.8	89	12	■

SP ASPHALTIC SAND - fine, black

NOTE: Drilling mud used in drilling process. Water level not established.

CL SILTY CLAY - asphalt stained, dark greyish-brown

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

Form 124 JOB A-84093 DATE 2/13/84 DR. JOHN W.P. DC CHKD

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

PLATE A-1.5b

BORING 6

DATE DRILLED: January 27 & 28, 1984

EQUIPMENT USED: 5"-Diameter Rotary Wash

ELEVATION 193

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD. PEN. TEST	VOLATILE LIQUIDS (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft-kips/ft.)	SAMPLE LOC		
								3" Asphaltic Paving
							CL	SILTY CLAY - grey
			18.4	108	1	▮		
190								
	5						CL	SANDY CLAY - light brown
			12.6	115	4	▮		
185								
							CL	SILTY CLAY - some fine Sand, light grey
	10							
			25.3	100	4	▮		
180								
	15							
								Greyish-brown
			19.2	105	13	▮		
175								
	20							Few seams of asphalt
			22.8	102	6	▮		
170							SP	ASPHALTIC SAND - fine, black
			8.4	117	10	▮		
	25							
			8.8	103	34	▮		
165								Some Clay
	30		7.4	118	9	▮		
160								
	35		11.8	108	11	▮		
155								
	40							

(CONTINUED ON FOLLOWING PLATE)

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

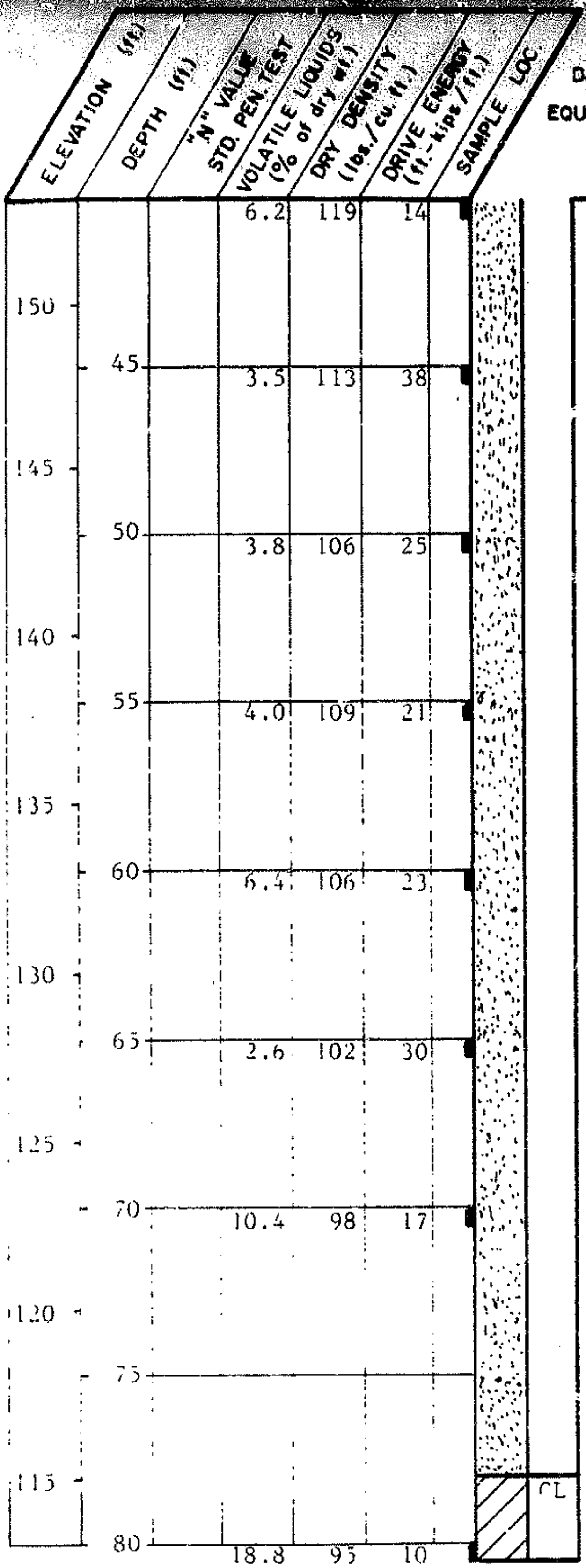
NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

Form 123 JOB A-84021 DATE 2/13/84 DR JOHN W.P. ЕС CHKD

BORING 6 (CONTINUED)

DATE DRILLED: January 27 & 28, 1984

EQUIPMENT USED: 5"-Diameter Rotary Wash



DEPTH (ft.)	VOLATILE LIQUIDS (% of dry wt.)	DRY DENSITY (lbs./cu.ft.)	DRIVE ENERGY (ft.-kips/ft.)
	6.2	119	14
45	3.5	113	38
50	3.8	106	25
55	4.0	109	21
60	6.4	106	23
65	2.6	102	30
70	10.4	98	17
80	18.8	95	10

CL SILTY CLAY - dark gray

NOTE: Drilling mud used in drilling process. Water level not established.

NOTE THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES

JOB 84121 DATE 2/13/84 DR JOHN W.P. CHKD

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

PLATE A-1.6b

BORING 7

DATE DRILLED: January 25, 1984

EQUIPMENT USED: 5"-Diameter Rotary Wash

ELEVATION 191

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD. PEN. TEST	VOLATILE LIQUIDS (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft. kips/ft.)	SAMPLE LOC.		
190							CL	3" Asphaltic Paving SANDY CLAY - grey
			24.6	101	2	■	CL	SILTY CLAY - greyish-black
	5							
185								
			33.1	90	1	■		Some fine Sand
	10							
180								
			19.0	102	6	■		Few seams of asphalt, black
	15							
175								
			20.3	100	11	■		Many seams of asphalt, few cemented lumps, black and grey
	20							
170								
			12.0	117	8	■		
	25							

(CONTINUED ON FOLLOWING PLATE)

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

FORM 123 JOB A-84021 DATE 2/13/84 DR JOHN W.P. CHKD

LOG OF BORING

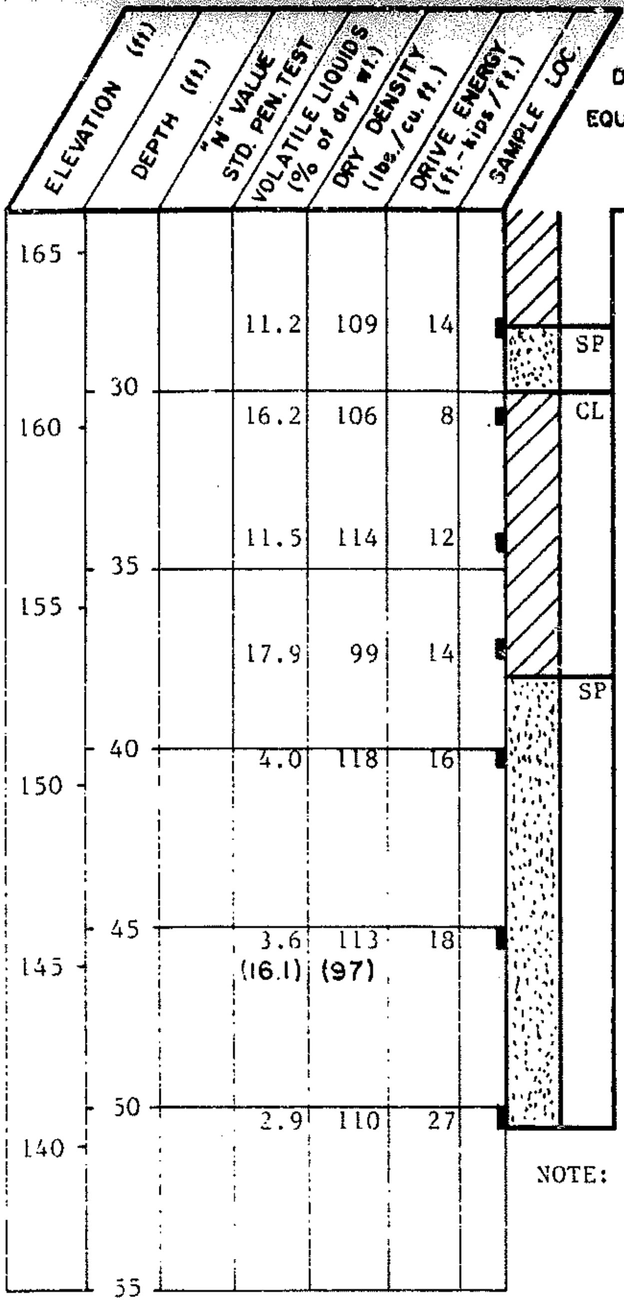
LeROY CRANDALL AND ASSOCIATES

PLATE A-1.7

BORING 7 (CONTINUED)

DATE DRILLED: January 25, 1984

EQUIPMENT USED: 5"-Diameter Rotary Wash



ELEVATION (ft.)	DEPTH (ft.)	"N" VALUE STD. PEN. TEST	VOLATILE LIQUIDS (% of dry wt.)	DRY DENSITY (lbs./cu.ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.
			11.2	109	14	■
			16.2	106	8	■
			11.5	114	12	■
			17.9	99	14	■
			4.0	118	16	■
			3.6 (16.1)	113 (97)	18	■
			2.9	110	27	■

SP ASPHALTIC SAND - fine, black

CL SILTY CLAY - few seams of asphalt, dark grey to bluish-grey

Many seams of asphalt

SP ASPHALTIC SAND - fine, black

NOTE: Drilling mud used in drilling process.
Water level not established.

NOTE THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

Form 124 JOB A-84021 DATE 2/13/84 DR JOHN YR 14 W.P. 89 CHKD

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

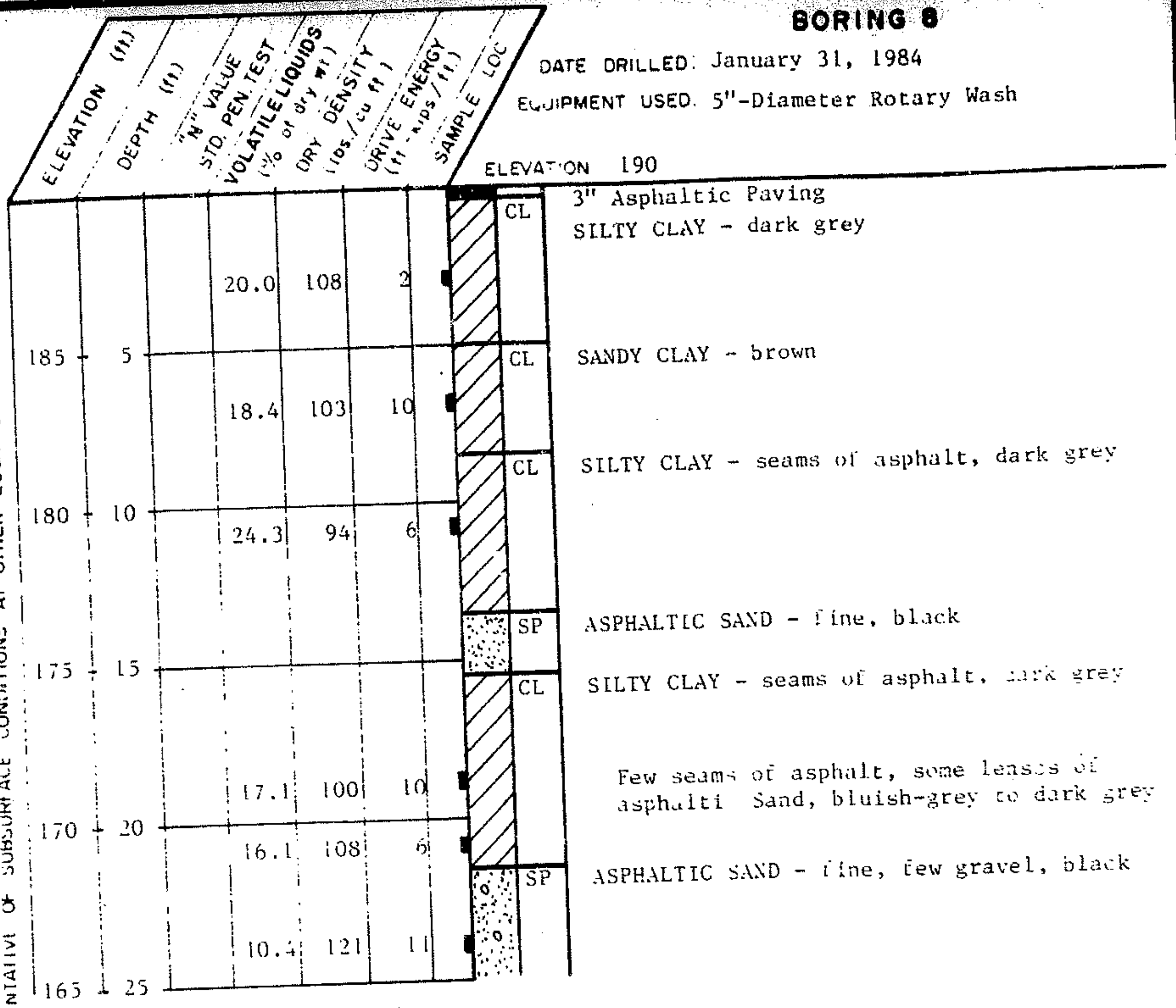
PLATE A-1.7

BORING 8

DATE DRILLED: January 31, 1984

EQUIPMENT USED: 5"-Diameter Rotary Wash

ELEVATION 190



ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD. PEN TEST	VOLATILE LIQUIDS (% of dry wt)	DRY DENSITY (lbs./cu ft)	DRIVE ENERGY (ft-kips/ft)	SAMPLE LOC		
							CL	3" Asphaltic Paving SILTY CLAY - dark grey
			20.0	108	2	▪		
185	5						CL	SANDY CLAY - brown
			18.4	103	10	▪		
							CL	SILTY CLAY - seams of asphalt, dark grey
180	10							
			24.3	94	6	▪		
							SP	ASPHALTIC SAND - fine, black
175	15						CL	SILTY CLAY - seams of asphalt, dark grey
			17.1	100	10	▪		Few seams of asphalt, some lenses of asphalti Sand, bluish-grey to dark grey
170	20							
			16.1	108	6	▪		
							SP	ASPHALTIC SAND - fine, few gravel, black
			10.4	121	11	▪		
165	25							

(CONTINUED ON FOLLOWING PLATE)

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES

Form 123 JOB # 84921 DATE 2/13/84 DR JOHN CK W.P. CARD

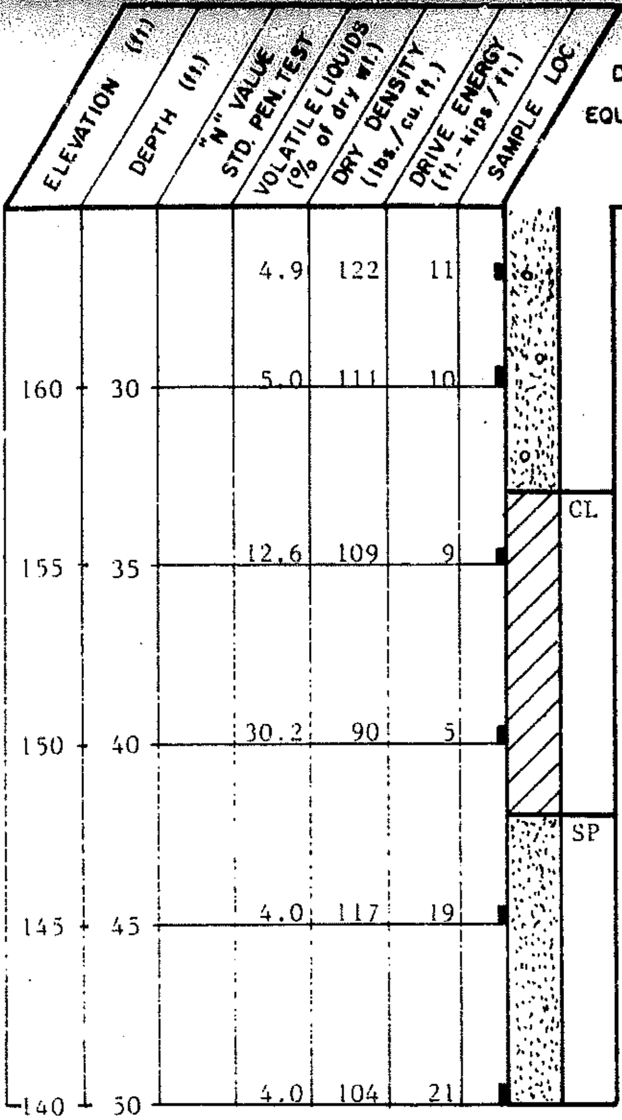
LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

PLATE A-1.8a

BORING 8 (CONTINUED)

DATE DRILLED: January 31, 1984
EQUIPMENT USED: 5"-Diameter Rotary Wash



ELEVATION (ft.)	DEPTH (ft.)	"N" VALUE STD. PEN. TEST	VOLATILE LIQUIDS (% of dry wt.)	DRY DENSITY (lbs./cu.ft.)	DRIVE ENERGY (ft.-kips/ft.)
			4.9	122	11
160	30		5.0	111	10
155	35		12.6	109	9
150	40		30.2	90	5
145	45		4.0	117	19
140	50		4.0	104	21

CL SILTY CLAY - few seams of asphalt, dark gray

Bluish-gray

SP ASPHALTIC SAND - fine, black

NOTE: Drilling mud used in drilling process.
Water level not established.

NOTE THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

Form 124 JOB A-84021 DATE 2/13/84 DR JOHN TR WP CHKD

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

PLATE A-1.8b

FORM 123 JOB A-84061 DATE 2/13/84 DR JOHN W.P. CHKD

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES

BORING 9

DATE DRILLED: January 25, 1984
EQUIPMENT USED: 5"-Diameter Rotary Wash

ELEVATION 190

ELEVATION (ft.)	DEPTH (ft.)	"N" VALUE STD. PEN. TEST	VOLATILE LIQUIDS (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft-kips/ft.)	SAMPLE LOC.		
								3" Asphaltic Paving SILTY CLAY - dark grey
			19.0	112	2	■		
185	5							
			27.7	95	3	■		Slightly Sandy, grey
180	10							
			28.7	90	3	■		
175	15							
			25.6	97	8	■		Seams of asphalt
170	20							
			9.2	113	10	■	SP	ASPHALTIC SAND - fine, some Clay, black
165	25							

(CONTINUED ON FOLLOWING PLATE)

LOG OF BORING

LeROY [illegible] ALL AND ASSOCIATES

FORM 124 JOB 84 DATE 2/13/84 DR JOHN W.P. CHKD

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

BORING 9 (CONTINUED)

DATE DRILLED: January 25, 1984

EQUIPMENT USED: 5"-Diameter Rotary Wash

ELEVATION (ft.)	DEPTH (ft.)	"N" VALUE STD. PEN. TEST	VOLATILE LIQUIDS (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.		
			12.5	108	17	■		
			6.7	113	11	■		
160	30						CL	SILTY CLAY - some fine Sand, seams of asphalt, grey
			18.4	103	4	■		
155	35							Some layers of asphaltic Sand, few seams of asphalt
			22.1	100	11	■		
							SP	ASPHALTIC SAND - fine, black
150	40							
			5.2	118	15	■		
145	45							
			4.1	112	16	■		
140	50							
			3.0	119	21	■		
135	55							

NOTE: Drilling mud used in drilling process. Water level not established.

LOG OF BORING

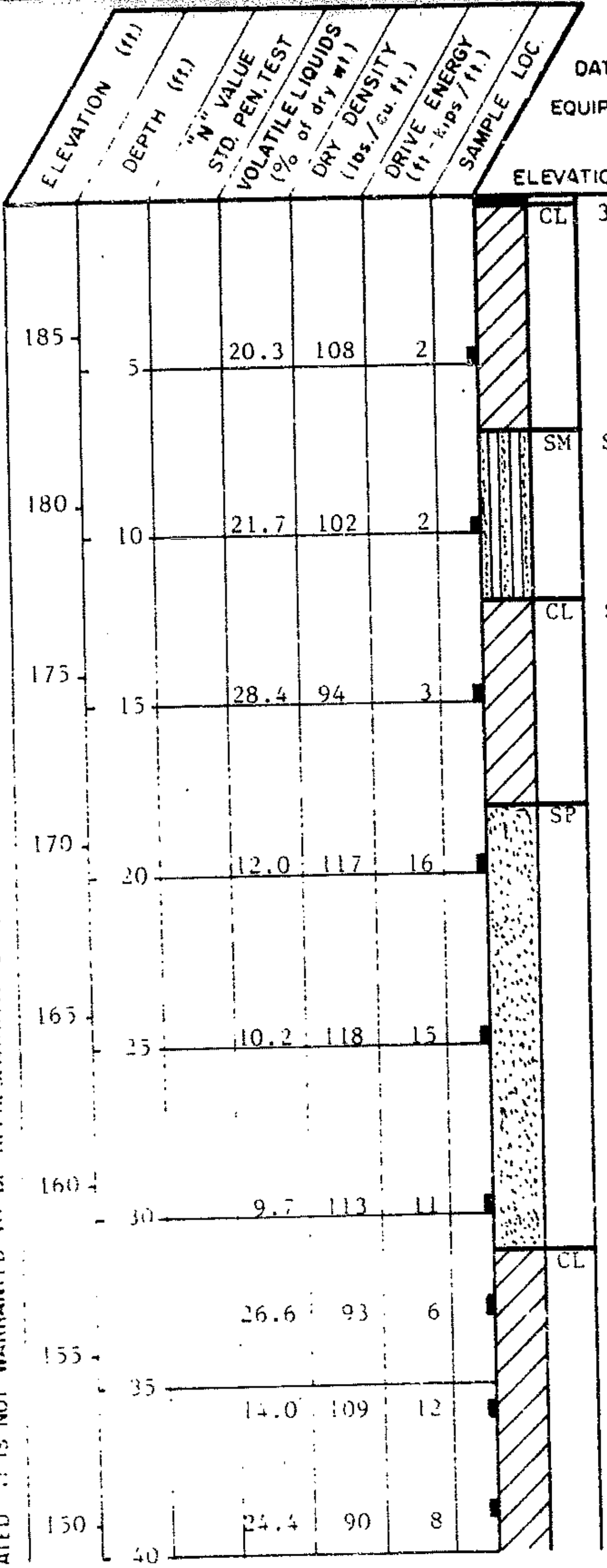
LeROY CRANDALL AND ASSOCIATES

PLATE A-1.9

BORING 10

DATE DRILLED: January 24 & 25, 1984
EQUIPMENT USED: 5"-Diameter Rotary Wash

ELEVATION 189



	5		20.3	108	2
	10		21.7	102	2
	15		28.4	94	3
	20		12.0	117	16
	25		10.2	118	15
	30		9.7	113	11
			26.6	93	6
	35		14.0	109	12
			24.4	90	8
	40				

CL 3" Asphaltic Paving
SANDY CLAY - bluish-grey

SM SILTY SAND - fine, bluish-grey

CL SILTY CLAY - bluish-grey

SP ASPHALTIC SAND - fine, black

CL SILTY CLAY - greyish-brown

Some asphalt

Large amount of asphalt

(CONTINUED ON FOLLOWING PLATE)

NOTE THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

PLATE A-1.10

FORM 123 JOB A-84022 DATE 2/13/84 DR JOHN W.P. CHKD

BORING 10 (CONTINUED)

DATE DRILLED: January 24 & 25, 1984
EQUIPMENT USED: 5"-Diameter Rotary Wash

ELEVATION (ft.)	DEPTH (ft.)	"N" VALUE STD. PEN. TEST	VOLATILE LIQUIDS (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.
			5.6	119	23	■
145	45					
			4.0	119	23	■
140	50					
			3.7	113	34	■
135	55					
			3.4	111	23	■
130	60					
			3.2	117	38	■
125	65					
			4.4	109	19	■
120	70					
			11.3	102	18	■
115	75					
			12.1	108	21	■
110	80					

SP ASPHALTIC SAND - fine, black

NOTE: Drilling mud used in drilling process. Mud removed at completion of drilling. Water level measured at 9½' on 2/2/84.

LOG OF BORING

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

Form 124 JOB A-84071 DATE 2/15/84 DR JOHN CHKD WP

LeROY CRANDALL AND ASSOCIATES

PLATE A-1.10b

BORING 11

DATE DRILLED: February 1, 1984

EQUIPMENT USED: 5"-Diameter Rotary Wash

ELEVATION 188

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD. PEN. TEST	VOLATILE LIQUIDS (% of dry wt.)	DRY DENSITY (lbs./cu.ft.)	DRIVE ENERGY (ft-kips/ft.)	SAMPLE LOC		
							CL	3" Asphaltic Paving SILTY CLAY - black
185			22.0	93	2	■		
	5							
180			22.4	101	5	■		Some Sand, few seams of asphalt, light bluish-grey
	10							
			20.2	102	7	■		Some asphaltic Sand, few seams of asphalt, brownish-grey
175								
	15		8.0	118	21	■		
170								
			14.1	103	14	■		
	20							
			17.0	101	8	■		
165								
							SP	ASPHALTIC SAND - fine, seams of asphalt, black
	25		5.7	120	8	■		
							CL	SILTY CLAY - seams of asphalt, bluish-grey
160			17.0	108	8	■		
	30							

(CONTINUED ON FOLLOWING PLATE)

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

Form 123 JOB # 84021 DATE 2/13/84 DR JOHN YK W.P. CHKD

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

BORING 11 (CONTINUED)

DATE DRILLED: February 1, 1984

EQUIPMENT USED: 5"-Diameter Rotary Wash

ELEVATION (ft.)	DEPTH (ft.)	"N" VALUE STD. PEN. TEST	VOLATILE LIQUIDS (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.		
			16.5	99	9	▪		Some lenses of asphaltic Sand
155								
	35							
			32.7	86	4	▪		
150								
	40						SP	ASPHALTIC SAND - fine, black
			3.0	117	15	▪		
145								Some gravel
	45							
			3.1	128	21	▪		
140								
	50							Some seashells
			8.3 (18.6)	106 (89)	13	▪		
135								
	55							

NOTE: Drilling mud used in drilling process.
Water level not established.

NOTE THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

FORM 124 JOB 84 DATE 2/13/84 DR JOHN CH'KD

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

PLATE A-1.11b

BORING 12

DATE DRILLED: February 1, 1984

EQUIPMENT USED: 5"-Diameter Rotary Wash

ELEVATION 189

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD. PEN. TEST	VOLATILE LIQUIDS (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft-kips/ft.)	SAMPLE LOC.
185			20.8	104	2	■
	5					
			15.5	115	5	■
180						
	10					
			30.3	92	3	■
175						
	15					
			30.1	92	6	■
170						
			15.0	109	38	■
	20					
			20.4	99	16	■
165						
	25					
			6.3	119	16	■
160			8.6	99	9	■
	30					
			13.8	108	8	■
155						
	35					
150			13.2	114	13	■
	40					

CL 3" Asphaltic Paving
SILTY CLAY - dark grey

CL SANDY CLAY - bluish-grey

CL SILTY CLAY - bluish-grey

Light grey

CL SANDY CLAY - seams of asphalt, some lenses of asphaltic Sand, light brownish-grey

Slightly cemented

SP ASPHALTIC SAND - fine, black

CL SILTY CLAY - seams of asphalt, some lenses of asphaltic Sand, bluish-grey

SP ASPHALTIC SAND - fine, black

(CONTINUED ON FOLLOWING PLATE)

LOG OF BORING

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES

123 JOB 84 DATE 2/13/83 DR JOHN W.P. CHKD

1 9 0 2 1 5

LeROY CRANDALL AND ASSOCIATES

BORING 12 (CONTINUED)

DATE DRILLED: February 1, 1984

EQUIPMENT USED: 5"-Diameter Rotary Wash

ELEVATION (ft.)	DEPTH (ft.)	"N" VALUE STD. PEN. TEST	VOLATILE LIQUIDS (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.		
145			2.7	112	19	■		
	45							
140			4.1	110	23	■		
	50							
135			2.2	113	23	■		
	55							
130			6.5	114	23	■		
	60							
125			2.4	111	25	■		
	65							
120			4.9	110	23	■		
	70							
115			8.1	103	19	■		
	75							
							CL	SILTY CLAY - dark grey
110			26.5	87	23	■		
	80							

NOTE: Drilling mud used in drilling process.
Water level not established.

NOTE THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

Form 124 JOB 84 DATE 3/ DR JOHN W.P CHKD

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

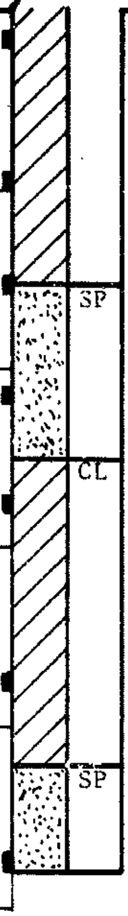
PLATE A-1.1

BORING 13 (CONTINUED)

DATE DRILLED: February 2, 1984

EQUIPMENT USED: 5"-Diameter Rotary Wash

ELEVATION (ft.)	DEPTH (ft.)	"N" VALUE STD. PEN. TEST	VOLATILE LIQUIDS (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.
			25.3	97	8	■
160						
			21.8	101	10	■
	30					
			25.7	92	13	■
155						
	35					
			4.9	115	8	■
150			25.7	90	10	■
	40					
145			41.2	77	8	■
	45					
140			5.3 (24.0)	112 (90)	23	■
	50					



Many seams of asphalt

SP ASPHALTIC SAND - fine, black

CL SILTY CLAY - seams of asphalt, bluish-grey

SP ASPHALTIC SAND - fine, black

NOTE: Drilling mud used in drilling process. Mud removed after completion of drilling. Water level measured at a depth of 9½' on 2/3/84.

NOTE THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES

Form 124 JOB A-84021 DATE 2/13/84 DR JOHN W.P. CHKD

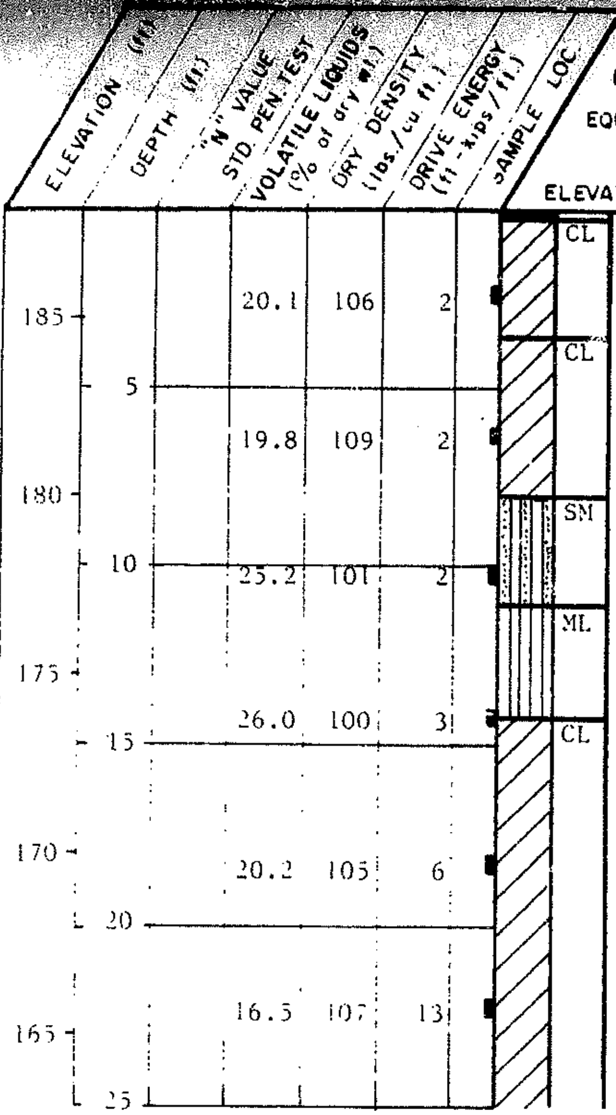
LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

PLATE A-1.13

BORING 14

DATE DRILLED: January 31, 1984
EQUIPMENT USED: 5"-Diameter Rotary Wash



3" Asphaltic Paving
SILTY CLAY - dark grey

SANDY CLAY - bluish-grey

SILTY SAND - fine, bluish-grey

SANDY SILT - bluish-grey

SILTY CLAY - bluish-grey

Some lenses of asphaltic Sand, light grey

(CONTINUED ON FOLLOWING PLATE)

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES

Form 123 JOB A-84021 DATE 2/13/84 DR JOHN CHKD

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

BORING 15 (CONTINUED)

DATE DRILLED: February 6 & 7, 1984
EQUIPMENT USED: 5"-Diameter Rotary Wash

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD. PEN. TEST	VOLATILE LIQUIDS (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC		
145								Some gravel
			3.1	113	25	■		
	45							
140								
			5.2	109	8	■		
	50							
135								
	55							
130								
			4.1	114	19	■		
	60							
125								
			7.3	110	21	■		Some seashells, few gravel
	65							
120								
			9.5 (11.2)	106 (96)	21	■		
	70							
							CL	SILTY CLAY - few seams of asphalt, dark grey
115								
			19.1	89	16	■		
	75							
110								
			21.2	90	14	■		
	80							

NOTE: Drilling mud used in drilling process. Water level not established.

NOTE THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES

Form 124 JOB A-84021 DATE 2/13/84 DR JOHN WP CHKD

LOG OF BORING

LeROY CR ..L AND ASSOCIATES

PLATE A-1..

BORING 15

DATE DRILLED: February 6 & 7, 1984
EQUIPMENT USED: 5"-Diameter Rotary Wash

ELEVATION 138

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD. PEN. TEST	VOLATILE LIQUIDS (% of dry wt.)	DRY DENSITY (lbs./cu ft.)	DRIVE ENERGY (ft-kips/ft.)	SAMPLE LOC.		
							CL	3" Asphaltic Paving SILTY CLAY - dark grey
185			21.8	92	1	■		
	5							
								Light greyish-brown
180			24.5	103	2	■		
	10							
			12.7	108	8	■		Seams of asphalt, brown
175								
	15							
			11.2	105	12	■		Many seams of asphalt, some asphaltic Sand, black and grey
170								
			13.1	106	6	■		
	20							
							SP	ASPHALTIC SAND - fine, black
165			4.7	116	8	■		
	25							
			7.4 (22.4)	100 (82)	6	■		
160								
			9.4	116	4	■	CL	SILTY CLAY - blue-grey
	30							
155								Dark grey
			30.3	83	5	■		
	35							
150							SP	ASPHALTIC SAND - fine, black
			4.1	108	16	■		
	40							

(CONTINUED ON FOLLOWING PLATE)

LOG OF BORING

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES

Form 123 JOB A-84021 DATE 2/13/84 DR JOHN W.P. CHKD

LeROY CRANDALL AND ASSOCIATES

Form 123 JOB A-84021 DATE 2/13/84 DR JOHN W.P. CHKD

NOTE THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES

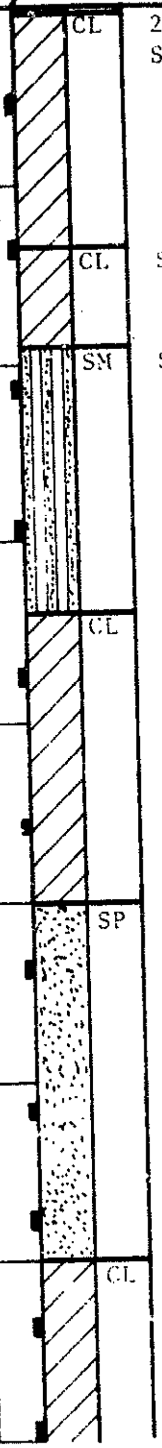
BORING [illegible]

DATE DRILLED: February 2, 1984

EQUIPMENT USED: 5"-Diameter Rotary Wash

ELEVATION 189

ELEVATION (ft.)	DEPTH (ft.)	"N" VALUE STD. PEN. TEST	VOLATILE LIQUIDS (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)
			16.8	110	2
185					
	5				
			12.9	114	3
180					
	10				
			23.2	107	2
175					
			27.6	100	3
	15				
170			30.6	90	4
	20				
			18.6	105	7
165					
	25				
			11.5	113	34
160					
	30				
			11.0	110	8
155			5.3	110	11
	35				
			29.6	86	16
150			20.3	101	19
	40				



CL 2" Asphaltic Paving
SILTY CLAY - dark grey

CL SANDY CLAY - bluish-grey

SM SILTY SAND - fine, bluish-grey

CL SILTY CLAY - dark grey

Seams of asphalt

SP ASPHALTIC SAND - fine, black

Some layers of Clay

CL SILTY CLAY - seams of asphalt, some lenses of asphaltic Sand

Many seams of asphalt

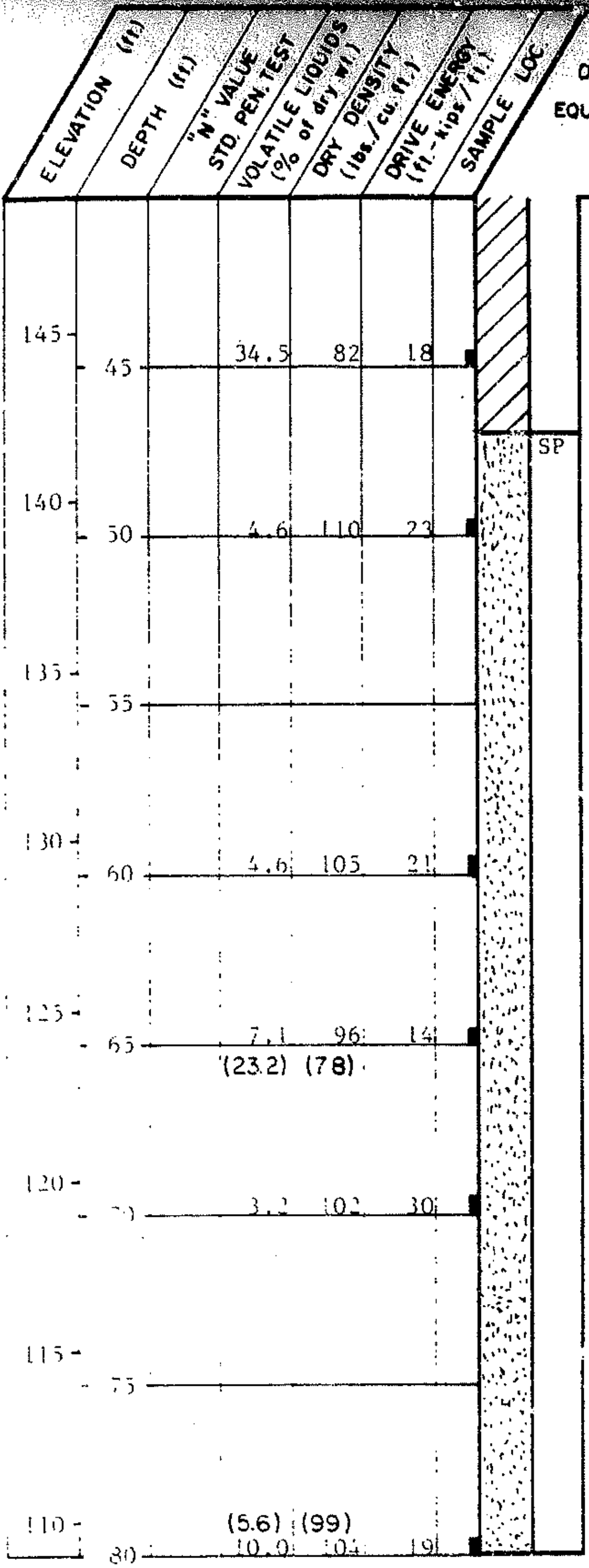
(CONTINUED ON FOLLOWING PLATE)

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

BORING 16 (CONTINUED)

DATE DRILLED: February 2, 1984
EQUIPMENT USED: 5"-Diameter Rotary Wash



DEPTH (ft)	"N" VALUE STD. PEN. TEST	VOLATILE LIQUIDS (% of dry wt.)	DRY DENSITY (lbs./cu.ft.)	DRIVE ENERGY (ft.-kips/ft.)
45		34.5	82	18
50		4.6	110	23
55				
60		4.6	105	21
65		7.1 (23.2)	96 (78)	14
70		3.2	102	30
75				
80		(5.6) 10.0	(99) 104	19

Many seams of asphalt

SP ASPHALTIC SAND - fine, black

NOTE: Drilling mud used in drilling process. Water level not established.

NOTE THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES

Form 124 JOB A-84021 DATE 2/13/84 DR JOHN WP CHKD

LOG OF BORING

LeROY C[illegible]L AND ASSOCIATES

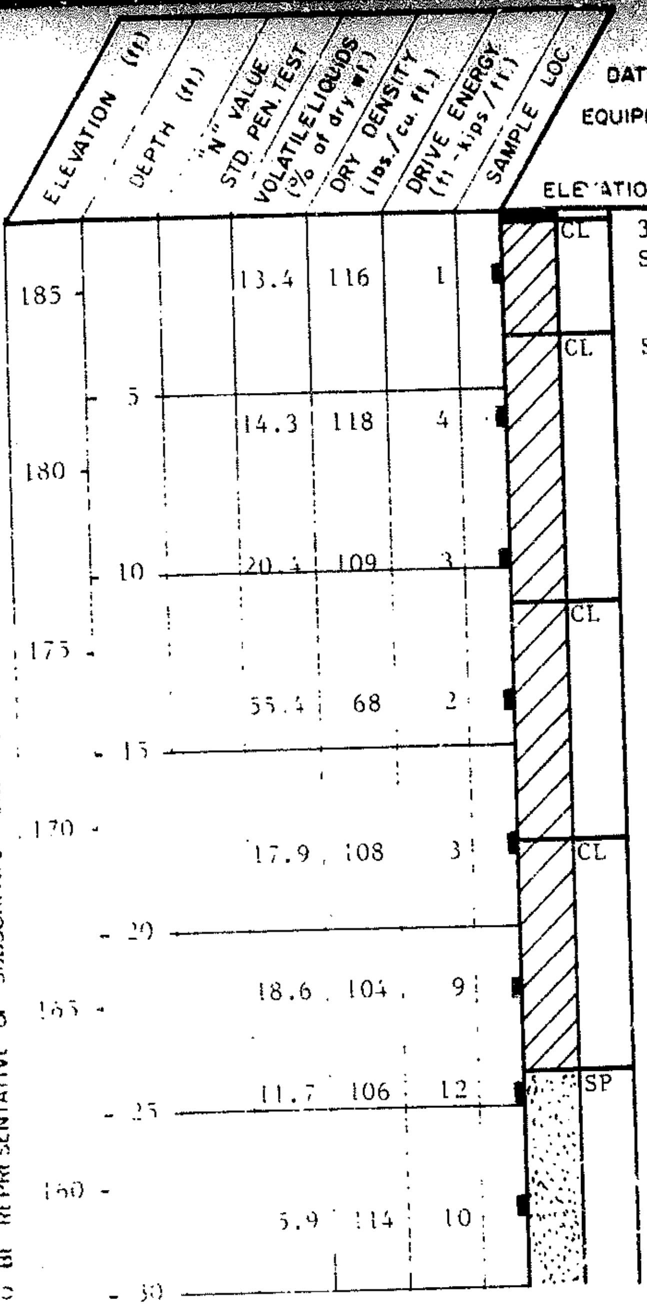
PLATE A-1.

BORING 17

DATE DRILLED: February 4, 1984

EQUIPMENT USED: 5"-Diameter Rotary Wash

ELEVATION 187



"N" VALUE STD. PEN. TEST	VOLATILE LIQUIDS (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft - kips / ft.)
	13.4	116	1
	14.3	118	4
	20.4	109	3
	55.4	68	2
	17.9	108	3
	18.6	104	9
	11.7	106	12
	5.9	114	10

CL	3" Asphaltic Paving SILTY CLAY - dark grey
CL	SANDY CLAY - bluish-grey Few cemented lumps, light grey
CL	SILTY CLAY - mottled grey
CL	SANDY CLAY - some asphaltic sand, dark grey to black
SP	ASPHALTIC SAND - fine, black

(CONTINUED ON FOLLOWING PLATE)

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

Form 123 JOB A-84021 DATE 2/13/84 DR JOHN WP CHKD

LOG OF BORING

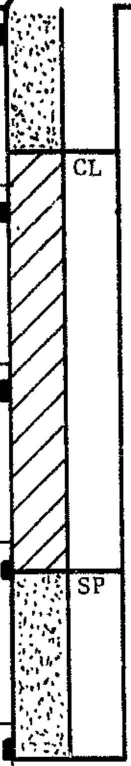
LeROY CRANDALL AND ASSOCIATES

PLATE A-1.17

BORING 17 (CONTINUED)

DATE DRILLED: February 4, 1984
EQUIPMENT USED: 5"-Diameter Rotary Wash

ELEVATION (ft.)	DEPTH (ft.)	"N" VALUE STD. PEN. TEST	VOLATILE LIQUIDS (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.
			27.5	88	10	■
155						
	35					
			14.3	106	16	■
150						
	40					
			29.2	94	14	■
145						
	45					
			45.6	75	19	■
140						
	50					
			2.1	113	14	■
135						
	55					



CL SILTY CLAY - seams of asphalt, dark grey

Seams of asphalt, some lenses of asphaltic Sand, bluish-grey

SP ASPHALTIC SAND - fine, black

NOTE: Drilling mud used in drilling process.
Water level not established.

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

JOB A-84021 DATE 2/13/84 DR JOHN CHKD W.P. YK

Form 124

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

BORING 18

DATE DRILLED: February 4, 1984

EQUIPMENT USED: 5"-Diameter Rotary Wash

ELEVATION 188

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD. PEN TEST	VOLATILE LIQUIDS (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.		
							CL	SILTY CLAY - dark grey
185			21.6	104	2	■		
	5						CL	SANDY CLAY - grey
			15.5	113	3	■		
180							CL	SILTY CLAY - some fine Sand, bluish-grey
	10							
			29.3	96	5	■		
175								
	15		33.0	91	1	■		brownish-grey
170			30.5	92	14	■		
	20							
165			24.9	98	7	■		
	25						SP	ASPHALTIC SAND - fine, black
			5.5	119	10	■		
160								
	30							

(CONTINUED ON FOLLOWING PLATE)

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

Form 123 JOB A-84021 DATE 2/13/84 DR JOHN W.P. YK CHKD

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

BORING 18 (CONTINUED)

DATE DRILLED: February 4, 1984
EQUIPMENT USED: 5"-Diameter Rotary Wash

ELEVATION (ft.)	DEPTH (ft.)	"N" VALUE STD. PEN. TEST	VOLATILE LIQUIDS (% of dry wt.)	DRY DENSITY (lbs./cu.ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.		
			5.4	119	10	■		
155							CL	SILTY CLAY - few seams of asphalt, bluish-grey
			19.6	107	10	■		
	35							
			16.0	106	10	■		Many seams of asphalt
150								
	40		33.1	87	6	■		
145							SP	ASPHALTIC SAND - fine, black
			(16.3)	(91)				
	45		5.4	106	3	■		
140								
	50		6.2	102	19	■		

NOTE: Drilling mud used in drilling process. Mud removed after completion of drilling. Water level measured at a depth of 8' on 2/8/84.

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

JOB A-8402 DATE 2/13/84 DR JOHN W.P. E CHKD

Form 124

LOG OF BORING

LeROY CR[illegible] ALL AND ASSOCIATES

PLATE A-1.18

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
COARSE GRAINED SOILS (More than 50% of material is LARGER than No. 200 sieve size)	GRAVELS (More than 50% of coarse fraction is LARGER than the No. 4 sieve size)	CLEAN GRAVELS (Little or no fines)	GW	Well graded gravels, gravel-sand mixtures, little or no fines.
			GP	Poorly graded gravels or gravel-sand mixtures, little or no fines.
		GRAVELS WITH FINES (Appreciable amt. of fines)	GM	Silty gravels, gravel-sand-silt mixtures.
			GC	Clayey gravels, gravel-sand-clay mixtures.
	SANDS (More than 50% of coarse fraction is SMALLER than the No. 4 sieve size)	CLEAN SANDS (Little or no fines)	SW	Well graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands or gravelly sands, little or no fines.
		SANDS WITH FINES (Appreciable amt. of fines)	SM	Silty sands, sand-silt mixtures.
			SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS (More than 50% of material is SMALLER than No. 200 sieve size)	SILTS AND CLAYS (Liquid limit LESS than 50)		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silty clays of low plasticity.
	SILTS AND CLAYS (Liquid limit GREATER than 50)		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS			Pt	Peat and other highly organic soils.

BOUNDARY CLASSIFICATIONS: Soils possessing characteristics of two groups are designated by combinations of group symbols.

PARTICLE SIZE LIMITS

SILT OR CLAY	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

NO. 200 NO. 40 NO. 10 NO. 4 3/4 in. 3 in. (12 in.)

U. S. STANDARD SIEVE SIZE

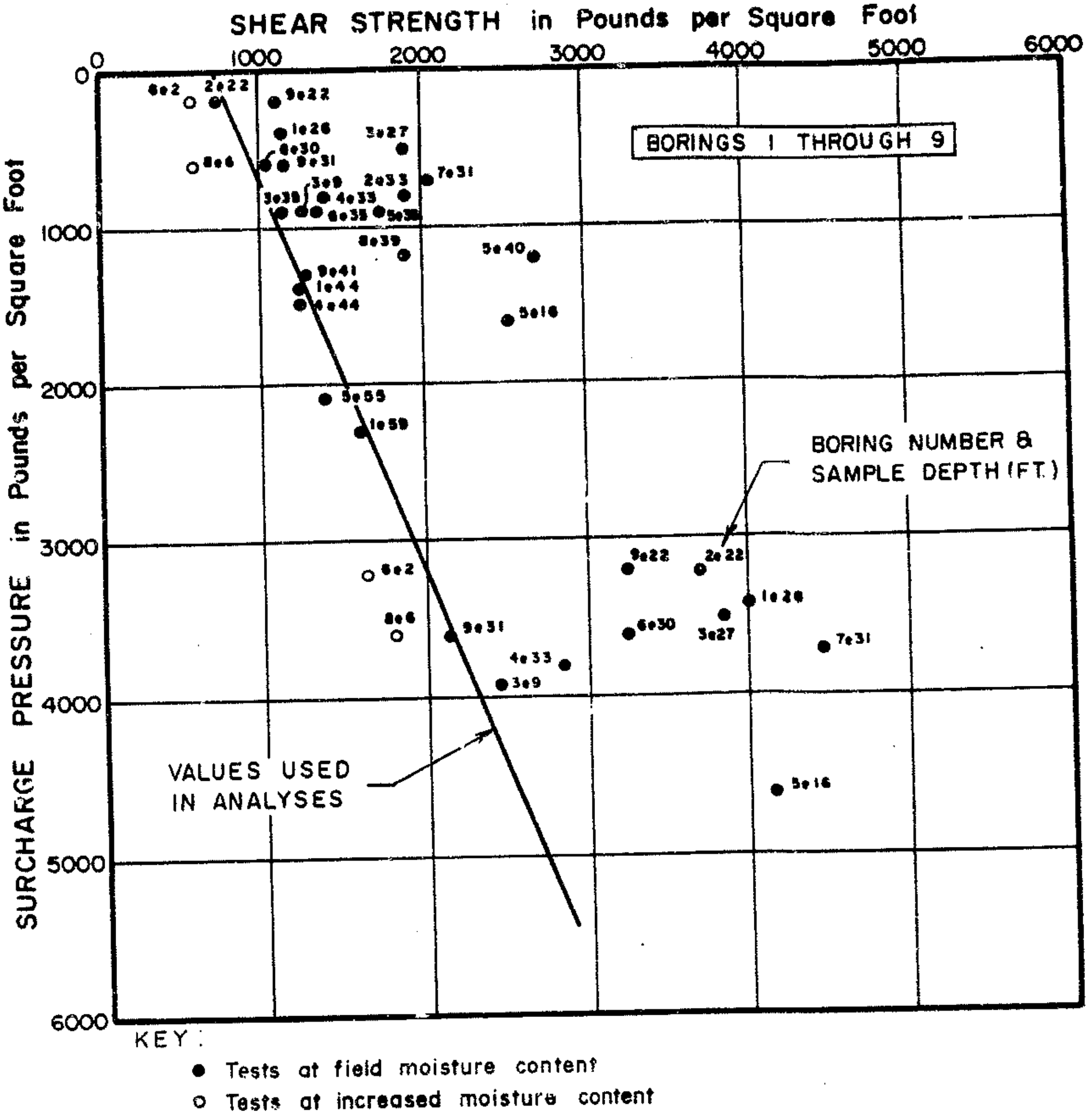
UNIFIED SOIL CLASSIFICATION SYSTEM

Reference :
The Unified Soil Classification System, Corps of Engineers, U. S. Army Technical Memorandum No. 3-357, Vol. 1, March, 1953. (Revised April, 1960)

LEROY CRANDALL AN [redacted] IATES

PLATE A-2

JOB A-8[illegible]1 DATE [illegible]/15/[illegible] BY JOHN O.F. CHKD

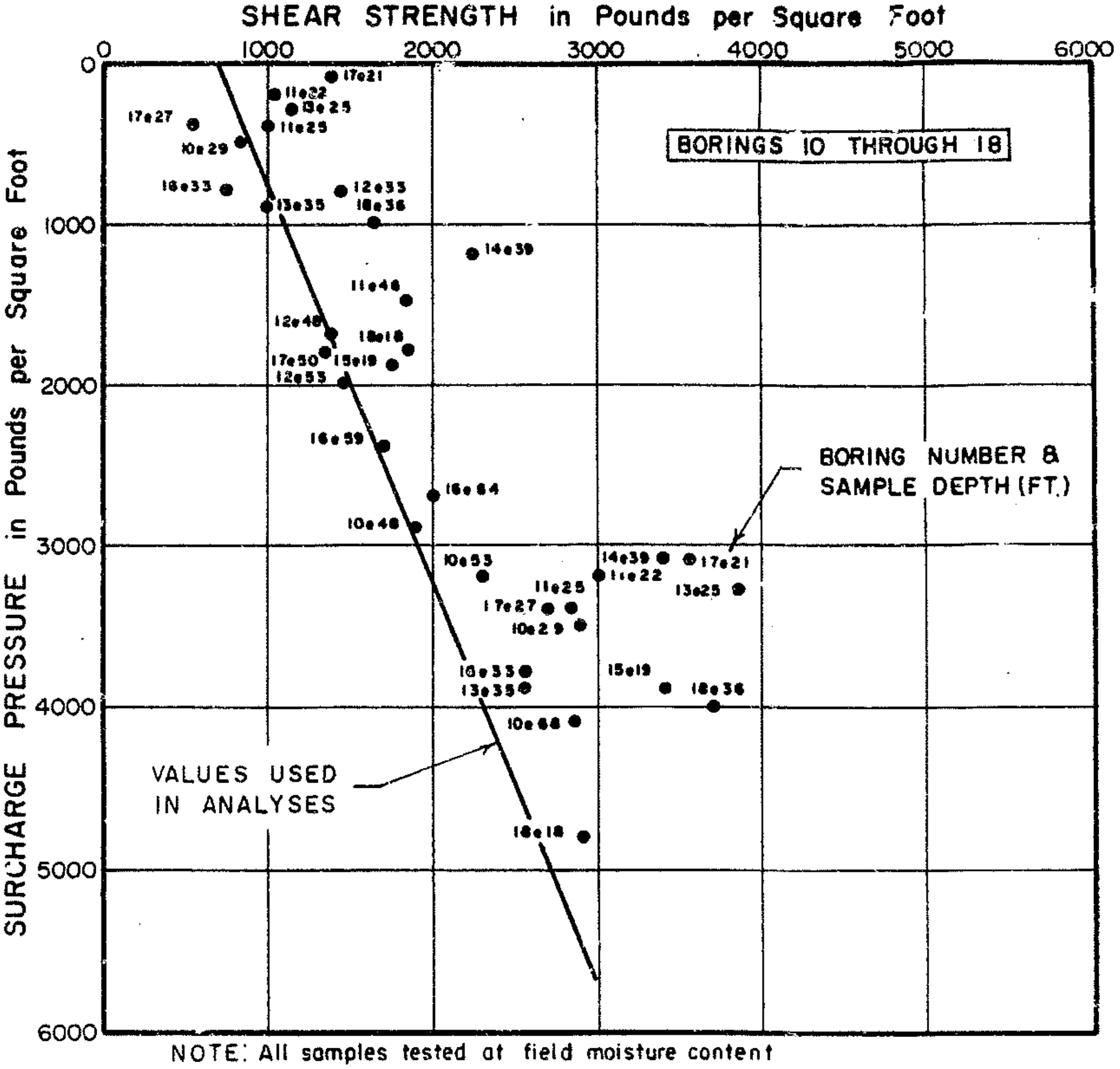


DIRECT SHEAR TEST DATA

LEROY CRANDALL & ASSOCIATES

PLATE A-3.1

A-84021 DATE 2/13/84 JOHN O.L. CHKD.



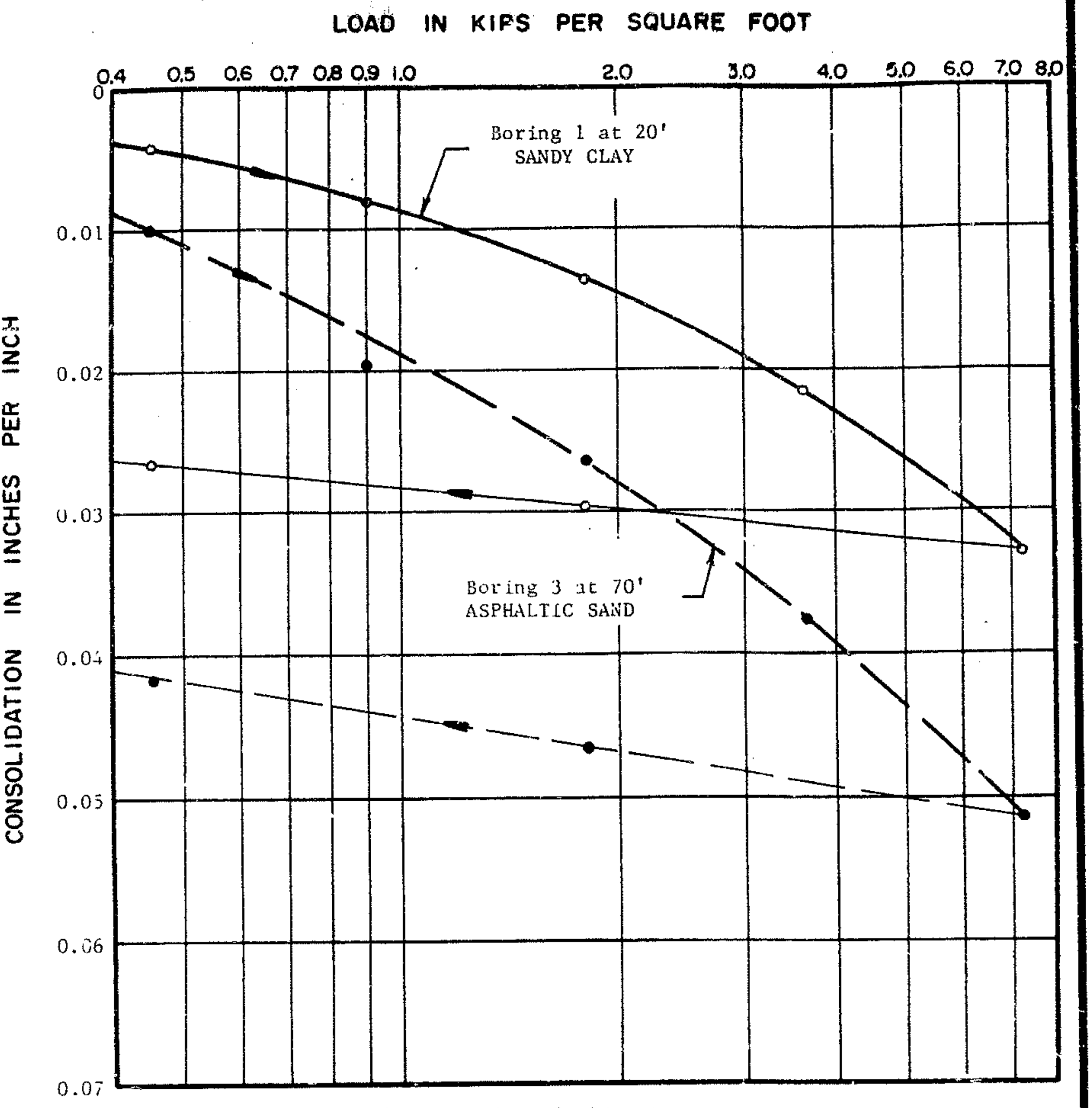
NOTE: All samples tested at field moisture content

DIRECT SHEAR TEST DATA

LEROY CRA... ...L & ASSOCIATES

PLATE A-3.2

JOB A-84021 DATE 2/20/84 DR. JOHN CHKD

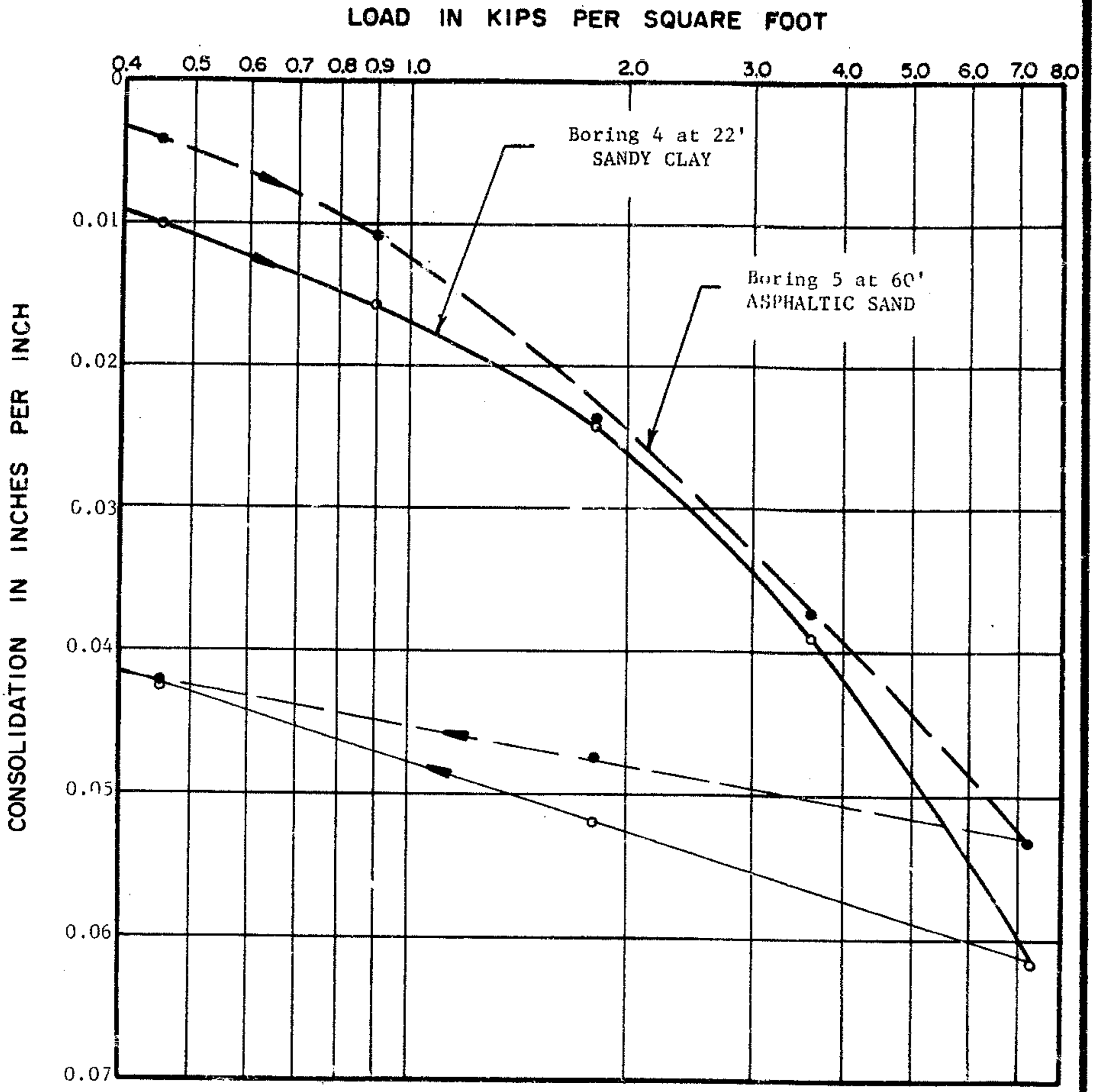


NOTE: Samples tested at field moisture content.

CONSOLIDATION TEST DATA

LeROY CRANDALL AND ASSOCIATES

JOB A-84021 DATE 2/20/84 DR. JOHN CHKD W.P.



NOTE: Samples tested at field moisture content.

CONSOLIDATION TEST DATA

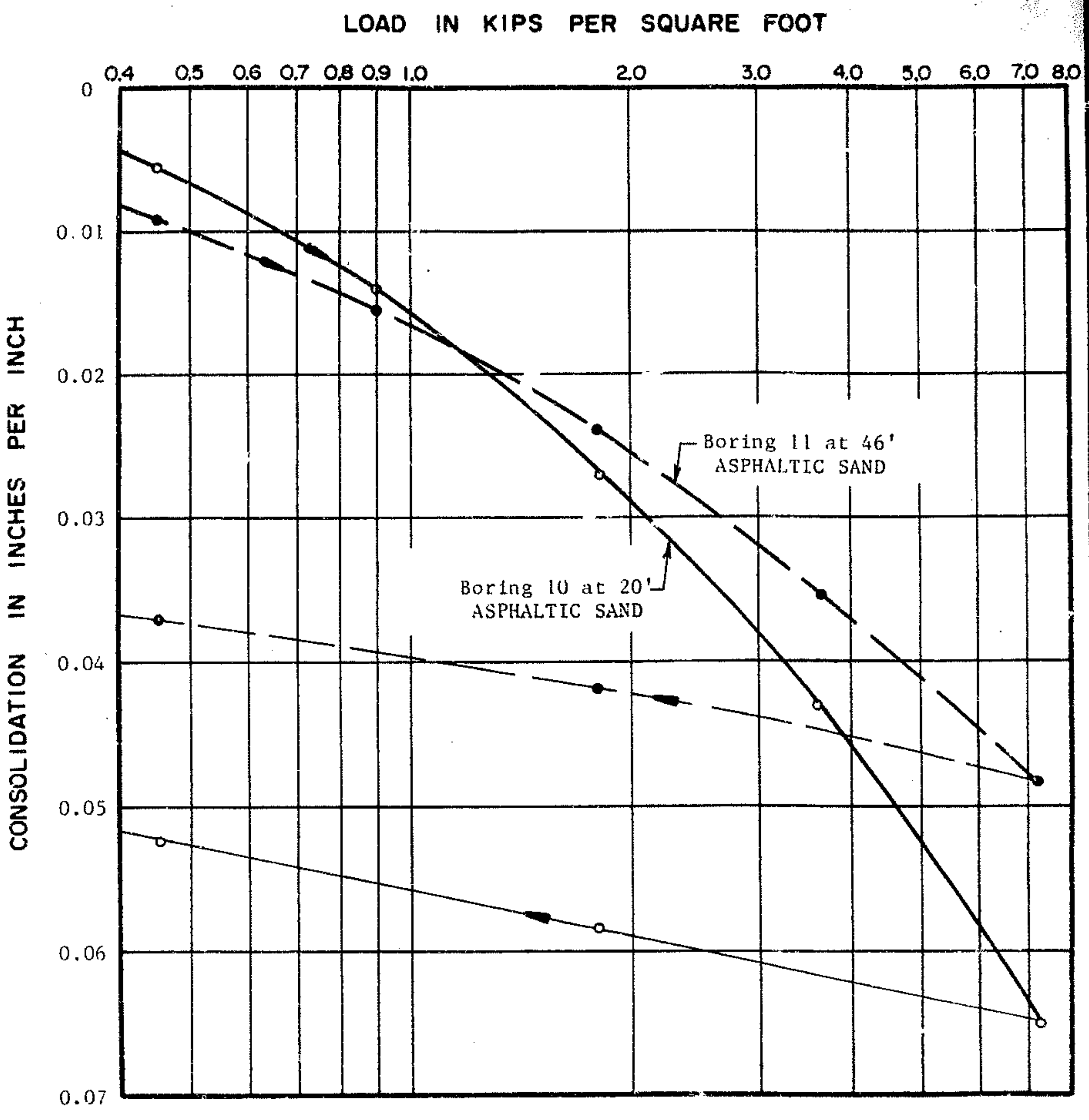
LeROY CRANDALL AND ASSOCIATES

PLATE A-4.2

FORM 115

FORM 116 JOB A-84021 DATE 2/20/84 DR. JOHN W.P. CHKD

pg



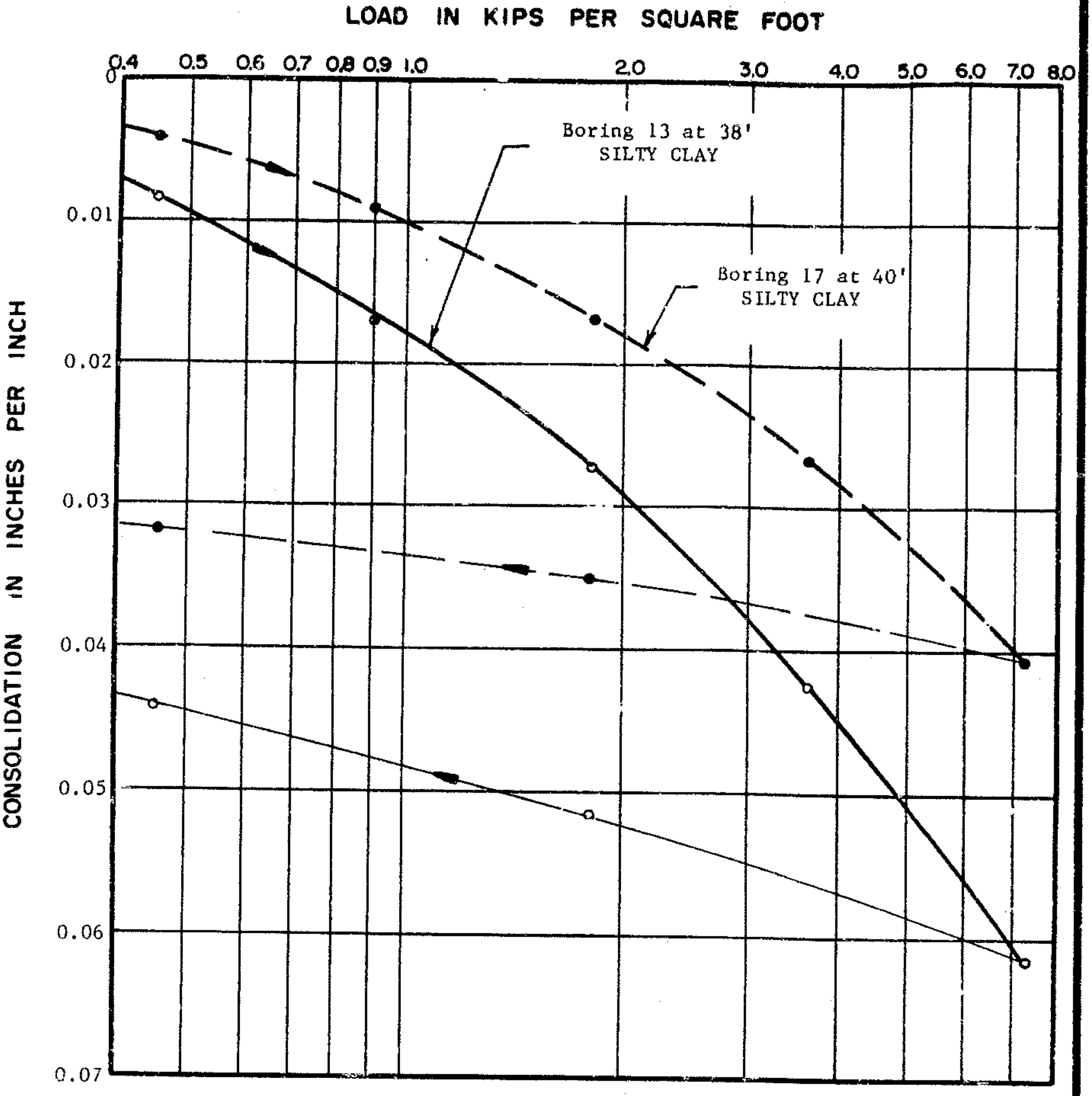
NOTE: Samples tested at field moisture content.

CONSOLIDATION TEST DATA

LeROY [illegible] AND ASSOCIATES

PLATE A-4.3

JOB A-84021 DATE 2/20/84 DR. JOHN W.P. CHKD



NOTE: Samples tested at field moisture content.

CONSOLIDATION TEST DATA

LeROY CRANDALL AND ASSOCIATES

PLATE A-4.4

FORM 118

LeROY CRANDALL AND ASSOCIATES consulting geotechnical engineers, 711 n. alvarado st., los angeles, ca. 90026, (213)413-3550, telex 69-8375

December 20, 1984

712-760 Sierra Bonita Ave
700-752 Curson Ave
713-761 Sierra Bonita Ave
712-760 Masselin Ave
5750 & 5700 Wilshire Blvd

J. H. Snyder Company
5757 Wilshire Boulevard
Los Angeles, California 90036

(Our Job No. A-84021)

Attention: Mr. Marsh Holtzman

Tract 5798
Lots: 1-8,44-52,83-93,127-135,
290-293

Gentlemen:

Supplementary Information
Driven Pile Capacities
Proposed Wilshire Courtyard
Wilshire Boulevard and Sierra Bonita Avenue
Los Angeles, California

SCOPE

This letter presents information for design of driven piling at the subject site. We previously performed a foundation investigation for the proposed project and submitted the results in a report dated March 12, 1984 (our Job No. A-84021).

Recommendations for support for the proposed building on spread footings or a mat foundation at basement level were presented in our report. The structure is being waterproofed and designed to resist the hydrostatic pressure resulting from the ground water. We have been informed that the hydrostatic uplift forces are significant and the building does not possess sufficient weight to resist these forces. Accordingly, the use of pile foundations, which will develop uplift resistance, is desired.

The professional opinions presented in this letter have been developed using that degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical engineers practicing in this or similar localities. No other warranty, expressed or implied, is intended as to the professional advice included in this letter.

PILE CAPACITIES

VERTICAL LOADS

The downward and upward capacities of 12- and 14-inch-square precast concrete piles are presented on the attached Plate 1, Driven Pile Capacities. Dead plus live load capacities are shown; a one-third increase may be used when considering wind or seismic loads. The pile capacities are based on the strength of the soils; the compressive and tensile strength of the pile section itself should be checked to verify the structural capacity of the piles.

Piles in groups should be spaced at least three feet on centers. If the piles are so spaced, no reduction in the downward capacity of the piles due to group action need be considered in design.

The settlement of the proposed building, supported on driven piles, will be less than one-half inch.

INSTALLATION

The piles should be driven to the predetermined design lengths except as may be modified on the basis of the driving criteria presented on Plate 2, Pile Driving Criteria. Continuous observation of the pile driving should be performed by personnel of our firm. Predrilling of the pile excavations will be required to achieve the desired penetration. The depth and diameter of the predrilled hole should be adjusted to achieve the desired driving resistance.

TEST PILES

The soils at the site consist primarily of asphalt sands from approximately basement level to depths of 40 to 50 feet below basement level. To confirm the supporting capability of the asphaltic sands, we recommend that at least two pile load tests be performed. One of the piles should be tested for downward capacity and one should be tested for upward capacity. The length and load on the test piles should be determined after the pile design has been completed and the design loads are known.

LATERAL LOADS

Lateral loads may be resisted by the piles. It may be assumed that the soils adjacent to a 12-inch-square concrete pile at least 30 feet long can resist horizontal loads at the top of the pile up to 12,000 pounds. The lateral resistance of other sizes of piles may be assumed to be proportional to the width.

In calculating the maximum bending moment in a pile, the lateral load imposed at the top of the pile may be multiplied by an assumed moment arm of four feet. For design, it may be assumed that the maximum bending moment will occur at or near the top of the pile and that the bending moment will decrease to zero at a depth of 20 feet below the pile cap. The lateral capacity and reduction in bending moment are based in part on the assumption that any required backfill adjacent to the pile caps and grade beams will be properly compacted.

Yours very truly,

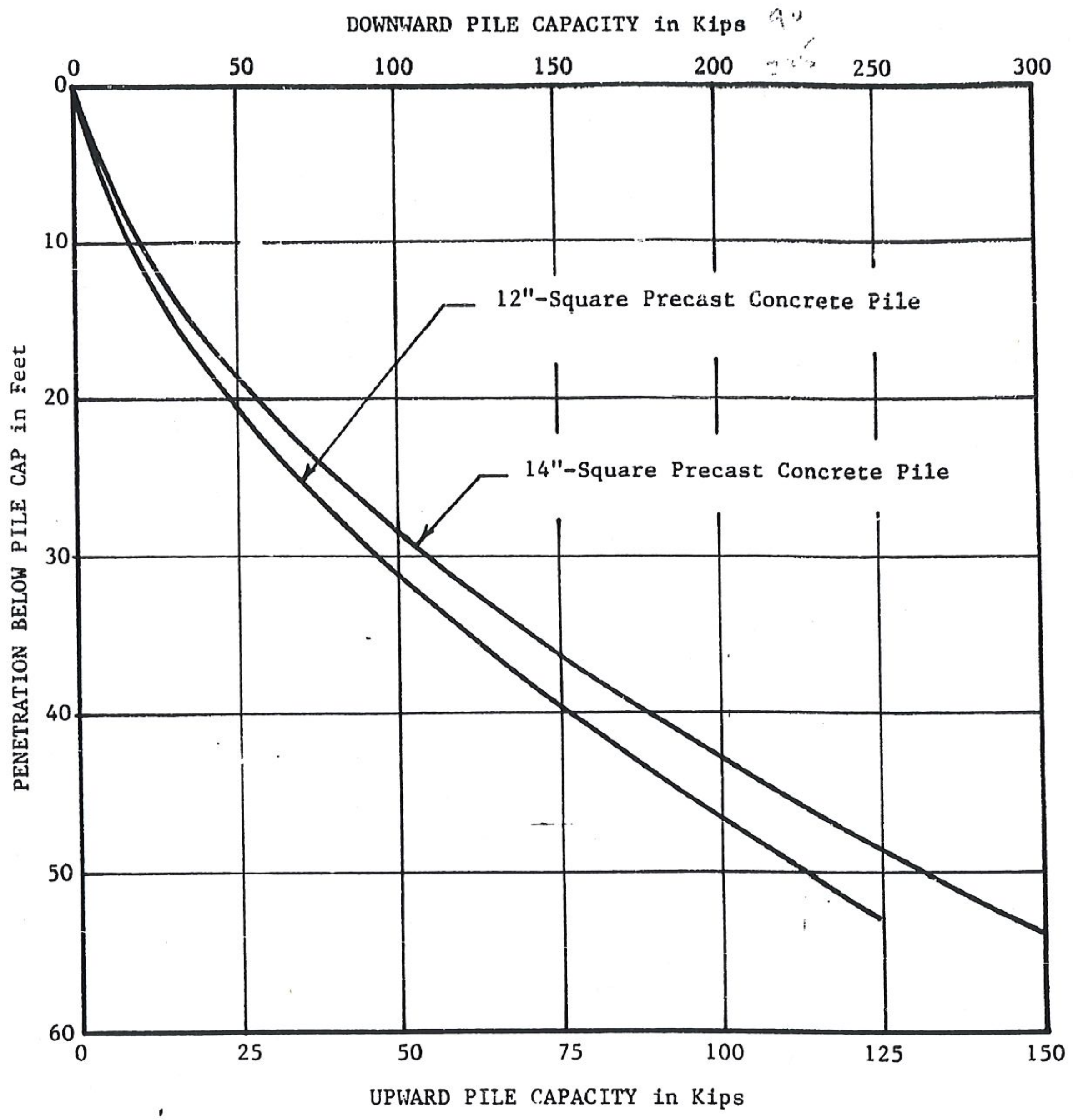
LeROY CRANDALL AND ASSOCIATES

by James L. Van Beveren
James L. Van Beveren, R.C.E. 17785
Project Engineer

by
J. D. Kirkgard, R.C.E. 10441
Executive Vice President

JK-VB/P11
Attachments (2)
(4 copies submitted)

cc: (2) Carl McLarand and Associates, Inc.
(1) Erkel Greenfield and Associates
(1) M. H. Golden Company



NOTES:

(1) The indicated values refer to the total of dead plus live loads; a one-third increase may be used when considering wind or seismic loads.

(2) Piles in groups should be spaced a minimum of 3 feet on centers.

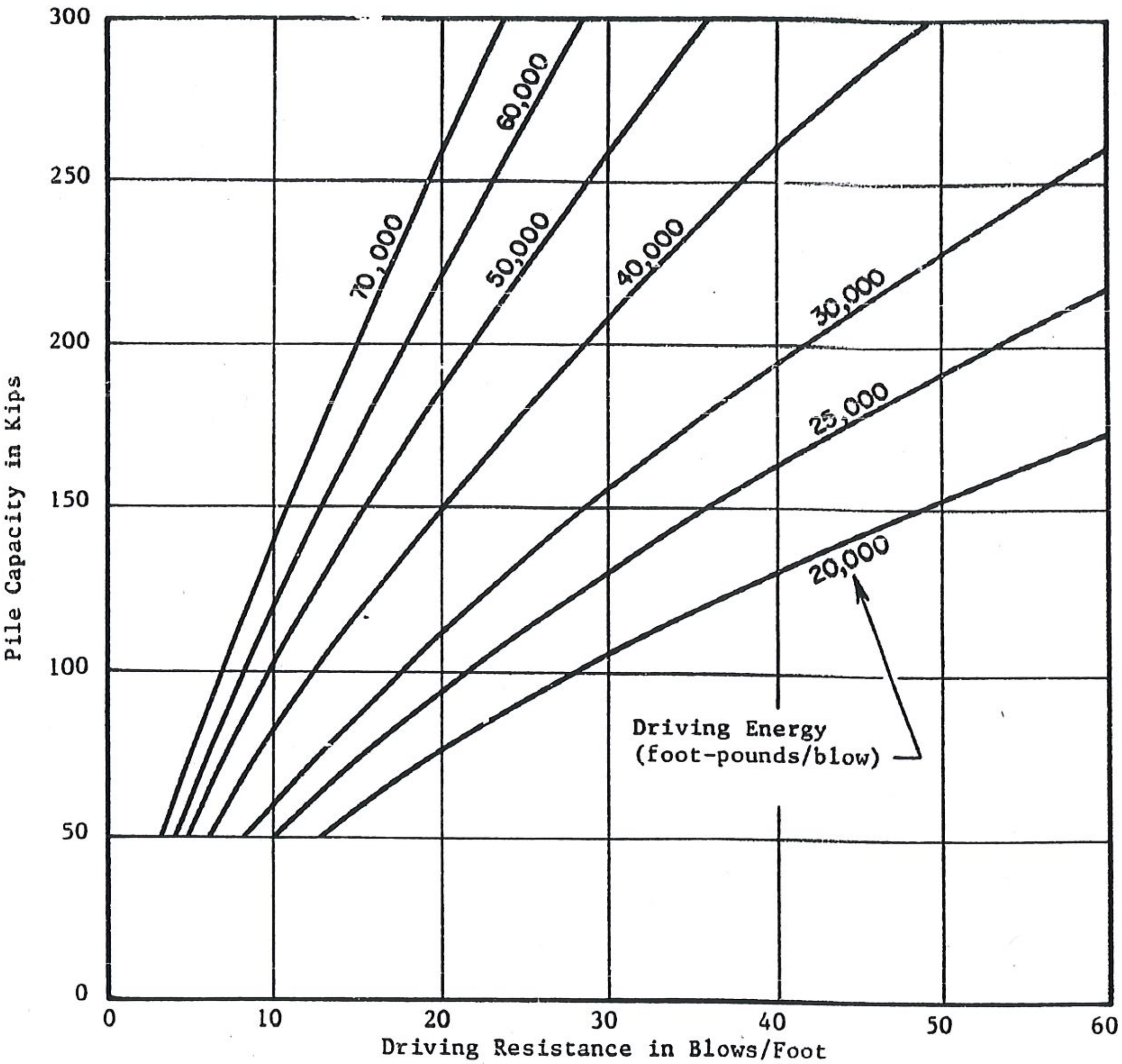
(3) The indicated values are based on the strength of the soils; the actual pile capacities may be limited to lesser values by the strength of the piles.

DRIVEN PILE CAPACITIES

FORM 12^
JOB A-84021 DATE 12/18/84 DR. JOHN P.E. VB W.P. ge CHKD YK VB

LeROY CRANDALL AND ASSOCIATES

JOB A-84021 DATE 12-19-84 DR. VB CHKD. VB



NOTES:

1) The above driving resistance should be obtained for the last foot of driving at design pile length. If the driving resistance at the design length is less than above, the piles should be lengthened until the desired driving resistance is obtained.

2) As an alternate to lengthening when low driving resistance is obtained, the piles may be allowed to set overnight and the number of blows to drive the pile one inch the following day should be determined. If the restarting resistance is at least two times the above criteria, the pile may be considered satisfactory.

3) If driving resistance of three times the above criteria is encountered within five feet of design length, the pile driving may be stopped.

PILE DRIVING CRITERIA

LeROY CRANDALL AND ASSOCIATES

LeROY CRANDALL AND ASSOCIATES consulting geotechnical engineers, 711 n. alvarado st., los angeles, ca. 90026, (213) 413-3550

TR. 5798
LOT: 1-8, 44-52, 85-93,
127-135, 290-295

F' 4229

February 25, 1985

RECEIVED FEB 26 1985

J. H. Snyder Company
5757 Wilshire Boulevard
Los Angeles, California 90036

(Our Job No. A-84021)

Attention: Mr. Marsh Holtzman

700-752 Carson Ave
713-761 Sierra Bonita
715-761 Masselin
712-760 Sierra Bonita

Gentlemen:

Supplementary Information for
Excavation and Shoring
Proposed Wilshire Courtyard
Wilshire Boulevard and Sierra Bonita Avenue
Los Angeles, California

SCOPE

This letter provides supplementary information for excavation and for design of shoring at the subject site. We previously performed a foundation investigation for the subject development and submitted the results in our report dated March 12, 1984 and supplementary letter dated December 20, 1984. At the time our report was submitted, the elevation of the lower level had not been definitely established, but it was anticipated that excavation some 20 to 30 feet deep would be required. We have now been informed that the bottom of the excavation will be established at Elevation 155, some 30 to 35 feet below the existing grade. The information in this letter supersedes the information in our prior report regarding excavation and shoring.

The information in this letter represents professional opinions that have been developed using that degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical engineers practicing in this or similar localities. No other warranty, expressed or implied, is made as to the professional advice included in this letter.

SUPPLEMENTARY RECOMMENDATIONS

GENERAL

At the time our report was submitted, it was contemplated that the excavation would extend only slightly into the asphaltic sands. Based on present plans, the excavation will extend some 5 to 15 feet into the sands, and some modifications of our prior recommendations are needed. The modifications are also based on recent experiences with excavation into the asphaltic sands on nearby projects. Supplementary recommendations for excavation, for design of shoring (including tie-back anchors), and for basement walls are presented below.

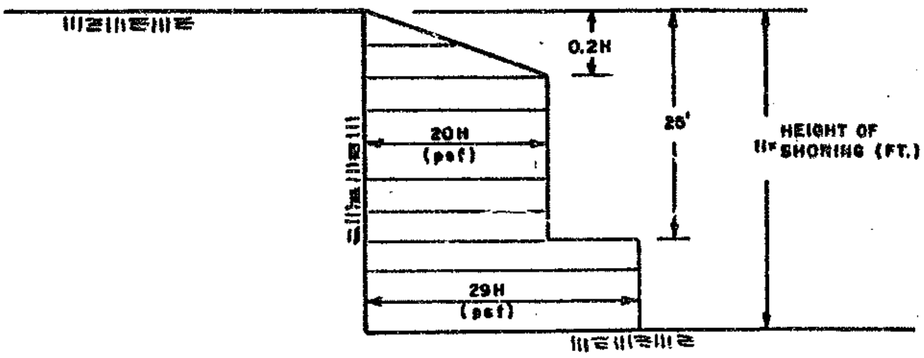
EXCAVATION

Where the necessary space is available, temporary unsurcharged excavations within the clay soils may be sloped back at 3/4:1 (horizontal to vertical) in lieu of using shoring. Where the excavation extends into the asphaltic sands, the slopes should be flattened to 1:1. For design, it may be assumed that the clays extend approximately 25 feet below the existing grade. A stability calculation for the temporary cut slopes is presented on the attached sheet.

SHORING

Lateral Pressures

For the design of braced or tied-back shoring, we recommend the use of a trapezoidal distribution of lateral earth pressure. The recommended pressure distribution, for the case where the grade is level behind the shoring, is illustrated below. (If a combination of sloped embankment and shoring is to be used, the pressure would be greater and must be determined for each combination.)



In addition to the recommended earth pressure, the upper ten feet of shoring adjacent to the streets should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pounds per square foot surcharge behind the shoring due to normal street traffic. If the traffic is kept back at least ten feet from the shoring, the traffic surcharge may be neglected.

Design of Soldier Piles

For the design of soldier piles spaced at least two diameters on centers, the allowable lateral bearing value (passive value) of the soils below the level of excavation may be assumed to be 500 pounds per square foot per foot of depth, up to a maximum of 6,000 pounds per square foot. To develop the full lateral value, provisions should be taken to assure firm contact between the soldier piles and the undisturbed soils. Structural concrete should be used for that portion of a soldier pile which is below the excavated level; lean mix concrete may be used above that level.

The frictional resistance between the soldier piles and the retained earth may be used in resisting a portion of the downward component of the anchor load. The coefficient of friction between the soldier piles and the retained earth may be 0.4 in the clay soils and 0.2 in the asphalt sands. (These values are based on the assumption that uniform full bearing will be developed between the steel soldier beam and the lean-mix concrete and between the lean-mix concrete and the retained earth.) In addition, the soldier piles below the excavated level may be used to resist downward loads. The frictional resistance between the concrete soldier piles and the soils below the excavated level may be taken as equal to 250 pounds per square foot.

Lagging

Continuous lagging will be required between the soldier piles within the asphaltic sands and within any zones of water seepage. If the clear spacing between soldier piles does not exceed four feet, it may be possible to omit lagging within the cohesive soils above the asphaltic sands and above any water seepage zones. We recommend that the exposed soils be observed by personnel of our firm to determine the areas where lagging may be omitted.

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 soils. We recommend that the lagging be designed for the recommended earth pressure but limited to a maximum value of 400 pounds per square foot.

Anchor Design

Tie-back anchors may be used to resist lateral loads. The anchors may be designed as friction anchors. However, the anchors will extend into the asphalt sands and may be subject to long term creep. Accordingly, we recommend that the anchors be belled to minimize the potential for creep. We recommend that the bells be at least 24 in in diameter.

Some difficulty should be anticipated in the drilling of the anchors due to water and asphalt within the soils. Also, caving could be experienced in the drilling of the anchors through the sand deposits.

The use of special techniques may be found necessary to permit the installation of the anchors.

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. Friction anchors should extend at least 25 feet beyond the potential active wedge and to a greater length if necessary to develop the desired capacities.

The capacities of the anchors should be determined by testing of the initial anchors as outlined in a following paragraph. For design purposes, it may be estimated that drilled friction anchors will develop an average friction value of 600 pounds per square foot. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads. If the anchors are spaced at least six feet on centers, no reduction in the capacity of the anchors need be considered due to group action.

Anchor Installation

The anchors may be installed at angles of 20 to 40 degrees below the horizontal. 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. In order 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 flush 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.

Anchor Testing

Our firm should select two of the initial anchors for 24-hour 200% tests and six additional anchors for quick 200% tests. The purpose of the 200% tests 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.

The total deflection during the 24-hour 200% tests should not exceed 12 inches. During the 24-hour tests, the anchor deflection should not exceed 0.75 inch measured after the 200% test load is applied. If the anchor movement after the 200% load has been applied for 12 hours is less than 0.5 inch, and the movement over the previous 4 hours has been less than 0.1 inch, the 24-hour test may be terminated.

For the quick 200% tests, the 200% test load should be maintained for 30 minutes. The total deflection of the anchor during the 200% quick tests should not exceed 12 inches; the deflection after the 200% test load has been applied should not exceed 0.25 inch during the 30-minute period.

All of the anchors should be pretested to at least 150% of the design load; the total deflection during the test should not exceed 12 inches. The rate of creep under the 150% test should not exceed 0.1 inch over a 15-minute period in order for the anchor to be approved for the design loading.

After a satisfactory test, each 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% from the design load, the load should be reset until the anchor is locked-off within 10% of the design load.

The installation of the anchors and the testing of the completed anchors should be observed by our firm.

Deflection

It is difficult to accurately predict the amount of deflection of a shored embankment. It should be realized, however, that some deflection will occur. We would estimate that this deflection could be on the order of one to two inches at the top of a 35-foot-deep shored embankment. If greater deflection occurs during construction, additional bracing may be necessary to minimize damage to utilities in the adjacent streets. If desired to reduce the deflection of the shoring, a greater active pressure could be used in the shoring design.

BASEMENT WALLS

As stated in our report, we recommend that the basement walls be designed to resist a trapezoidal distribution of lateral earth pressure plus the hydrostatic pressure. The previously presented pressures for design of walls below grade are still appropriate.

Yours very truly,

LeROY CRANDALL AND ASSOCIATES

by James L. Van Beveren
James L. Van Beveren, R.C.E. 17785
Project Engineer

by J D Kirkgaard
J. D. Kirkgaard, R.C.E. 10441
Executive Vice President

JK-VB/pa
Attachment
(2 copies submitted)

cc: (2) Carl McLarand and Associates, Inc.
(1) Erkel, Greenfield and Associates
(2) M. H. Golden Company
(1) KaWes & Associates
(1) Eugene D. Birnbaum and Associates

Form 107

LEROY CRANDALL AND ASSOCIATES
Consulting Geotechnical Engineers

SHEET 1 OF 1
BY VB 2-22-85 (date)
CHECKED BY YK 2-25-85 (date)

CLIENT J.H. Snyder (Wilshire Courtyard)

SUBJECT Stability of temporary Cut slopes

JOB NO. A 84021

Soil Strengths

Clay:

$c = 900$ psf
$\phi = 22°$

Sand:

$c = 200$ psf
$\phi = 31°$

The excavation for the mat will extend to Elevation 155. The ground surface around the perimeter of the site varies from Elev. 185 to 190. 30'-35' high temporary cut slopes will result.

The soils exposed in the cut slopes will vary from all clay to 20' of clay underlain by 15' of asphalt sand.

Analyze using Taylor and assume uniform soil conditions throughout
F.S. = 1¼ (temporary) $\gamma = 120$ pcf

Clay

$C_d = 720$ psf ✓
$\phi_d = 18°$ ✓

$N = {}^{720}/_{(120)(35)} = 0.171$ ✓

$i = 84°$ ✓

Sand

$C_d = 160$ psf ✓
$\phi_d = 26°$ ✓

$N = {}^{160}/_{(120)(35)} = 0.038$ ✓

$i = 43°$ ✓

Recommend ¾:1 in Clay ✓
1:1 in Sand ✓

City of Los Angeles
DEPARTMENT OF BUILDING AND SAFETY
Grading Division

District: L.A.
Thomas Bros. Map Index
Log No.
District Map 5103

APPLICATION FOR REVIEW OF TECHNICAL REPORTS AND IMPORT-EXPORT ROUTES

INSTRUCTIONS

A. Address all communications to the Grading Division, Department of Building and Safety, Room 460A, City Hall, Los Angeles, California 90012-4869. Phone (Area Code 213) 485-3435.
B. Submit 3 copies of application with items ① through ⑩ completed (Please print).
C. Attach 3 copies (4 for fault study zone) of reports.
D. Check should be made payable to the Department of Building and Safety.

① LEGAL DESCRIPTION
Tract 5798
Blk — Lots 1-8 inc, 44-52 inc, 88-95 inc ~~127-135 inc., 270-295 inc~~

② PROJECT ADDRESS 700-752 CULLSON AV
713-761 SIERRA BONITA
716-761 MARGELA
712-760 SIERRA BONITA

③ OWNER J.H. SYNDER CO
Address 5757 WILSHIRE BLVD P.H
City LOS ANGELES CA Zip 90036
Phone 213 857-5546

④ APPLICANT Ka WEB & ASSOCIATES
Address 2804 BEVERLY BLVD
City LOS ANGELES CA
Phone (213) 384-7418 Zip 90057

⑤ Report(s) Prepared by LEROY CRANDALL & ASSOCIATES

⑥ Report Date(s) MAR 12, 1984

⑦ Status of project: ☑ Proposed ☐ Under Construction ☐ Storm Damage

⑧ Previous site reports? ____ If yes, give date(s) of report(s) and name of company(s) who prepared report(s).

⑨ Previous Department actions? ____ If yes, please give dates and attach a copy to expedite processing.
Dates

⑩ Signature of applicant
Position CIVIL ENGR AS AGENT FOR J.H. SYNDER

(DEPARTMENT USE ONLY)

REVIEW REQUESTED & PROCESSING	FEES	REVIEW REQUESTED & PROCESSING	FEES
☑ Foundation Investigation	168	☐ Seismology report per 91.2305(d)	
☐ Soils Engineering		☐ Environmental Assessment	
☐ Geology		☐ Import-Export Route	
☑ Combined Soils Engr. & Geol.		☐ Division of Land	
☐ Supplemental		Sub-total	
☐ Combined Supplemental		One-Stop Surcharge	3 36
		TOTAL FEE	171 36

THE REPORT IS ☑ APPROVED WITH CONDITIONS ☐ NOT APPROVED

DEPARTMENT ACTION BY: ____ For Geology ____ Date ____ For Soils & Foundation ____ Date

☑ Conditions of Approval ☐ Reasons for Non-Approval ☐ See Attached letter ☑ Supplemental Sheet 1 Attached

① Provide a supplemental report giving data ~~for~~ design of tie-back anchors prior to issuance of permits.

② Conditions No. 1-3, 9, 10, 11, 13, 14-16, 21, 25 of the attached "Supplemental Conditions For Foundation Investigation Reports."

(Continued Over)

C 3.36 OSS
C 168.00 GR&R
C4306 2 01/16/85 171.36 CHT
(Cashier Use Only)

DEPARTMENT USE ONLY
Fee Due 171 36
Fee Verified AJF
Date 1/16/84

DISTRIBUTION
☑ Owner
☑ Petitioner
☐ ____
☑ Soil Engineer
☐ Geologist
☐ Board files:
☑ Tract file

☒ LA Plan Check
☐ VN
☐ WLA
☐ SP/WLA

☒ LA Inspection
☐ VN
☐ WLA
☐ SP/WLA

② A grading permit for basement wall backfill shall be obtained.

③ Provide a letter from the adjoining property owners allowing tie-back anchors on their property.

④ No fill shall be placed until City grading inspector has inspected and approved the bottom excavation.

⑤ Provide calculations to substantiate 30 high temporary cuts at ¾:1 with Factor of Safety = 1.25 and 1.5 with surcharge.

[illegible] geotechnical engineers, 711 n. alvarado st., los angeles, ca. 90026, (213)413-3550, telex 69-8375

4774

April 24, 1985

RECEIVED APR 2 6 [illegible]

J.H. Snyder Company
5757 Wilshire Boulevard
Los Angeles, California 90036

(Our Job No. A-84021)

TRACT 5798
LOT 1-85 WC 44-52 [illegible]
85-93 WC, 127-137 [illegible]
ADDRESS 300-[illegible] [illegible]
715-761 Sierra [illegible]
714-761 [illegible]
712-760 Sierra Bonita

Attention: Mr. Marsh Holtzman

Gentlemen:

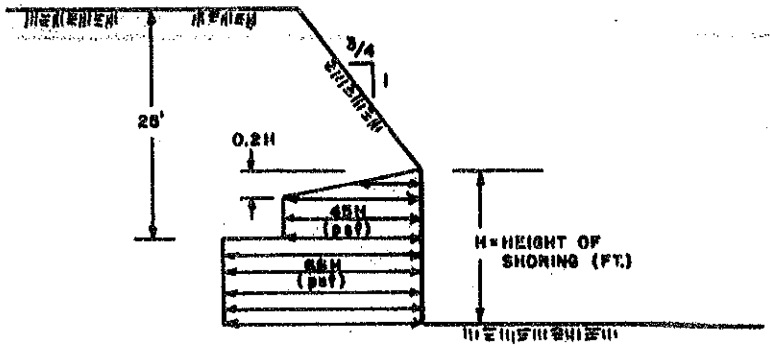
Lateral Pressures on Shoring
Proposed Wilshire Courtyard
Wilshire Boulevard and Sierra Bonita Avenue
Los Angeles, California

This letter presents supplementary information for design of shoring at the subject site. We previously performed a foundation investigation for the subject development and submitted the results in a report dated March 12, 1984 and in supplementary letters dated December 20, 1984 and February 25, 1985.

The professional opinions presented in this letter have been developed using that degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical engineers practicing in this or similar localities. No other warranty, expressed or implied, is intended as to the professional advice included in this letter.

Lateral earth pressures for design of shoring were presented in our letter of February 25, 1985. The pressures presented were for the case where the surface of the retained earth is level. The excavation for the project will be approximately 35 feet deep. Where shoring will be used, the grade above the shoring will be sloped at 3/4:1 (horizontal to vertical). The vertical portion of the shoring will vary from about 6 to 17 feet in height.

The recommended lateral earth pressure where the shoring will retain a 3/4:1 slope is presented below.



Yours very truly,

LeROY CRANDALL AND ASSOCIATES

by James L. Van Beveren

James L. Van Beveren, R.C.E. 17785
Project Engineer

by

J. D. Kirkgard, R.C.E. 10441
Executive Vice President

JK-VB/B4
(2 copies submitted)

cc: (2) Carl McLarand and Associates, Inc.
(1) Erkel/Greenfield and Associates
(2) M. H. Golden Company
(1) KaWes and Associates
(1) Eugene D. Birnbaum and Associates

LeROY CRANDALL AND ASSOCIATES consulting geotechnical engineers, 711 n. alvarado st., los angeles, ca. 90026, (213)413-3890, telex 69-8375

4808

[redacted]

May 1, 1985

Location
7900-452 Curson
713-761 Sierra Bonita
715-761 Marcelin
712-760 Sierra Bonita

J.H. Snyder Company
5757 Wilshire Boulevard
Los Angeles, California 90036

(Our Job No. A-84021)

Attention: Mr. Marsh Holtzman

Gentlemen:

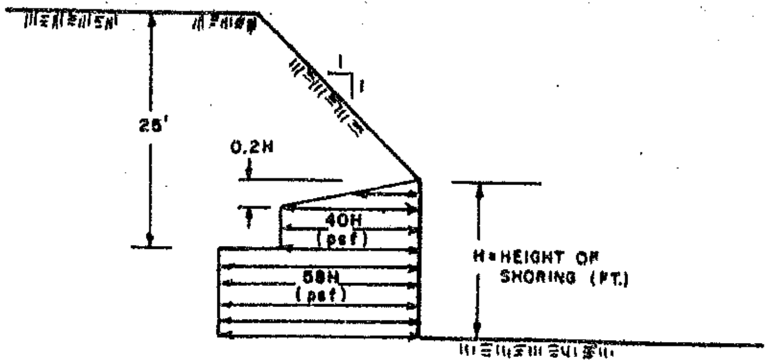
Lateral Pressures on Shoring
Proposed Wilshire Courtyard
Wilshire Boulevard and Sierra Bonita Avenue
Los Angeles, California

This letter presents supplementary information for design of shoring at the subject site. We previously performed a foundation investigation for the subject development and submitted the results in a report dated March 12, 1984 and in supplementary letters dated December 20, 1984, February 25, and April 24, 1985.

The professional opinions presented in this letter have been developed using that degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical engineers practicing in this or similar localities. No other warranty, expressed or implied, is intended as to the professional advice included in this letter.

Lateral earth pressures for design of shoring were presented in our letters of February 25, and April 24, 1985. The pressures presented were for the cases where the surface of the retained earth was level or sloped at 3/4:1 (horizontal to vertical). We have now been requested to provide the anticipated lateral earth pressure for the case where the shoring will retain an excavation sloped at 1:1.

The recommended lateral earth pressure where the shoring will retain a 1:1 slope is presented below.



Yours very truly,

LeROY CRANDALL AND ASSOCIATES

by James L. Van Beveren
James L. Van Beveren, R.C.E. 17785
Project Engineer

by
J. D. Kirkgard, R.C.E. 10441
Executive Vice President

JK-VB/B4
(2 copies submitted)

cc: (2) Carl McLarand and Associates, Inc.
    (1) Erkel/Greenfield and Associates
    (2) M. H. Golden Company
    (4) KaWes and Associates
    (1) Eugene D. Birnbaum and Associates

LeROY CRANDALL AND ASSOCIATES consulting geotechnical engineers, 711 n. alvarado st., los angeles, ca. 90026, (213) 413-3550, telex 69-8375

712-760 Sierra Bonita Ave
700-752 Curson Ave
713-761 Sierra Bonita Ave
712-760 Masselin Ave
5750 & 5700 Wilshir

5396

July 9, 1985

TRACT: 5798
BLOCK:
LOT: 293 + 291
JOB ADDRESS: 5750½ 5700 Wilshire Blvd
L.A. 90036

J. H. Snyder Company
5757 Wilshire Boulevard
Los Angeles, California 90036

(Our Job Nos. A-84021,
A-84021-2 and B-85086)

Attention: Mr. Marsh Holtzman

Gentlemen:

Driven Pile Capacities
Proposed Wilshire Courtyard
Wilshire Boulevard and Sierra Bonita Avenue
Los Angeles, California

This letter presents revised capacities for driven piling at the subject project. This letter supplements our report of foundation investigation for the project dated March 12, 1984 and a supplementary letter dated December 20, 1984. The anticipated capacities of 12- and 14-inch-square precast concrete piles were presented in our letter of December 20, 1984. Because of uncertainties in the supporting capacity of the asphaltic sand, we recommended that at least two pile load tests be performed. The pile capacities presented in this letter are based upon three pile load tests recently performed at the subject site.

Our professional services have been performed using that degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical engineers practicing in this or similar localities. No other warranty, expressed or implied, is made as to the professional advice included in this letter.

To provide information on the pile driving characteristics of the soils at the site, thirteen 12-inch-square precast concrete indicator piles have been driven. Load tests were performed on three of these piles. Two of the piles were tested for upward resistance and one of the piles was tested for downward resistance. The results of the pile load test program will be submitted under separate cover.

Based on the results of the pile load tests, we have revised the anticipated capacities of the driven piles. The capacities of 12- and 14-inch-square precast concrete piles are presented on the attached Plate 1, Driven Pile Capacities. As shown by the installation of the indicator piles, pile driving will be difficult and predrilling of the pile locations will be required to permit installation of the piles to the desired lengths. The diameter of the predrilled holes should not be larger than the width of the pile. The predrilling should not extend deeper than five feet from the pile tip.

The piles should be driven to the predetermined design lengths except as may be modified on the basis of the driving criteria presented on Plate 2, Pile Driving Criteria. Continuous observation of the pile driving and predrilling should be performed by personnel of our firm.

Yours very truly,

LeROY CRANDALL AND ASSOCIATES

by James L. Van Beveren

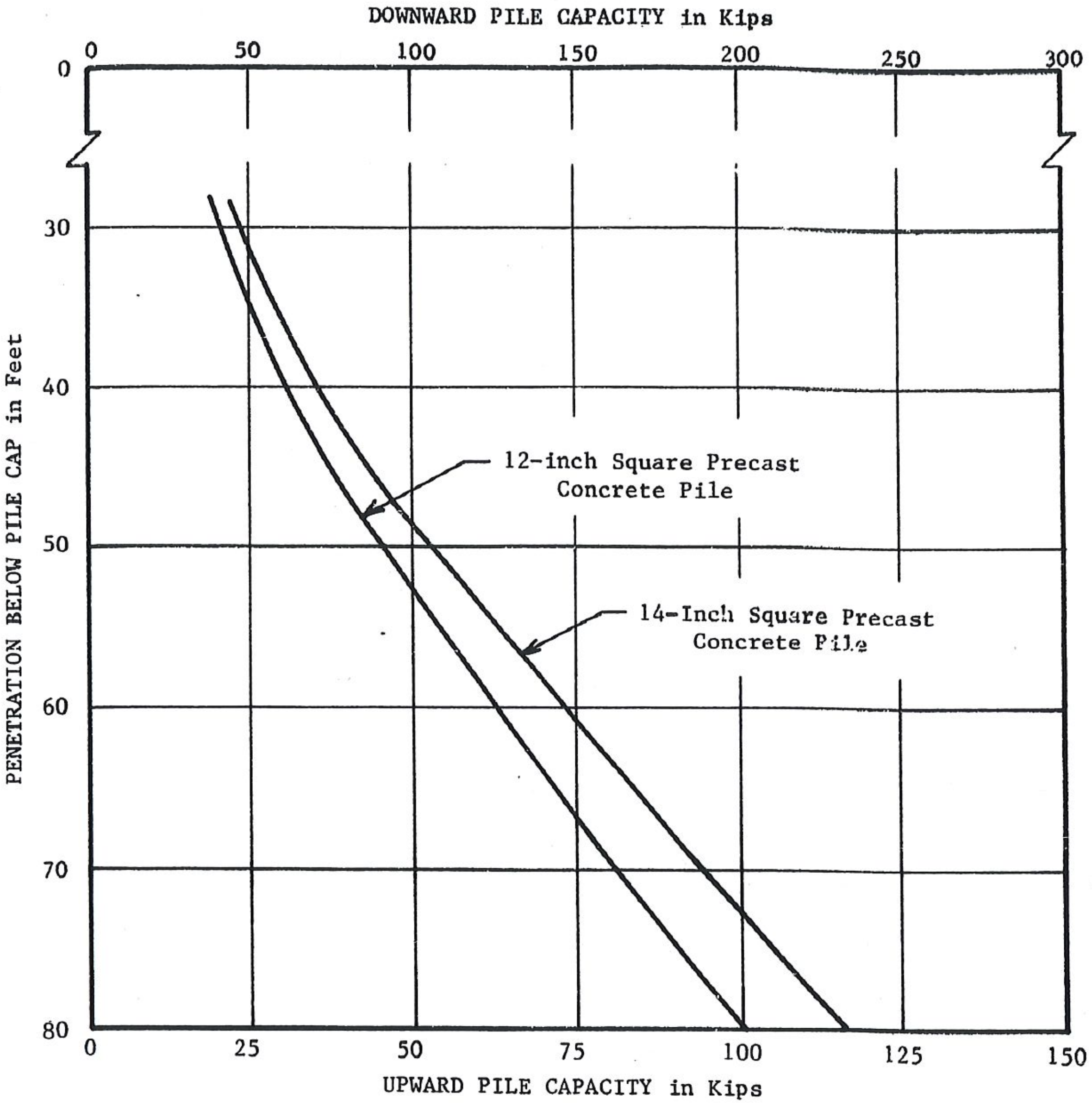
James L. Van Beveren, R.C.E. 17785
Project Engineer

by JD Kirkgard

J. D. Kirkgard, R.C.E. 10441
President

L4/ge
Attachments (2)
(4 copies submitted)

cc: (2) Carl McLarand Associates, Inc.
(1) Erkel, Greenfield and Associates
(2) M.H. Golden Company



NOTES: (1) The indicated values refer to the total of dead plus live loads; a one-third increase may be used when considering wind or seismic loads.

(2) Piles in groups should be spaced a minimum of 2½ diameters, but not less than 3 feet, on centers.

(3) The indicated values are based on the strength of the soils; the actual pile capacities may be limited to lesser values by the strength of the piles.

DRIVEN PILE CAPACITIES

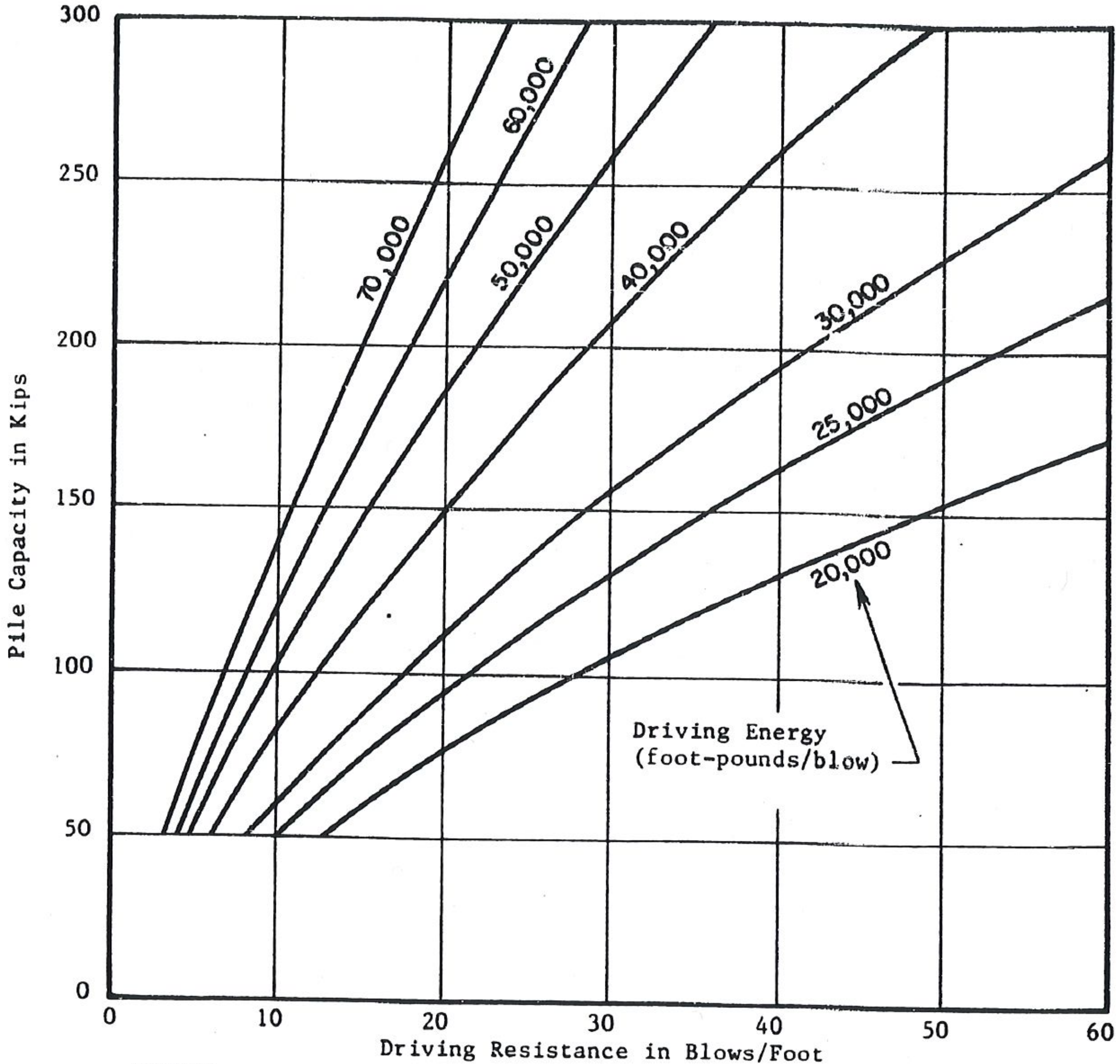
LeROY CRANDALL AND ASSOCIATES

PLATE I

JOB A-84021E DATE 6/26/85 DR. RV O.E. VB W.P. ge CHKD VB

FORM 120

JOB A-84021-E DATE 7/9/85 DR. Todd O.E. VB CHKD. VB



NOTES:

1) The above driving resistance should be obtained for the last foot of driving at design pile length. If the driving resistance at the design length is less than above, the piles should be lengthened until the desired driving resistance is obtained.

2) As an alternate to lengthening when low driving resistance is obtained, the piles may be allowed to set overnight and the number of blows to drive the pile one inch the following day should be determined. If the restarting resistance is at least two times the above criteria, the pile may be considered satisfactory.

3) If driving resistance of three times the above criteria is encountered within five feet of design length, the pile driving may be stopped.

PILE DRIVING CRITERIA

LeROY CRANDALL AND ASSOCIATES

LeROY CRANDALL AND ASSOCIATES consulting geotechnical engineers, 711 n. alvarado st., los angeles, ca. 90026, (213)413-3550, telex 69-8375

5396

July 9, 1985

TRACT: 5798
BLOCK:
LOT: 293 & 291
JOB ADDRESS: 5750 & 5700 Wilshire Blvd
L.A. 90036

J. P Snyder Company
5757 Wilshire Boulevard
Los Angeles, California 90036

(Our Job Nos. A-84021,
A-84021-E and B-85086)

Attention: Mr. Marsh Holtzman

Gentlemen:

Driven Pile Capacities
Proposed Wilshire Courtyard
Wilshire Boulevard and Sierra Bonita Avenue
Los Angeles, California

This letter presents revised capacities for driven piling at the subject project. This letter supplements our report of foundation investigation for the project dated March 12, 1984 and a supplementary letter dated December 20, 1984. The anticipated capacities of 12- and 14-inch-square precast concrete piles were presented in our letter of December 20, 1984. Because of uncertainties in the supporting capacity of the asphaltic sand, we recommended that at least two pile load tests be performed. The pile capacities presented in this letter are based upon three pile load tests recently performed at the subject site.

Our professional services have been performed using that degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical engineers practicing in this or similar localities. No other warranty, expressed or implied, is made as to the professional advice included in this letter.

To provide information on the pile driving characteristics of the soils at the site, thirteen 12-inch-square precast concrete indicator piles have been driven. Load tests were performed on three of these piles. Two of the piles were tested for upward resistance and one of the piles was tested for downward resistance. The results of the pile load test program will be submitted under separate cover.

Based on the results of the pile load tests, we have revised the anticipated capacities of the driven piles. The capacities of 12- and 14-inch-square precast concrete piles are presented on the attached Plate 1, Driven Pile Capacities. As shown by the installation of the indicator piles, pile driving will be difficult and predrilling of the pile locations will be required to permit installation of the piles to the desired lengths. The diameter of the predrilled holes should not be larger than the width of the pile. The predrilling should not extend deeper than five feet from the pile tip.

The piles should be driven to the predetermined design lengths except as may be modified on the basis of the driving criteria presented on Plate 2, Pile Driving Criteria. Continuous observation of the pile driving and predrilling should be performed by personnel of our firm.

Yours very truly,

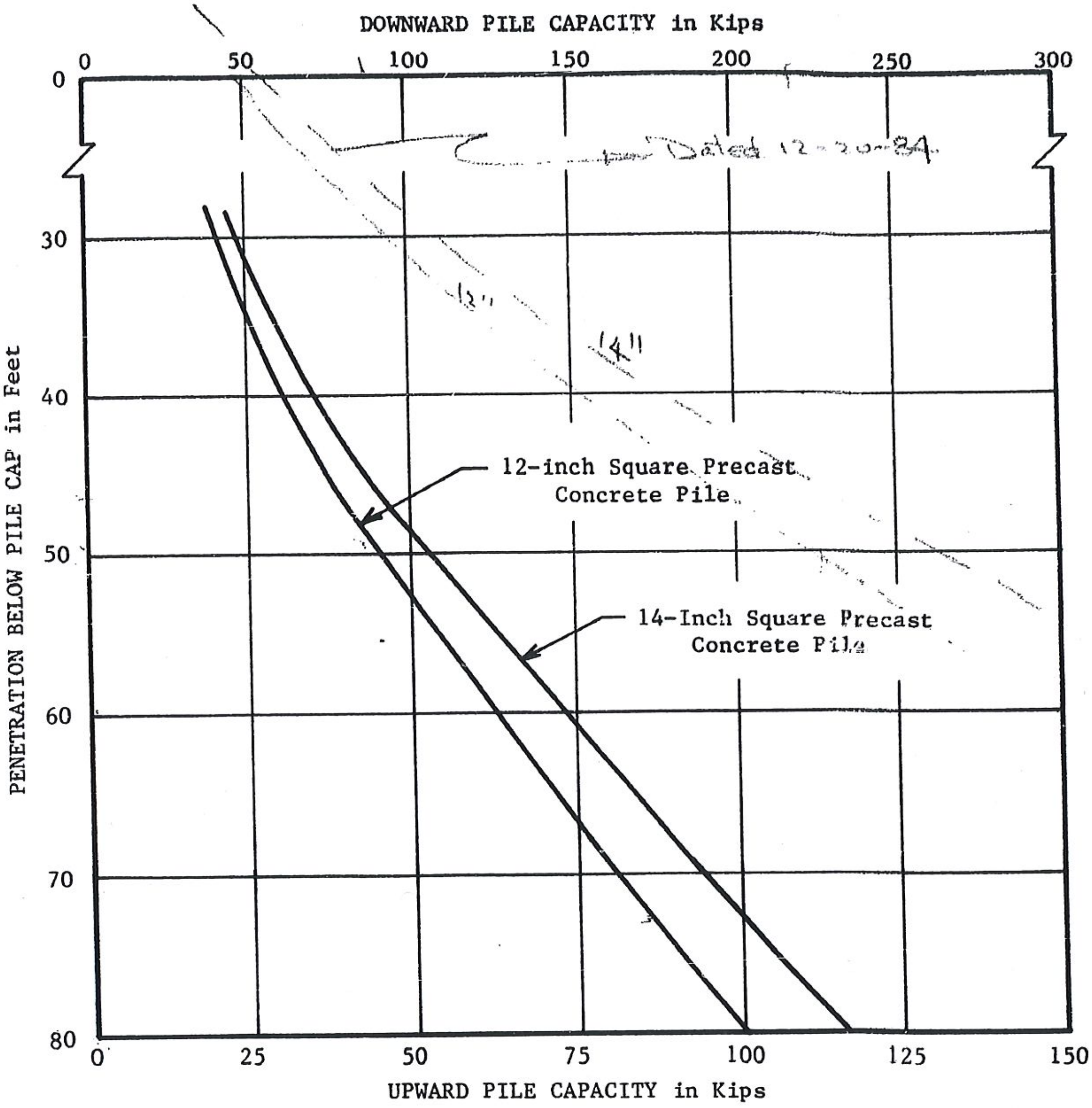
LeROY CRANDALL AND ASSOCIATES

by James L. Van Beveren
James L. Van Beveren, R.C.E. 17785
Project Engineer

by J D Kirkgard
J. D. Kirkgard, R.C.E. 10441
President

L4/ge
Attachments (2)
(4 copies submitted)

cc: (2) Carl McLarand Associates, Inc.
(1) Erkel, Greenfield and Associates
(2) M.H. Golden Company



NOTES: (1) The indicated values refer to the total of dead plus live loads; a one-third increase may be used when considering wind or seismic loads.

(2) Piles in groups should be spaced a minimum of 2½ diameters, but not less than 3 feet, on centers.

(3) The indicated values are based on the strength of the soils; the actual pile capacities may be limited to lesser values by the strength of the piles.

DRIVEN PILE CAPACITIES

LeROY CRANDALL AND ASSOCIATES

PLATE I

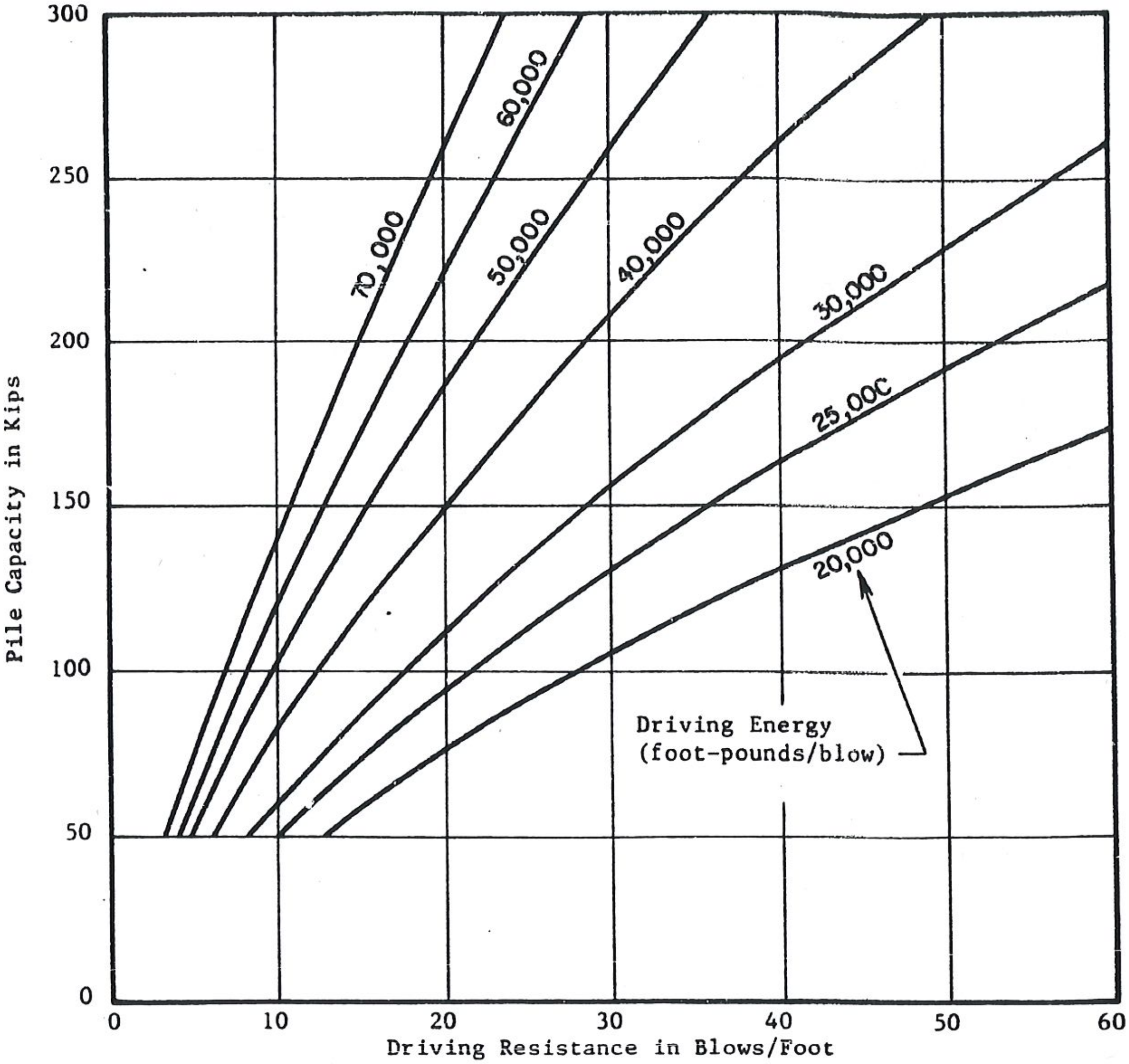
FORM 120

JOB A-84021E DATE 6/26/85 DR. AV O.E. VB W.P. ge CHKD VB

062

JOB A-84021-E DATE 7/9/85 DR. Todd O.E. VB CHKD. VB

1 1 0 0 6 3



NOTES:

1) The above driving resistance should be obtained for the last foot of driving at design pile length. If the driving resistance at the design length is less than above, the piles should be lengthened until the desired driving resistance is obtained.

2) As an alternate to lengthening when low driving resistance is obtained, the piles may be allowed to set overnight and the number of blows to drive the pile one inch the following day should be determined. If the restarting resistance is at least two times the above criteria, the pile may be considered satisfactory.

3) If driving resistance of three times the above criteria is encountered within five feet of design length, the pile driving may be stopped.

PILE DRIVING CRITERIA

LeROY CRANDALL AND ASSOCIATES

PLATE 2

LeROY CRANDALL AND ASSOCIATES consulting geotechnical engineers, 711 n. alvarado st., los angeles, ca. 90026, (213)413-3550, telex 69-8375

5993

TRACT: 5798
BLOCK:
LOT: 1-8, 44-52, 85-93, 127-135, 290-293
JOB ADDRESS:
780 CURSON AVE ET. AL

25.0200433

August 29, 1985

Wilshire Courtyard
5757 Wilshire Boulevard
Los Angeles, California 90036

(Our Job No. A-84021-G)

Attention: Mr. William Lieberman

Gentlemen:

Report of Supplementary Pile Load Tests
and Modification of Pile Capacities
Proposed Wilshire Courtyard
Wilshire Boulevard between Masselin
and Curson Avenues
Los Angeles, California

This letter presents the results of supplementary pile load tests and modifications to the driven pile capacities at the subject site. Our report of foundation investigation for the project was submitted on March 12, 1984 (A-84021). Supplementary information regarding driven pile capacities was presented in a letter dated December 20, 1984. Our report of pile load test program was submitted on July 16, 1985 (A-84021-E).

Pile installation is in progress. Since the piles are achieving a greater resistance than the indicator piles used for the initial test program, these additional load tests have been performed to justify appropriate reductions in the required pile lengths. As previously indicated, the upward loadings on the piles are the critical factor in establishing the required pile lengths.

Our professional services have been developed using that degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical engineers practicing in this or similar localities. No other warranty, expressed or implied, is made as to the professional advice included in this letter.

Two 14-inch-square, 73-foot long production piles within the east building were selected for upward load tests. The tests were performed on Piles B-10-5 and B-10-8. Data relative to the test piles are presented on the following page:

Pile Designation	Date Driven	Penetration Below Adjacent Grade (feet)	Predrilled Depth (feet)	Driving Resistance Last 5 feet (blows per foot)	Drive Energy (foot-pounds per blow)
B-10-5	8/8/85	73	69	42 47 50 49 49	30,400
B-10-8	8/8/85	73	71	38 33 36 37 37	30,400

The load tests were performed in accordance with the procedures described in our prior pile load test report dated July 16, 1985. The results of the pile load tests are shown on Plate 1, Pile Load Test Data.

Based upon the results of the pile load tests, a moderate increase can be made in the previously recommended capacities. The revised capacities for a 14-inch square precast concrete pile are presented on Plate 2, Driven Pile Capacities. For comparative purposes, the previously determined capacities are also shown on Plate 1. The remainder of the recommendations contained in our report of July 16 should be followed.

Yours very truly,

LeROY CRANDALL AND ASSOCIATES

by James L. Van Beveren

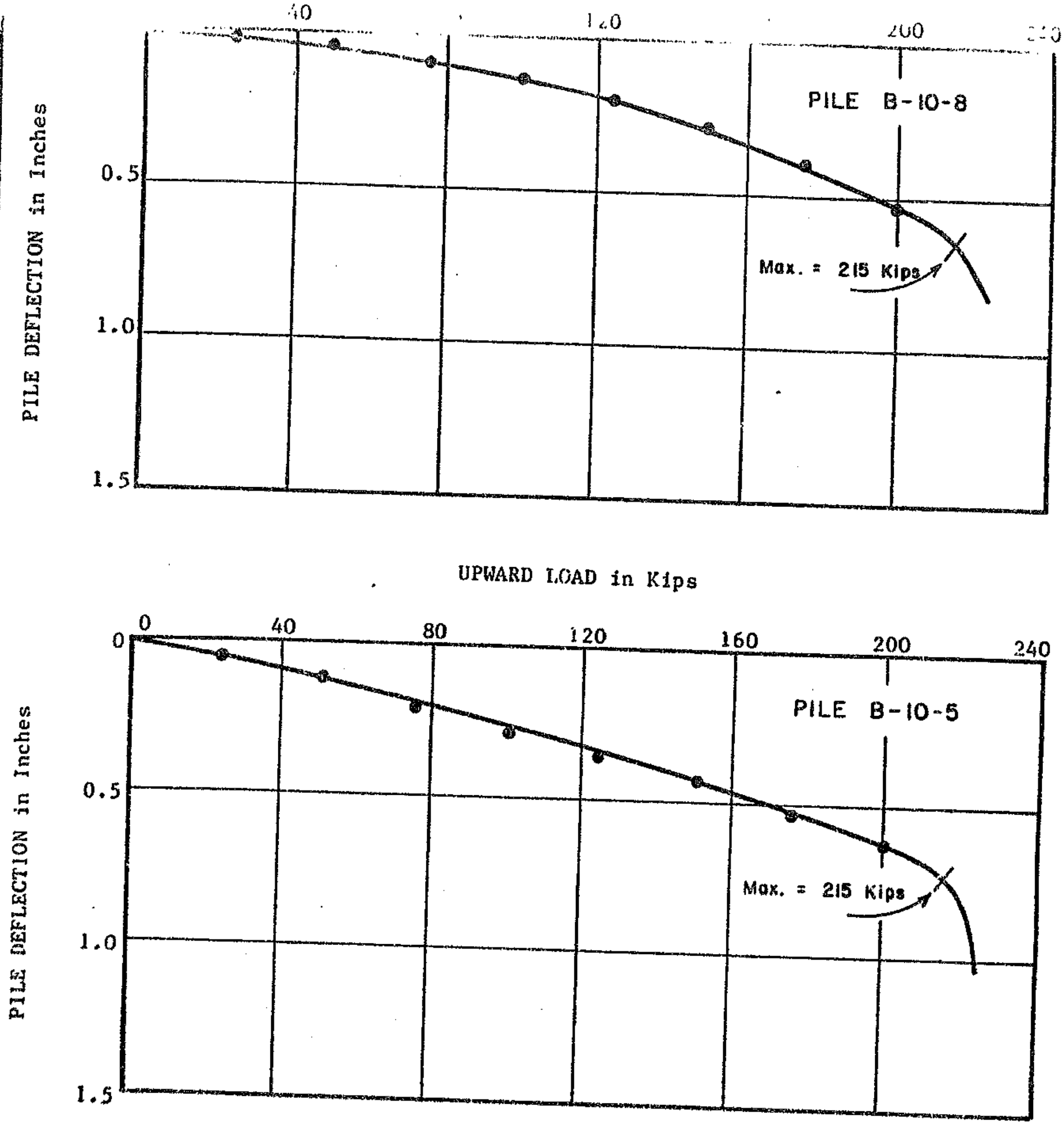
James L. Van Beveren, R.C.E. 17785
Project Engineer

by LeRoy Crandall

LeRoy Crandall, R.C.E. 6157
Chairman

E9/ge
Attachments (2)
(6 copies submitted)

cc: (2) McLarand, Vasquez & Partners, Inc.
(1) Erkel/Greenfield and Associates
(2) M.H. Golden Company (Job Site)
(2) B-85086 File



PILE LOAD TEST DATA

JOB 4-840216 DATE 8/28/85 DR. RJ. O.E. VB W.P. dmh CHKD VB

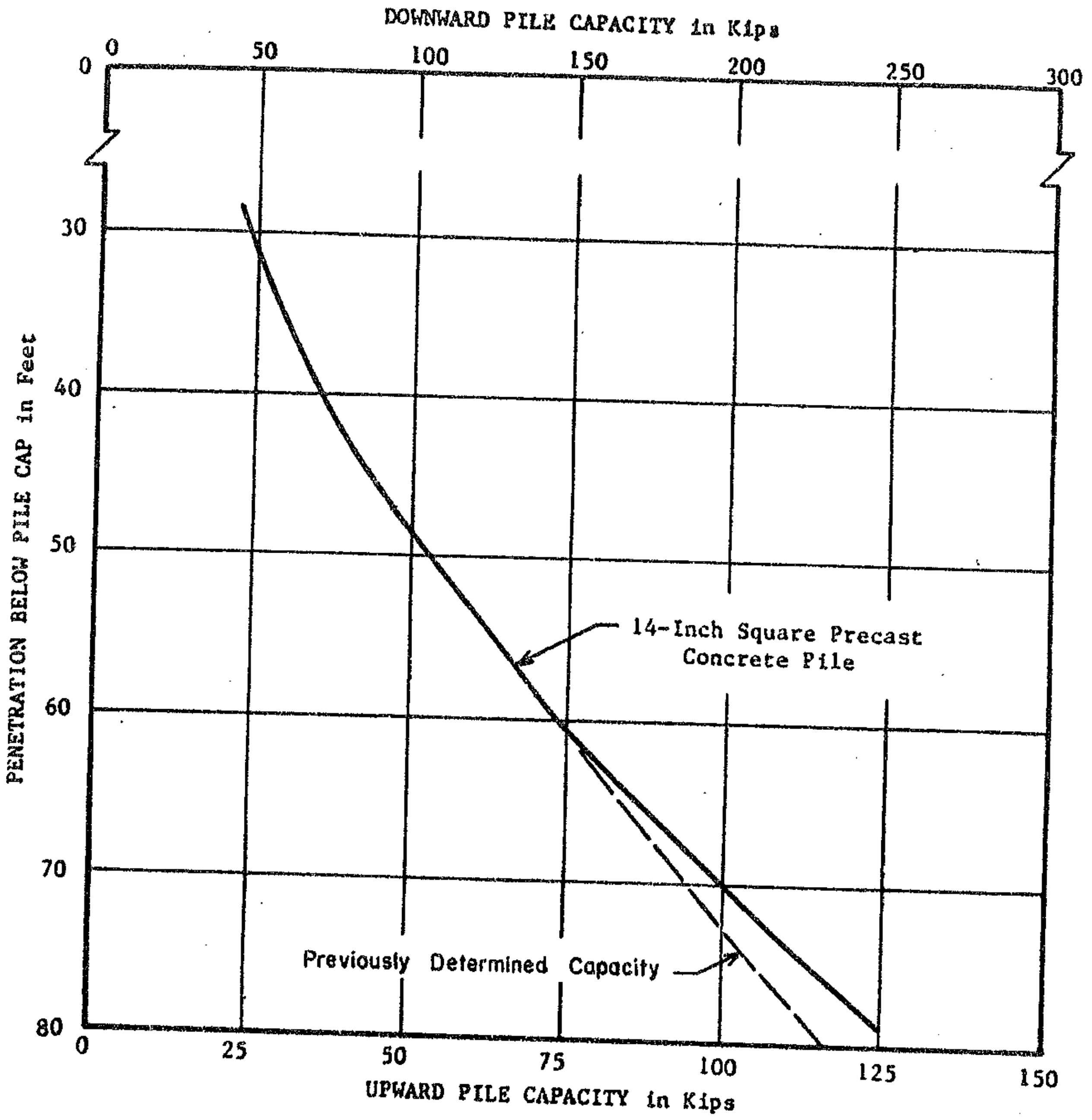
FORM 120

LeROY CRANDALL AND ASSOCIATES

PLATE 1

JOB A-36021/6 DATE 6/26/85 DR RL O.E. VB W.P. ge CHKD VB

0200436



NOTES: (1) The indicated values refer to the total of dead plus live loads; a one-third increase may be used when considering wind or seismic loads.

(2) Piles in groups should be spaced a minimum of 2½ diameters, but not less than 3 feet, on centers.

(3) The indicated values are based on the strength of the soils; the actual pile capacities may be limited to lesser values by the strength of the piles.

DRIVEN PILE CAPACITIES

LeROY CRANDALL AND ASSOCIATES consulting geotechnical engineers, 711 n. alvarado st., los angeles, ca. 90026, (213)413-3650, telex 69-0376

7061

September 23, 1985

Wilshire Courtyard
5757 Wilshire Boulevard
Los Angeles, California 90036

(Our Job Nos. A-84021
& C-850069)

Attention: Mr. William Lieberman

Tract 5798
Lot 1-8, 44-52, 290, 291
5750 Wilshire Blvd

Gentlemen:

Lateral Earth Pressure on North Basement Wall
Proposed Wilshire Courtyard
Wilshire Boulevard between Masselin
and Curson Avenues
Los Angeles, California

This letter presents the recommended lateral earth pressure for a temporary condition where backfill will be placed against the lower portion of the north basement wall. The basement wall is currently under construction. Current plans are to complete the lower 20 feet of the basement wall and the lowest supported basement floor slab as soon as possible. Backfill will be placed between the basement wall and the shored excavation along Wilshire Boulevard to provide permanent support for the excavation adjacent to Wilshire Boulevard.

There will be a temporary condition, prior to completion of the lower supported floor, where the lower portion of the wall will be backfilled and the backfill will support a 1:1 slope. For design of this cantilevered basement wall, it may be assumed that the soils will exert an active pressure equal to that developed by a fluid with a density of 60 pounds per cubic foot.

Yours very truly,

LeROY CRANDALL AND ASSOCIATES

by James L. Van Beveren
James L. Van Beveren, R.C.E. 17785
Project Engineer

B7/bmc
(2 copies submitted)

cc: (2) McLarand Vasquez and Partners, Inc.
(1) Erkel, Greenfield and Associates
(2) M. H. Golden Company (Job Site)
(2) City of Los Angeles, Department of
Building and Safety Plan Check
Attn: Mr. Abe Hibashi, Rm. 422
(2) B-85086 File

APPENDIX D

REPORT OF SUSPENSION PS VELOCITIES BY GEOVISION

APPENDIX D

PS SUSPENSION VELOCITIES
WILSHIRE BOULEVARD
LOS ANGELES, CALIFORNIA

Prepared for

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March 12, 2020

Report 19545-01 rev 0

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APPENDICES

INTRODUCTION

GEO*Vision* acquired PS Suspension velocity data in one uncased borehole at 5700 Wilshire Boulevard in Los Angeles, California for the GEOCON West . A GEOVision Professional Geophysicist or Engineer reviewed fieldwork, data analysis, and report preparation. A summary of the instrumentation, methods, data analysis, and results follow.

SCOPE OF WORK

This report presents the results of PS Suspension velocity data acquired in one borehole on Sunday February 16, 2020, 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. Additionally, the soil site class was determined in the upper 30 meters (Vs30) using the NEHRP method.

The OYO PS Suspension Logging System was used to obtain in-situ horizontal shear (S_H), and compressional (P) wave velocity measurements in a cased boreholes 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 PS Suspension 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 PS suspension logging system, manufactured by OYO Corporation, and their subsidiary, Robertson Geo (RG). 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 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, the 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. The 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 typical 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 the 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 PS Suspension 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.

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 PS Suspension 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

The borehole was logged uncased filled with fluid to the surface. Measurements followed the **GEO***Vision* Procedure for PS Suspension Seismic Velocity Logging, revision 1.5. Before logging, the top of the probe was positioned 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. Data from each depth were viewed on the computer display, checked, and saved to disk before moving to the next depth.

Upon completion, 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

The recorded digital waveforms were analyzed to locate the most prominent first minima, first maxima, or the 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 template to complete the velocity calculations based on the arrival time picks. The Microsoft Excel® analysis files accompany this report.

The P-wave velocity over the 6.3-foot interval from source to receiver 1 (S-R1) was also picked, calculated, and plotted 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 the acceleration of the solenoid before the 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 actuation.

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 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 the S_H-waveform record with an 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 the 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.

Vs30 Analysis

The average shear wave velocity in the upper 30 meters (Vs30) was calculated using the NEHRP method. The PS Suspension logger measures directly the travel time over a 1 meter interval. However, data are logged at ½ meter intervals. The overlapped measurements (at nominal 0.5 m intervals) are overlapping travel times. It is not explicitly correct to use these as representing individual 0.5 m interval velocities. As a result, it is necessary to interpolate to obtain a distance-weighted average Vs value at each 1 m interval. These are then used to calculate the interval times, which are then accumulated to obtain the total travel time over 30 m. Vs30 is 30 m divided by this total travel time.

RESULTS

Suspension Velocity Results

Suspension R1-R2 P- and S_H-wave velocities for borehole B-2 are plot in Figure 4, and data are compiled in Table 3. The associated Microsoft Excel® analysis files accompany this report. Included in the analysis files are Poisson's Ratio calculations, tabulated data, and plots.

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. Note 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 compiled in Table A-1.

Vs30 Results

The Vs30 value for Wilshire Blvd Boring B-1 is 283 meters/second, characterizing it as NEHRP site class D.*

* Site Classifications are taken from Table 1615 1.1 Site Class Definitions published in 2000 International Building code, International Code Council, Inc. on page 350

SUMMARY

Discussion of Suspension Velocity Results

Suspension PS velocity data were collected in an uncased fluid-filled borehole. Field documentation indicate the presence of hydrocarbons, tar, in the borehole below approximately 30 ft. Possible effects of the hydrocarbon on the data are noted below.

Suspension PS velocity data quality are judged on five criteria:

	Criteria	B-1
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 Full saturation appears to occur at about 57ft BGS. However, the Geocon log does not show saturation until below 104 ft. There is unusual behavior of the P-wave response between 15 and 45 ft BGS. The P-wave velocity (and Poisson's ratio) is higher than would be expected for native formation, but not high enough to be considered saturated. The influence of tar on the P-wave velocity of sands is not well studied.
4	Clarity of P-wave and S_H-wave onset, as well as damping of later oscillations.	This is acceptable data. There was significant noise near the surface that also showed up in deeper measurements. Also, usually P-wave data below water table is characterized by higher frequency response, but this is not the case in this hole. Again, the presence of tar may have affected this response.
5	Consistency of profile between adjacent borings, if available.	Not applicable

Quality Assurance

These borehole geophysical measurements were performed using industry-standard or better methods for measurements and 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 an estimated precision of +/- 5%. Depth indications are very reliable, with an 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 or Engineer.

Prepared by:

J Jordan

3/12/2020

Jonathan J Jordan Date
GEO*Vision* Geophysical Services

Reviewed and approved by

VICTOR GONZALEZ
PGp 1074

3/12/2020

Victor M Gonzalez Date
California Professional Geophysicist PGp 1074
GEO*Vision* Geophysical Services

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

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

Table 1. Borehole Logging Dates and Coordinates

BOREHOLE NUMBER	LOGGING DATE	COORDINATES (1) (US Survey Feet)		
		Elevation	Northing	Easting
B-1	02/16/2020			

(1) Coordinates unavailable at the time of the report preparation

Table 2. Logging Tool, Depth Range and Sample Interval

BOREHOLE NUMBER	TOOL AND RUN NUMBER	DEPTH RANGE (FEET)	OPEN HOLE (FEET)	SAMPLE INTERVAL (FEET)
B-1	SUSPENSION DOWN01	6.6 – 136.2	150	1.6

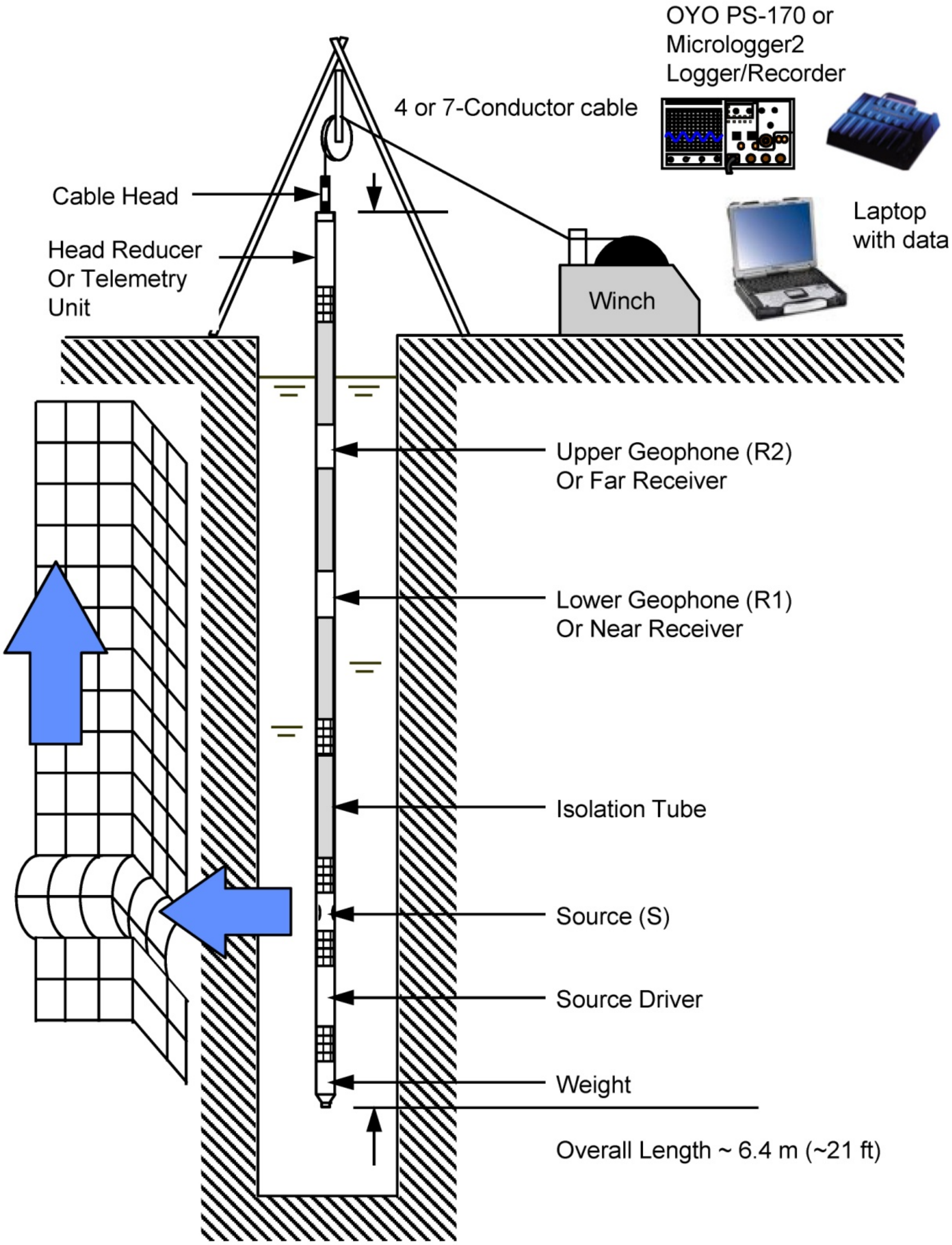

Figure 1: Concept illustration of PS logging system

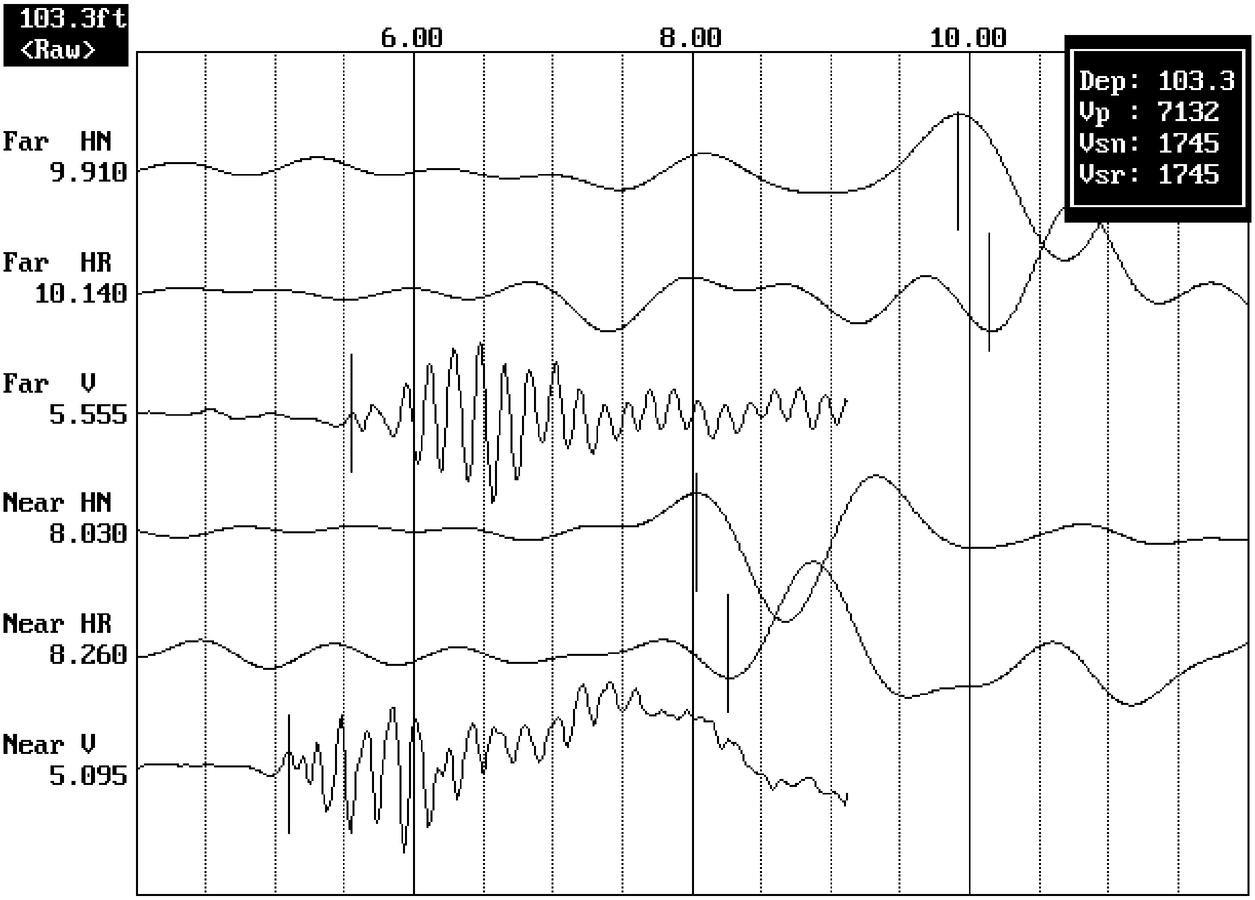

Figure 2: Example of filtered (1400 Hz lowpass) suspension record

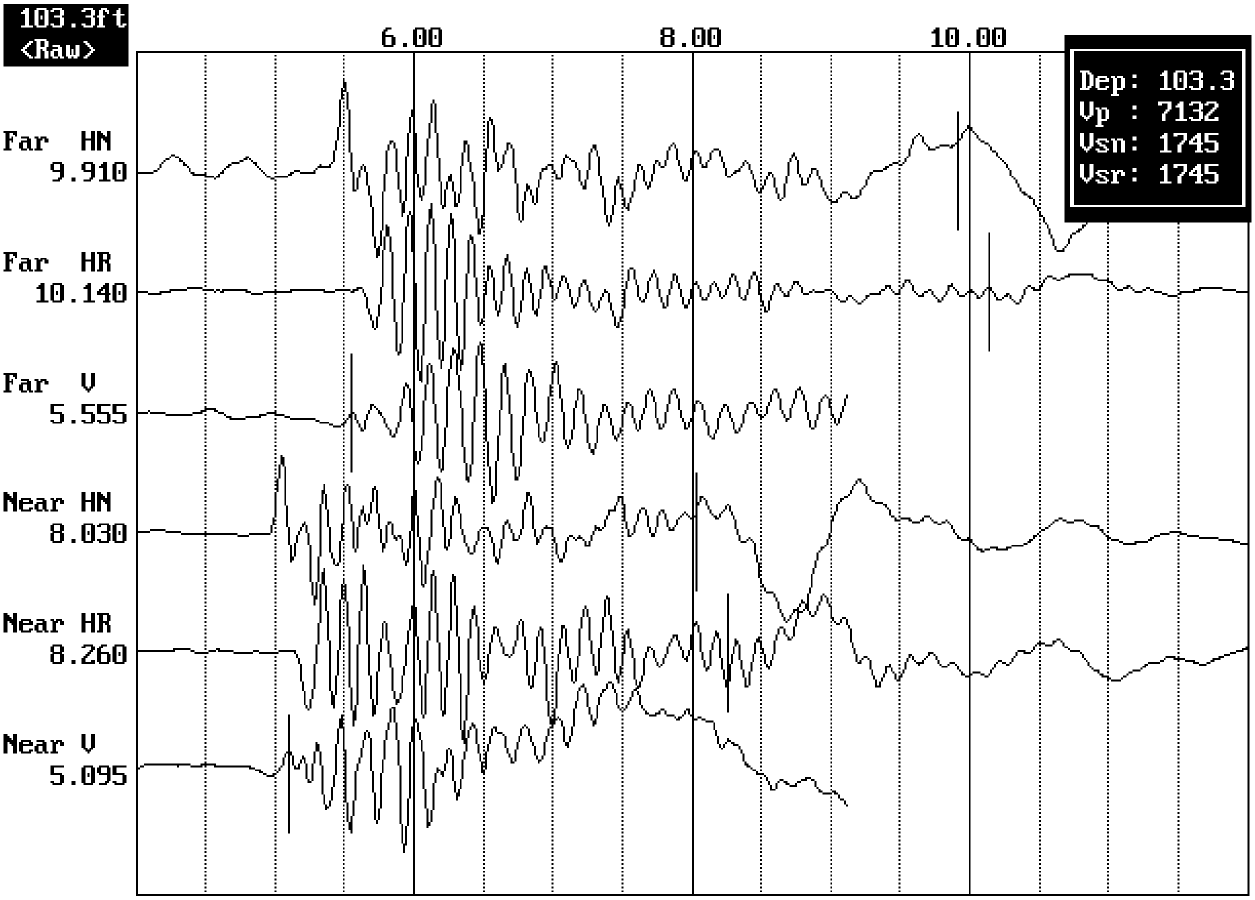

Figure 3. Example of unfiltered suspension record

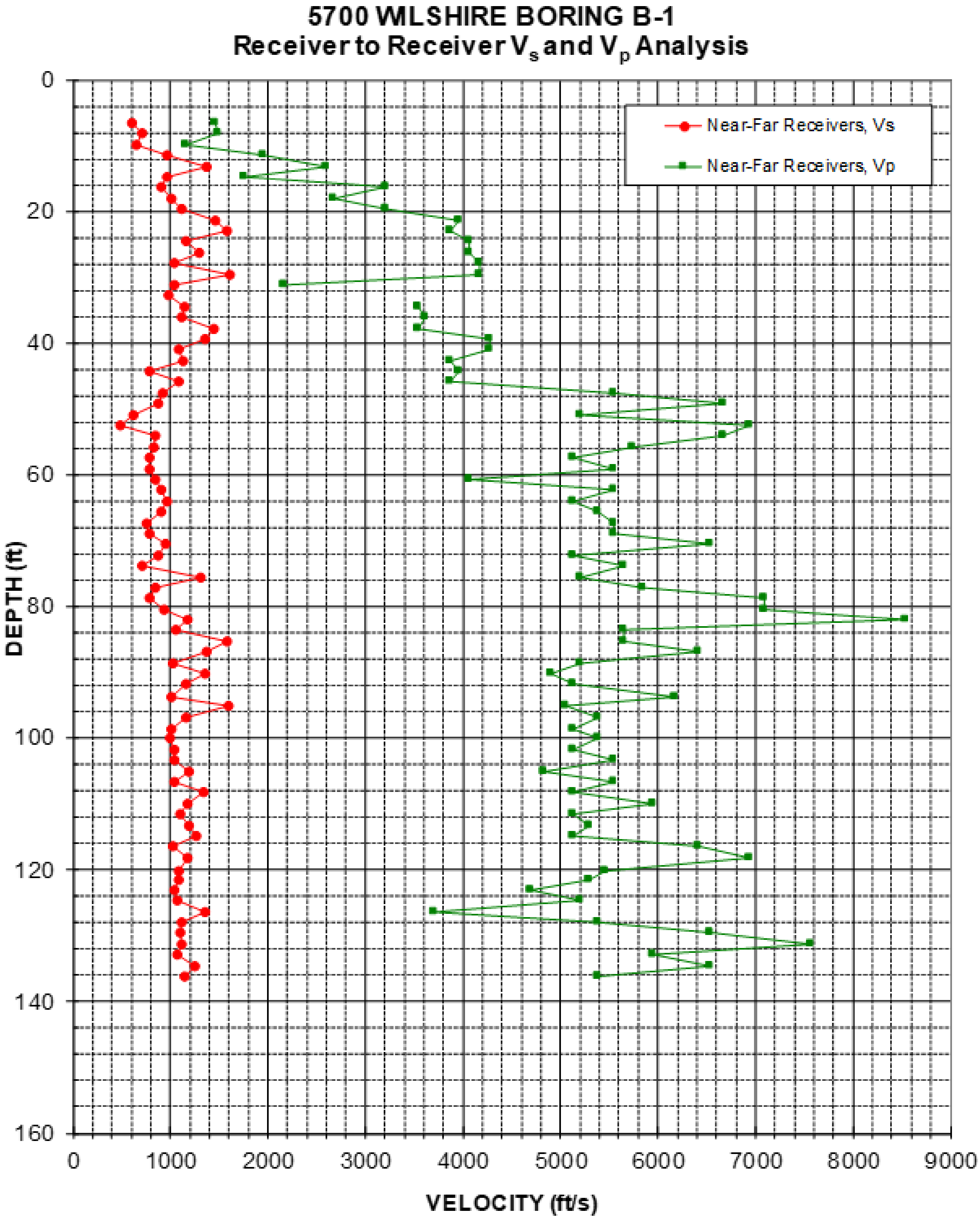

Figure 4: Borehole B-1, Suspension R1-R2 P- and S_H-wave velocities

Table 3. Borehole B-1, 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-1

American Units				Metric Units			
Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio	Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio
	V_s	V_p			V_s	V_p	
(ft)	(ft/s)	(ft/s)		(m)	(m/s)	(m/s)	
4.9	-	-	-	1.5	-	-	-
6.6	610	1460	0.40	2.0	180	450	0.40
8.2	710	1490	0.35	2.5	220	450	0.35
9.8	650	1170	0.28	3.0	200	360	0.28
11.5	970	1950	0.33	3.5	300	590	0.33
13.1	1370	2600	0.31	4.0	420	790	0.31
14.8	970	1750	0.28	4.5	300	530	0.28
16.4	900	3210	0.46	5.0	270	980	0.46
18.0	1010	2670	0.42	5.5	310	810	0.42
19.7	1110	3210	0.43	6.0	340	980	0.43
21.3	1470	3970	0.42	6.5	450	1210	0.42
23.0	1590	3880	0.40	7.0	480	1180	0.40
24.6	1160	4070	0.46	7.5	350	1240	0.46
26.3	1290	4070	0.44	8.0	390	1240	0.44
27.9	1040	4170	0.47	8.5	320	1270	0.47
29.5	1610	4170	0.41	9.0	490	1270	0.41
31.2	1030	2160	0.35	9.5	320	660	0.35
32.8	990	-	-	10.0	300	-	-
34.5	1140	3550	0.44	10.5	350	1080	0.44
36.1	1120	3620	0.45	11.0	340	1100	0.45
37.7	1450	3550	0.40	11.5	440	1080	0.40
39.4	1360	4270	0.44	12.0	410	1300	0.44
41.0	1090	4270	0.47	12.5	330	1300	0.47
42.7	1130	3880	0.45	13.0	340	1180	0.45
44.3	780	3970	0.48	13.5	240	1210	0.48
45.9	1090	3880	0.46	14.0	330	1180	0.46
47.6	930	5560	0.49	14.5	280	1690	0.49
49.2	880	6670	0.49	15.0	270	2030	0.49
50.9	630	5210	0.49	15.5	190	1590	0.49
52.5	490	6940	0.50	16.0	150	2120	0.50
54.1	840	6670	0.49	16.5	260	2030	0.49
55.8	840	5750	0.49	17.0	260	1750	0.49
57.4	790	5130	0.49	17.5	240	1560	0.49
59.1	780	5560	0.49	18.0	240	1690	0.49
60.7	840	4070	0.48	18.5	260	1240	0.48
62.3	910	5560	0.49	19.0	280	1690	0.49
64.0	960	5130	0.48	19.5	290	1560	0.48
65.6	900	5380	0.49	20.0	270	1640	0.49

Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio Based on Receiver-to-Receiver Travel Time Data - Borehole B-1

American Units				Metric Units			
Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio	Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio
	V_s	V_p			V_s	V_p	
(ft)	(ft/s)	(ft/s)		(m)	(m/s)	(m/s)	
67.3	750	5560	0.49	20.5	230	1690	0.49
68.9	790	5560	0.49	21.0	240	1690	0.49
70.5	950	6540	0.49	21.5	290	1990	0.49
72.2	880	5130	0.48	22.0	270	1560	0.48
73.8	720	5650	0.49	22.5	220	1720	0.49
75.5	1310	5210	0.47	23.0	400	1590	0.47
77.1	840	5850	0.49	23.5	260	1780	0.49
78.7	790	7090	0.49	24.0	240	2160	0.49
80.4	930	7090	0.49	24.5	280	2160	0.49
82.0	1180	8550	0.49	25.0	360	2610	0.49
83.7	1060	5650	0.48	25.5	320	1720	0.48
85.3	1580	5650	0.46	26.0	480	1720	0.46
86.9	1370	6410	0.48	26.5	420	1950	0.48
88.6	1030	5210	0.48	27.0	310	1590	0.48
90.2	1360	4900	0.46	27.5	410	1490	0.46
91.9	1170	5130	0.47	28.0	360	1560	0.47
93.8	1000	6170	0.49	28.6	310	1880	0.49
95.1	1590	5050	0.44	29.0	490	1540	0.44
96.8	1160	5380	0.48	29.5	350	1640	0.48
98.8	1020	5130	0.48	30.1	310	1560	0.48
100.1	1000	5380	0.48	30.5	310	1640	0.48
101.7	1040	5130	0.48	31.0	320	1560	0.48
103.4	1050	5560	0.48	31.5	320	1690	0.48
105.0	1190	4830	0.47	32.0	360	1470	0.47
106.6	1040	5560	0.48	32.5	320	1690	0.48
108.3	1330	5130	0.46	33.0	410	1560	0.46
109.9	1170	5950	0.48	33.5	360	1810	0.48
111.6	1110	5130	0.48	34.0	340	1560	0.48
113.2	1190	5290	0.47	34.5	360	1610	0.47
114.8	1270	5130	0.47	35.0	390	1560	0.47
116.5	1030	6410	0.49	35.5	310	1950	0.49
118.1	1170	6940	0.49	36.0	360	2120	0.49
120.1	1080	5460	0.48	36.6	330	1670	0.48
121.4	1090	5290	0.48	37.0	330	1610	0.48
123.0	1050	4690	0.47	37.5	320	1430	0.47
124.7	1070	5210	0.48	38.0	330	1590	0.48
126.3	1360	3700	0.42	38.5	410	1130	0.42

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

APPENDIX A

SUSPENSION VELOCITY MEASUREMENT QUALITY ASSURANCE SUSPENSION SOURCE TO RECEIVER ANALYSIS RESULTS

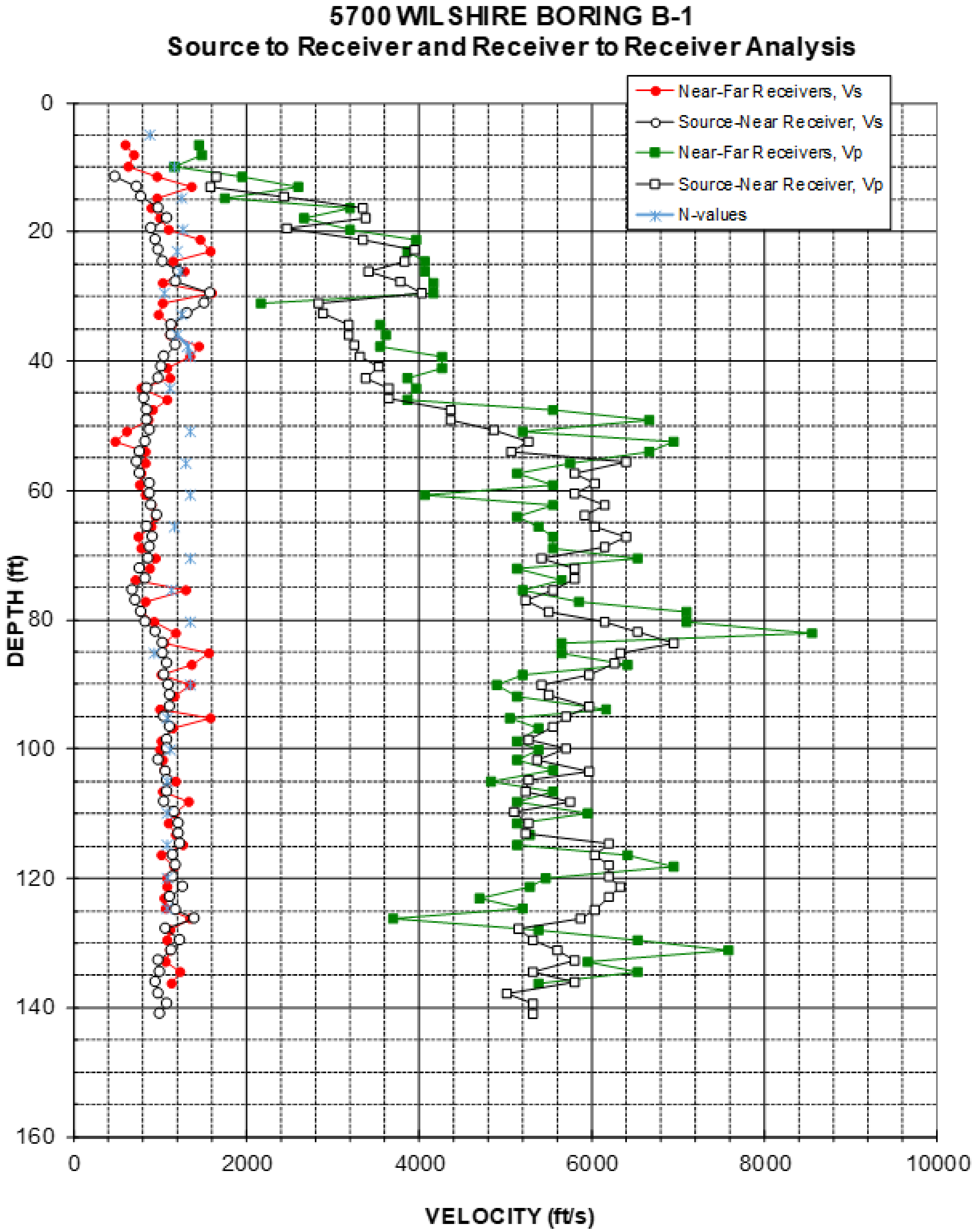

Figure A-1: Borehole B-1, Suspension S-R1 P- and SH-wave velocities

Table A-1. Borehole B-1, 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-1

American Units			
Depth at Midpoint Between Source and Near Receiver	Velocity		Poisson's Ratio
	V_s	V_p	
(ft)	(ft/s)	(ft/s)	
9.8	-	-	-
11.4	490	1650	0.45
13.0	730	1590	0.36
14.7	800	2440	0.44
16.3	1000	3350	0.45
18.0	1090	3390	0.44
19.6	900	2460	0.42
21.2	960	3350	0.46
22.9	990	3960	0.47
24.5	1040	3840	0.46
26.2	1230	3420	0.43
27.8	1190	3790	0.44
29.4	1580	4030	0.41
31.1	1530	2840	0.30
32.7	1320	2890	0.37
34.4	1140	3180	0.43
36.0	1140	3180	0.43
37.6	1190	3250	0.42
39.3	1060	3310	0.44
40.9	1020	3540	0.45
42.6	990	3390	0.45
44.2	860	3660	0.47
45.8	820	3660	0.47
47.5	850	4370	0.48
49.1	860	4370	0.48
50.8	890	4870	0.48
52.4	840	5280	0.49
54.0	770	5060	0.49
55.7	740	6390	0.49
57.3	770	5810	0.49
59.0	890	6030	0.49
60.6	890	5810	0.49
62.2	910	6150	0.49
63.9	970	5920	0.49
65.5	850	6030	0.49
67.2	920	6390	0.49
68.8	890	6150	0.49
70.5	870	5410	0.49

Metric Units			
Depth at Midpoint Between Source and Near Receiver	Velocity		Poisson's Ratio
	V_s	V_p	
(m)	(m/s)	(m/s)	
3.0	-	-	-
3.5	150	500	0.45
4.0	220	480	0.36
4.5	240	740	0.44
5.0	300	1020	0.45
5.5	330	1030	0.44
6.0	270	750	0.42
6.5	290	1020	0.46
7.0	300	1210	0.47
7.5	320	1170	0.46
8.0	370	1040	0.43
8.5	360	1160	0.44
9.0	480	1230	0.41
9.5	460	870	0.30
10.0	400	880	0.37
10.5	350	970	0.43
11.0	350	970	0.43
11.5	360	990	0.42
12.0	320	1010	0.44
12.5	310	1080	0.45
13.0	300	1030	0.45
13.5	260	1120	0.47
14.0	250	1120	0.47
14.5	260	1330	0.48
15.0	260	1330	0.48
15.5	270	1480	0.48
16.0	260	1610	0.49
16.5	240	1540	0.49
17.0	220	1950	0.49
17.5	230	1770	0.49
18.0	270	1840	0.49
18.5	270	1770	0.49
19.0	280	1870	0.49
19.5	300	1800	0.49
20.0	260	1840	0.49
20.5	280	1950	0.49
21.0	270	1870	0.49
21.5	270	1650	0.49

Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio
Based on Source-to-Receiver Travel Time Data - Borehole B-1

American Units			
Depth at Midpoint Between Source and Near Receiver	**Velocity**		**Poisson's Ratio**
	V_s	**V_p**	
(ft)	(ft/s)	(ft/s)	
72.1	770	5810	0.49
73.7	840	5810	0.49
75.4	700	5550	0.49
77.0	730	5230	0.49
78.7	790	5500	0.49
80.3	830	6150	0.49
81.9	950	6530	0.49
83.6	1030	6960	0.49
85.2	1050	6330	0.49
86.9	1090	6270	0.48
88.5	1060	5970	0.48
90.1	1100	5410	0.48
91.8	1120	5500	0.48
93.4	1120	5970	0.48
95.1	1060	5700	0.48
96.7	1120	5550	0.48
98.7	1090	5280	0.48
100.0	1090	5700	0.48
101.6	990	5360	0.48
103.6	1080	5970	0.48
104.9	1090	5280	0.48
106.5	1100	5230	0.48
108.2	1060	5750	0.48
109.8	1170	5100	0.47
111.5	1230	5280	0.47
113.1	1220	5230	0.47
114.7	1240	6210	0.48
116.4	1150	6030	0.48
118.0	1190	6210	0.48
119.7	1150	6210	0.48
121.3	1280	6330	0.48
122.9	1120	6210	0.48
124.9	1190	6030	0.48
126.2	1400	5860	0.47
127.9	1070	5150	0.48
129.5	1230	5320	0.47
131.1	1130	5600	0.48
132.8	1000	5810	0.48
134.4	1010	5320	0.48

Metric Units			
Depth at Midpoint Between Source and Near Receiver	**Velocity**		**Poisson's Ratio**
	V_s	**V_p**	
(m)	(m/s)	(m/s)	
22.0	230	1770	0.49
22.5	250	1770	0.49
23.0	210	1690	0.49
23.5	220	1590	0.49
24.0	240	1680	0.49
24.5	250	1870	0.49
25.0	290	1990	0.49
25.5	310	2120	0.49
26.0	320	1930	0.49
26.5	330	1910	0.48
27.0	320	1820	0.48
27.5	340	1650	0.48
28.0	340	1680	0.48
28.5	340	1820	0.48
29.0	320	1740	0.48
29.5	340	1690	0.48
30.1	330	1610	0.48
30.5	330	1740	0.48
31.0	300	1640	0.48
31.6	330	1820	0.48
32.0	330	1610	0.48
32.5	330	1590	0.48
33.0	320	1750	0.48
33.5	360	1560	0.47
34.0	370	1610	0.47
34.5	370	1590	0.47
35.0	380	1890	0.48
35.5	350	1840	0.48
36.0	360	1890	0.48
36.5	350	1890	0.48
37.0	390	1930	0.48
37.5	340	1890	0.48
38.1	360	1840	0.48
38.5	430	1790	0.47
39.0	330	1570	0.48
39.5	380	1620	0.47
40.0	350	1710	0.48
40.5	300	1770	0.48
41.0	310	1620	0.48

Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio Based on Source-to-Receiver Travel Time Data - Borehole B-1

American Units			
Depth at Midpoint Between Source and Near Receiver	**Velocity**		**Poisson's Ratio**
	V_s	**V_p**	
(ft)	(ft/s)	(ft/s)	
136.1	950	5810	0.49
137.7	990	5020	0.48
139.3	1080	5320	0.48
141.0	1000	5320	

Metric Units			
Depth at Midpoint Between Source and Near Receiver	**Velocity**		**Poisson's Ratio**
	V_s	**V_p**	
(m)	(m/s)	(m/s)	
41.5	290	1770	0.49
42.0	300	1530	0.48
42.5	330	1620	0.48
43.0	300	1620	0.48

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

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: May 28, 2019 **Cert No. 551220083036897**

Customer:

GEOVISION

1124 OLYMPIC DRIVE
CORONA CA 92881

Work Order #: LA-90043197
Purchase Order #: 19160-190520-01
Serial Number: 160023
Department: N/A
Performed By: TYLER MCKEEN
Received Condition: IN TOLERANCE
Returned Condition: IN TOLERANCE
Cal. Date: May 24, 2019
Cal. Interval: 12 MONTHS
Cal. Due Date: May 24, 2020

MPC Control #: AM6767
Asset ID: 160023
Gage Type: LOGGER
Manufacturer: OYO
Model Number: 3403
Size: N/A
Temp/RH: 22.5°C / 42.9%
Location: Calibration performed at MPC facility

Calibration Notes:

See attached data sheet for calculations. (1 Page)

Calibrated IAW customer supplied data form Rev 2.1
Frequency measurement uncertainty = 0.0005 Hz
Unit calibrated with Laptop Panasonic Model CF-29,s/n: 6AKSB01291 and RG Micrologger II Serial No. 5772
Calibrated To 4:1 Accuracy Ratio

Calibration performed in accordance with approved GEOVision calibration procedures included in work Instruction No. 06
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	Apr 30, 2021	551220083021224
LAS0018	ARB / FUNC GENERATOR	33250A	US40001522	AGILENT	Apr 30, 2020	551220083009506
BD7715	UNIVERSAL COUNTER	53131A	3416A05377	HEWLETT PACKARD	Apr 30, 2020	551220082934517

Calibrating Technician:

TYLER MCKEEN

QC Approval:

Ilya Vaks

ILYA VAKS

Statements of Pass or Fail Conformance: The uncertainty of measurement has been taken into account when determining compliance with specification, as per ILAC-G8:03/2009. All measurements and test results guard banded to ensure the probability of false-accept does not exceed 2% in compliance with ANSI/NCSL Z540.3-2006.

The status of compliance with the acceptance criteria is reported as:
PASS - Compliant with specification; .
FAIL - Not compliant with specification.
FAIL^{z} - The measured value is not within the acceptance limits. However, a portion of the expanded uncertainty of measurement at 95% is within the specified tolerance.
PASS^{z} - The measured value is within acceptance limits. However, a portion of the expanded uncertainty of measurement at 95% exceeds the specified tolerance.

The expanded uncertainty of measurement is stated as the standard uncertainty of measurement multiplied by the coverage factor k=2, which for a normal distribution corresponds to a coverage probability of approximately 95%, unless otherwise stated. This calibration report complies with ISO/IEC 17025:2017 and ANSI/NCSL Z540.3 Method 6-Guard Bands based on Test Uncertainty Ratio. 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. 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. The information on this report pertains only to the instrument identified. this may not be reproduced in part or in a whole without the prior written approval of the issuing MP Calibration Laboratory.

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

AC-1969.03

Certificate of Calibration

Date: **May 28, 2019**

Cert No. **551220083036897**

Procedures Used in this Event

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

Calibrating Technician:

TYLER MCKEEN

QC Approval:

Ilya Vaks

ILYA VAKS

Statements of Pass or Fail Conformance: The uncertainty of measurement has been taken into account when determining compliance with specification, as per ILAC-G8:03/2009. All measurements and test results guard banded to ensure the probability of false-accept does not exceed 2% in compliance with ANSI/NCSL Z540.3-2006.

The status of compliance with the acceptance criteria is reported as:

PASS - Compliant with specification; .

FAIL - Not compliant with specification.

FAIL^z - The measured value is not within the acceptance limits. However, a portion of the expanded uncertainty of measurement at 95% is within the specified tolerance.

PASS^z - The measured value is within acceptance limits. However, a portion of the expanded uncertainty of measurement at 95% exceeds the specified tolerance.

The expanded uncertainty of measurement is stated as the standard uncertainty of measurement multiplied by the coverage factor k=2, which for a normal distribution corresponds to a coverage probability of approximately 95%, unless otherwise stated. This calibration report complies with ISO/IEC 17025:2017 and ANSI/NCSL Z540.3 Method 6-Guard Bands based on Test Uncertainty Ratio. 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. 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. The information on this report pertains only to the instrument identified, this may not be reproduced in part or in a whole without the prior written approval of the issuing MP Calibration Laboratory.

GEOVision
geophysical services

SUSPENSION PS SEISMIC LOGGER/RECORDER CALIBRATION DATA FORM

INSTRUMENT DATA

System mfg.: OYO
Model no.: 3403
Serial no.: 160023
Calibration date: 5/24/2019
By: Micro Precision
Due date: 5/24/2020

Counter mfg.: Hewlett Packard
Model no.: 53131A
Serial no.: 3416A05377
Calibration date: 4/02/2019
By: Micro Precision
Due date: 4/30/2020

Signal generator mfg.: Agilent
Model no.: 33250A
Serial no.: US40001527
Calibration date: 4/03/2019
By: Micro Precision
Due date: 4/30/2020

Laptop controller mfg.: Panasonic
Model no.: CF-29
Serial no.: 6AKSB01291
Calibration date: N/A

SYSTEM SETTINGS:

Gain: 0
Filter: Low Pass 1K
Range: 5 - 200
Delay: 0
Stack (1 std): 1
System date = correct date and time: ✓ 12:48 PM 5/24/2019

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 PS 4.00
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 0.22% As left 0.22%

Target Frequency (Hz)	Actual Frequency (Hz)	Sample Period (microS)	File Name	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	201	180.2	49.94	179.6	50.11	180.2	49.94
100.0	100.0	100	202	90.00	100.0	90.00	100.0	90.10	99.90
200.0	200.0	50	203	45.00	200.0	~~20.~~ 45.00	200.0	45.00	200.0
500.0	500.0	20	204	18.00	500.0	18.00	500.0	17.98	500.6
1000	1000	10	205	9.010	999.0	9.000	1000	8.990	1001
2000	2000	5	206	4.500	2000	4.490	2004	4.500	2002

Calibrated by: Tyler McKeen 5/24/19
Name Date Signature

Witnessed by: Emily Feldman 5/24/19
Name Date Signature

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