GEOTECHNICAL INVESTIGATION

PROPOSED MIXED-USE DEVELOPMENT
THE SOUTHEAST CORNER OF 3RD STREET AND FAIRFAX AVENUE
LOS ANGELES, CALIFORNIA

TRACT 215, LOT 12, ARB 1 & 2

GEOCON
WEST, INC.
GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

HOLLAND ACQUISITION CO, LLC
LOS ANGELES, CALIFORNIA

PROJECT NO. A9713-06-01

REVISED NOVEMBER 16, 2018

GEOCON
WEST, INC.
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. A9713-06-01
Revised November 16, 2018

Mr. Shaun Evans
Holland Acquisition Co., LLC
731 South Spring Street, Suite 202
Los Angeles, California 90014

Subject: GEOTECHNICAL INVESTIGATION
PROPOSED MIXED-USE DEVELOPMENT
3rd STREET AND FAIRFAX AVENUE
LOS ANGELES, CALIFORNIA
TRACT 215, LOT 12, ARB 1 & 2

Dear Mr. Evans:

In accordance with your authorization of our proposal dated December 15, 2017, we have performed a geotechnical investigation for the proposed mixed-use development located at the southeast corner of 3rd Street and Fairfax Avenue 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.

Raymond Antoine	Jelisa Thomas Adams	Susan F. Kirkgard
Staff Geologist	GE 3092	CEG 1754

(EMAIL) Addressee

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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a geotechnical investigation for the proposed mixed-use development located at the southeast corner of 3rd Street and Fairfax Avenue 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 design and construction.

The scope of this investigation included review of a previous geotechnical investigation report for the site (Krazan & Associates, Inc., 2017), a site reconnaissance, field exploration, laboratory testing, engineering analysis, and the preparation of this report. The site was explored on January 2 and 3, 2018, by excavating six 8-inch diameter borings to depths ranging from approximately 15½ to 100½ feet below the existing ground surface utilizing a truck-mounted hollow-stem auger drilling machine. The approximate locations of the exploratory borings are depicted on the Site Plan (see Figure 2). A detailed discussion of the field investigation, including boring logs, 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 analysis of the data obtained during the investigation and our experience with similar soil and geologic conditions. 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 the southeast corner of 3rd Street and Fairfax Avenue in the City of Los Angeles, California. The site is a rectangular-shaped parcel and is currently occupied by a two-story commercial structure with a basement level and an associated asphalt-paved parking lot. The site is bounded by 3rd Street to the north, by an existing commercial development and then Fairfax Avenue to the west, by South Ogden Drive to the east, and by an alleyway and then Hancock Park Elementary School to the south. The site is relatively level, with no pronounced highs or lows. Surface water drainage at the site appears to be by sheet flow along the existing ground contours to the city streets.

Based on the information provided by the Client, it is our understanding that the existing structure will be demolished. The proposed development will consist of a mid-rise structure comprised of eight-stories of residential, retail, and parking all underlain by two levels of subterranean parking. The proposed subterranean parking levels are anticipated to extend to depths of approximately 30 to 35 feet, including foundation depths. The proposed development is depicted on the Site Plan and Cross Section (see Figures 2 and 3).

Preliminary wall and column loads were provided by Bryson Markulin Zickmantel Structural Engineers, the project structural engineers. It is anticipated that column loads will be up to 980 and 1,120 kips (dead + live loads) for the south and north portions of the project, respectively. It is anticipated that wall loads will be up to 10 kips per linear foot (dead + live loads).

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. At this time, the report contains sufficient information to adequately inform impact analysis pursuant to the California Environmental Quality Act (CEQA). 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. PRIOR INVESTIGATION

Krazan & Associates, Inc. (Krazan) performed a prior geotechnical report for the subject site titled:

> *Preliminary Geotechnical Engineering Investigation, Proposed Mixed Use Development, SEC of 3rd and Fairfax Avenue, Los Angeles, California*, prepared by Krazan & Associates, Inc., dated July 31, 2017.

The prior geotechnical study addressed the design and construction of six-stories of mixed-use space over three levels of subterranean parking, and included the adjacent parcel to the west. Krazan excavated 11 borings to depths ranging from approximately 4 to 80 feet below the ground surface. The locations of the prior borings which fall within the subject site's boundaries are shown on our Site Plan (see Figure 2). The report and boring logs do not differentiate artificial fill and native soils; therefore, it is unknown how much fill, if any, the prior borings encountered. Groundwater was encountered at depths ranging from 20 to 22 feet below the existing grade. Laboratory testing consisting of direct shear, grain size distribution, consolidation, expansion index, and corrosivity was performed. The near surface soils were found to have a medium expansion potential. Near-surface infiltration testing was also performed.

We have reviewed the report by Krazan & Associates, Inc. (2017). It is relevant, to an extent, for this report as we have reviewed the boring logs and laboratory data contained therein. However, we do not concur with the conclusions and recommendations presented therein as they prepared for a different project scope and are not applicable to the currently proposed project. Geocon assumes responsibility for the utilization of the exploration and laboratory data, as it relates to the characteristics of the soils at the site, presented within the geotechnical report by Krazan. A copy of the prior report is provided in Appendix C.

Geocon West, Inc. is the Geotechnical Consultant of Record and will be providing all necessary geotechnical consultation, plan review, design recommendations, inspection and testing services for this project. Where differing, the recommendations presented herein supersede all previous recommendations.

4. GEOLOGIC SETTING

The site is located in the northern 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 Newport-Inglewood Fault Zone located approximately 2.0 miles to the southwest.

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 alluvium in the northwest portion of the site (California Geological Survey [CGS], 2012). The southeast portion of the site is underlain by artificial fill and Holocene age alluvium that is in turn underlain by Pleistocene age alluvium at depth (CGS, 2012). Detailed stratigraphic profiles 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 5½ feet below existing ground surface. The artificial fill generally consists of brown dark brown or grayish brown sand, silty sand, sandy silt, and silt. The artificial fill is characterized as slightly moist and soft or very loose to 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 Alluvium

Quaternary age alluvium was encountered beneath the fill. The alluvium consists of brown to dark brown, grayish brown, olive brown, light gray to dark gray, or yellowish brown to dark yellowish brown interbedded sand, sand with silt, silty sand, clayey sand, clay, clayey silt, silt, and sandy silt. The alluvial soils are primarily fine- to medium-grained, slightly moist to wet, and very loose to very dense or soft to stiff.

6. GROUNDWATER

Review of the Seismic Hazard Zone Report for the Hollywood Quadrangle (California Division of Mines and Geology [CDMG], 1998) indicates the historically highest groundwater level in the area is approximately 10 feet beneath the 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.

The Los Angeles County Department of Public Works (LACDPW) has maintained various wells in the vicinity of the subject site over the past 90 years. The closest active groundwater monitoring well to the site is Well No. 2642P located approximately 0.6 mile to the north (LACDPW, 2018a). Review of the monitoring data for this well indicates monitoring data is available for the monitoring period between 1984 and 2012. During this time, the depth to groundwater has fluctuated between high and low measurements of 0 feet below the existing ground surface (measured on February 19, 1987) to 14.9 feet below the existing ground surface (measured on April 18, 1995), respectively (LACDPW, 2018a). The most recent groundwater level measurement for Well No. 2642P was measured in September 2012, and groundwater was at a depth of approximately 11.9 feet below the existing ground surface (LACDPW, 2018a).

Groundwater was encountered in borings B1, B2, B3 and B4 drilled on January 2 and 3, 2018, at depths of 30 feet, 20 feet, 27 feet, and 25 feet below the existing ground surface, respectively. Also, groundwater was encountered in the previous Krazan borings drilled at the site in June 2017, at depths ranging from 20 to 22 feet beneath the existing ground surface (Krazan, 2017). 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 8.24).

7. GEOLOGIC HAZARDS

7.1 Surface Fault Rupture

The numerous faults in Southern California include active, potentially active, 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, an active fault is one that has had surface displacement within Holocene time (about the last 11,700 years). A potentially active 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, 2018b; CGS, 2014b) or a city-designated Preliminary Fault Rupture Study Area (City of Los Angeles, 2018) for surface fault rupture hazards. No active or potentially active 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 4, Regional Fault Map.

The closest surface trace of an active fault to the site is the Hollywood Fault located approximately 1.8 miles to the north (CGS, 2014b; CGS, 2018b). Other nearby active faults include the Newport-Inglewood Fault Zone, the Santa Monica Fault, the Raymond Fault, the Verdugo Fault, the Sierra Madre Fault Zone, and the Palos Verdes Fault Zone located approximately 2.0 miles southwest, 3.2 miles west, 8.2 miles east-northeast, 9.0 miles northeast, 15.5 miles north, and 17.3 miles southwest of the site, respectively (Ziony and Jones, 1989). The active San Andreas Fault Zone is located approximately 36 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 deep thrust faults and others in the Los Angeles area are not exposed at the surface and do not underlie the site at depth or present a potential surface fault rupture hazard at the site. The closest buried thrust fault to the site is the Puente Hills Blind Thrust located approximately 4 miles to east. Although this deep thrust faults and others in the Los Angeles area do not underlie the site, they 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 5, 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 following table.

LIST OF HISTORIC EARTHQUAKES

Earthquake (Oldest to Youngest)	Date of Earthquake	Magnitude	Distance to Epicenter (Miles)	Direction to Epicenter
San Jacinto-Hemet area	April 21, 1918	6.8	81	ESE
Near Redlands	July 23, 1923	6.3	64	E
Long Beach	March 10, 1933	6.4	39	SE
Tehachapi	July 21, 1952	7.5	74	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	14	NW
Hector Mine	October 16, 1999	7.1	125	ENE

Similar to other sites in the region, the project 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 adequately reduced if the proposed structures are designed and constructed in conformance with current building codes and engineering practices.

7.3 Seismic Design Criteria

The following table summarizes site-specific design criteria obtained from the 2016 California Building Code (CBC; Based on the 2015 International Building Code [IBC] and ASCE 7-10), Chapter 16 Structural Design, Section 1613 Earthquake Loads. The data was calculated using the computer program *U.S. Seismic Design Maps*, provided by the USGS. The short spectral response uses a period of 0.2 second. We evaluated the Site Class based on the discussion in Section 1613.3.2 of the 2016 CBC and Table 20.3-1 of ASCE 7-10. The values presented below are for the risk-targeted maximum considered earthquake (MCE_R).

2016 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2016 CBC Reference
Site Class	D	Section 1613.3.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	1.944g	Figure 1613.3.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.813g	Figure 1613.3.1(2)
Site Coefficient, F_A	1.0	Table 1613.3.3(1)
Site Coefficient, F_V	1.5	Table 1613.3.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	1.944g	Section 1613.3.3 (Eqn 16-37)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	1.220g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.296g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.813g	Section 1613.3.4 (Eqn 16-40)

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 7-10.

ASCE 7-10 PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-10 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.749g	Figure 22-7
Site Coefficient, F_{PGA}	1.0	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.749g	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 2016 California Building Code and ASCE 7-10, 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, 2008 Conterminous U.S. Dynamic edition. The result of the deaggregation analysis indicates that the predominant earthquake contributing to the MCE peak ground acceleration is characterized as a 6.74 magnitude event occurring at a hypocentral distance of 6.61 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 6.70 magnitude occurring at a hypocentral distance of 11.15 kilometers from the site.

Conformance to the criteria in the above tables for seismic design would adequately mitigate potentially significant impacts due to strong ground shaking in the event of an earthquake. Adherence to the applicable regulatory requirements for building on the site is required and minimizes the likelihood of structure collapse. Nonetheless, as with all sites in the area subject to strong ground shaking during an earthquake, this report does not constitute any kind of guarantee or assurance that serious structural damage or ground failure would not occur in the event of a large earthquake. The primary goal of seismic design is to protect life, not to avoid all damage, and compliance with applicable regulatory requirements would result in development on the site that is adequate to reduce potential impacts to people and structures to an acceptable level.

7.4 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.

The State of California Seismic Hazard Zone Map for the Hollywood Quadrangle (CGS, 2014b; CDMG, 1999) indicates that the site is not located in an area designated as having a potential for liquefaction. In addition, a review of the County of Los Angeles Safety Element (Leighton, 1990) indicates that the site is not located within an area identified as having a potential for liquefaction. Based on these considerations, as well as the relatively dense and well-consolidated nature of the soils at and below the depth of proposed construction, it is our opinion that the potential for liquefaction and associated ground deformations beneath the site is very low.

7.5 Slope Stability

The topography at the site is relatively level and the topography in the immediate site vicinity slopes gently to the west-southwest. The site is not located within a City of Los Angeles Hillside Grading Area or a Hillside Ordinance Area (City of Los Angeles, 2018). The County of Los Angeles Safety Element (Leighton, 1990), indicates the site is not 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, 2014b). 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.

7.6 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.

7.7 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 within an area of minimal flooding (Zone X) as defined by the Federal Emergency Management Agency (FEMA, 2018: LACDPW, 2018b).

7.8 Oil Fields & Methane Potential

Based on a review of the California Division of Oil, Gas and Geothermal Resources (DOGGR) Well Finder Website, the site is located within the Salt Lake Oil Field (DOGGR, 2018). The nearest well to the site is the Chevron USA Well Number 99, a plugged oil and gas production well, located within the northern portion of the site (DOGGR, 2018). 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 DOGGR.

The site is located within the boundaries of a city-designated Methane Zone (City of Los Angeles, 2018). Prior to approval of the proposed project, the City of Los Angeles will require a site-specific methane study be performed to evaluate the potential for methane and other volatile gases to impact the proposed development. We recommend that a qualified methane consultant be retained to perform the study and provide mitigation measures as necessary.

7.9 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. The site is not located within an area of known on-going ground subsidence. No large-scale extraction of groundwater, gas, oil, or geothermal energy is occurring or planned at the site or in the general site vicinity. 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 on the site during the investigation that would preclude the construction of the proposed development provided that construction adheres to applicable regulatory requirements and the recommendations presented herein implemented during design and construction.

8.1.2 Up to 5½ 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. Future demolition of the existing commercial structure and basement level which occupies the site will likely disturb the upper few feet of soil below those existing improvements. 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 8.6).

8.1.3 Excavations for the subterranean level are anticipated to penetrate through the existing artificial fill and expose alluvial soils throughout the excavation bottom.

8.1.4 Groundwater was encountered during current and prior site exploration at depths ranging from 20 to 30 feet below existing ground surface. Excavation for construction of the proposed subterranean levels is anticipated to extend to depths of 30 to 35 feet, including foundation excavations, and/or methane systems. Based on these considerations, groundwater may be encountered during construction and the contractor should be prepared for these conditions.

8.1.5 The historic high groundwater level beneath the site is reported as 10 feet below the existing ground surface. If the subterranean portion of the structure which extends below the historic high groundwater level is not designed for full hydrostatic pressure, a permanent dewatering system will be required to relieve and mitigate the water pressure. Recommendations for *Permanent Dewatering* are provided in Section 8.3 of this report.

8.1.6 If a permanent dewatering system is not implemented, the structure must be designed for hydrostatic pressure based on the historic high groundwater level of 10 feet below the ground surface. 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 recommendations for uplift mitigation will be required. Such recommendations, if deemed necessary, will be provided under separate cover as the design becomes more finalized.

8.1.7 Based on the depth of proposed construction, the potential for a methane mitigation system and potential hydrostatic pressures, it is recommended that proposed structure be supported on a reinforced concrete mat foundation system. A mat foundation system is anticipated to be a very cost-effective foundation system for this project since the pad can remain relatively flat which allows for more efficient construction of waterproofing or a methane system, saving a significant amount of time and labor. In order to minimize differential settlement between the ramp, ramp walls, and basement level, it is recommended that the ramp and ramp walls for the subterranean parking garage be structurally supported on the mat foundation. Recommendations for the design of a mat foundation system are provided in Section 8.8 of this report.

8.1.8 Once proposed building loads become available and subterranean elevations are established, additional analyses will be required to evaluate the anticipated total and differential settlements between the foundation elements. Updated foundation design recommendations will be provided as necessary in an addendum report.

8.1.9 Excavation for construction of the proposed subterranean levels is anticipated to extend to depths of 30 to 35 feet, including foundation excavations, and/or methane systems. Due to the depth of the excavation and the proximity to the property lines, city streets and adjacent offsite structures and improvements, excavation of the proposed subterranean level will require sloping and/or shoring 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 8.18 of this report.

8.1.10 Due to the nature of the proposed design and intent for a subterranean level, waterproofing of subterranean walls and slabs is suggested. 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.

8.1.11 Based on the historic and current groundwater levels, as well as the footprint and depth of proposed construction, infiltration of stormwater is not recommended for this project. It is suggested that stormwater be retained, filtered and discharged in accordance with the requirements of the local governing agency.

8.1.12 Any changes in the design, location or elevation, as outlined in this report, should be reviewed by this office. Once the foundation loading configuration and design elevations for proposed structure proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, as necessary. Based on the final foundation loading configurations and building elevations, the potential for settlement should be reevaluated by this office.

8.1.13 The most recent ASTM standards apply to this project and must be utilized, even if older ASTM standards are indicated in this report.

8.2 Temporary Dewatering

8.2.1 Groundwater was encountered during current and prior site exploration at depths ranging from 20 to 30 feet below existing ground surface. Based on the conditions encountered at the time of exploration, groundwater may be encountered during construction activities. The depth to groundwater at the time of construction can be further verified during initial dewatering well or shoring pile installation. If groundwater is present above the depth of the proposed foundation excavation bottom, temporary dewatering will be necessary to maintain a safe working environment during excavation and construction activities.

8.2.2 It is recommended that a qualified dewatering consultant be retained to design the dewatering system and determine the design flow rates for dewatering. 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.

8.2.3 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.

8.3 Permanent Dewatering

8.3.1 If any portion of the proposed structure extends below the historic high groundwater depth and is not designed for full hydrostatic pressure and buoyancy, a permanent dewatering system will be required to relieve and mitigate the water pressure. The historically highest groundwater is reported to be at a depth of 10 feet below the existing ground surface. If permanent dewatering is to be utilized, a sub-slab drainage system consisting of perforated pipes placed in gravel-filled trenches may be installed beneath the subterranean slab-on-grade to intercept and control groundwater. A separate retaining wall drainage system is also required around the perimeter of the structure. The sub-slab drainage system can be combined with the perimeter retaining wall drainage system provided backflow valves are installed at the base of the wall drainage system.

8.3.2 A typical permanent sub-slab drainage system would consist of a 12-inch thick layer of ¾-inch gravel that is placed upon a layer of filter fabric (Miami 500X or equivalent), and vibrated to a dense state. Subdrain pipes leading to sump areas, provided with automatic pumping units, should drain the gravel layer. The drain lines should consist of perforated pipe, placed with perforations down, in trenches that are at least 6 inches below the gravel layer. The excavation bottom, as well as the trench bottoms should be lined with filter fabric prior to placing and compacting gravel. The trenches should be spaced approximately 40 feet apart at most, within the interior, and should extend along to the perimeter of the building. Subsequent to the installation of the drainage system, the waterproofing system and building slab may then be placed on the densified gravel. A mud- or rat-slab may be placed above and below the waterproofing system for protection during placement of rebar and mat slab construction.

8.3.3 Recommendations for design flow rates for the permanent dewatering system should be determined by a qualified contractor or dewatering consultant.

8.4 Soil and Excavation Characteristics

8.4.1 The in-situ soils can be excavated with moderate effort using conventional excavation equipment. Some caving should be anticipated in unshored excavations, especially where granular and/or saturated soils are encountered.

8.4.2 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 existing adjacent improvements.

8.4.3 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 or shoring. Excavation recommendations are provided in the *Temporary Excavations* section of this report (see Section 8.17).

8.4.4 The upper 5 feet of existing site soils encountered during this investigation are considered to have a "very low" expansive potential (EI = 17); and are classified as "non-expansive" based on the 2016 California Building Code (CBC) Section 1803.5.3. The recommendations presented herein assume that foundations and slabs at the ground surface will derive support in these materials. Furthermore, based on depth of the proposed subterranean levels, the proposed structure would not be prone to the effects of expansive soils.

8.5 Minimum Resistivity, pH, and Water-Soluble Sulfate

8.5.1 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 "corrosive" with respect to corrosion of buried ferrous metals on site. The results are presented in Appendix B (Figure B8) and should be considered for design of underground structures.

8.5.2 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 B8) and indicate that the on-site materials possess "negligible" sulfate exposure to concrete structures as defined by 2016 CBC Section 1904 and ACI 318-11 Sections 4.2 and 4.3.

8.5.3 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.

8.6 Grading

8.6.1 Grading is anticipated to include excavation of site soils for the subterranean level, foundations, and utility trenches, as well as placement of backfill for walls, ramps, and trenches.

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

8.6.3 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.

8.6.4 Grading should commence with the removal of all 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. All 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 observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.) and the City of Los Angeles Inspector.

8.6.5 Based on the depth of proposed construction, the potential for a methane mitigation system, and potential hydrostatic pressures, it is recommended that proposed structure be supported on a reinforced concrete mat foundation system deriving support in competent alluvial soils found at and below the proposed excavation bottom. For the purposes of this report, the foundation depth has been assumed to be 35 feet below the existing ground surface. Foundations should be deepened as necessary to extend into satisfactory soils and must be observed and approved by the Geotechnical Engineer (a representative of Geocon), prior to placing steel or concrete.

8.6.6 Due to the potential for high-moisture content soils at the excavation bottom, or if construction is performed during the rainy season and the excavation bottom becomes saturated, stabilization measures may have to be implemented to prevent excessive disturbance 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. Track mounted equipment should be considered to minimize disturbance to the soils.

8.6.7 Where permanent dewatering is not used, an alternative method of subgrade stabilization would 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.).

8.6.8 If a permanent dewatering system is to be installed, subgrade stabilization may be accomplished by placing a 1-foot-thick layer of washed, angular 3/4-inch gravel atop a stabilization fabric (Mirafi 500X or equivalent), subsequent to subgrade approval. This procedure should be conducted in sections until the entire excavation bottom has been blanketed by fabric and gravel. Heavy equipment may operate upon the gravel once it has been placed. The gravel should be compacted to a dense state utilizing a vibratory drum roller. The placement of gravel at the subgrade level should be coordinated with the temporary or permanent dewatering of the site. The gravel and fabric system will function as both a permeable material for any necessary dewatering procedures as well as a stable material upon which heavy equipment may operate. It is recommended that the contractor meet with the Geotechnical Engineer to discuss this procedure in more detail.

8.6.9 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 to be utilized in the 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 90 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).

8.6.10 Prior to construction of exterior slabs, the upper 12 inches of the subgrade should be moisture conditioned to optimum moisture content and properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D1557 (latest edition).

8.6.11 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 B8).

8.6.12 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 is also acceptable as backfill (see Section 8.7). 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).

8.6.13 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.

8.7 Controlled Low Strength Material (CLSM)

8.7.1 Controlled Low Strength Material (CLSM) may be utilized in lieu of compacted soil as engineered fill where approved in writing by the Geotechnical Engineer. Where utilized within the City of Los Angeles use of CLSM is subject to the following requirements:

Standard Requirements

1. CLSM shall be ready-mixed by a City of Los Angeles approved batch plant;
2. CLSM shall not be placed on uncertified fill, on incompetent natural soil, nor below water;
3. CLSM shall not be placed on a sloping surface with a gradient steeper than 5:1 (horizontal to vertical);
4. Placement of the CLSM shall be under the continuous inspection of a concrete deputy inspector;
5. The excavation bottom shall be accepted by the soil engineer and the City Inspector prior to placing CLSM.

Requirements for CLSM that will be used for support of footings

1. The cement content of the CLSM shall not be less than 188 pounds per cubic yard (min. 2 sacks);
2. The excavation bottom must be level, cleaned of loose soils and approved in writing by Geocon prior to placement of the CLSM;
3. The ultimate compressive strength of the CLSM shall be no less than 100 pounds per square inch (psi) when tested on the 28th-day per ASTM D4832 (latest edition), Standard Test Method for Preparation and Testing of Controlled Low Strength Material Test Cylinders. Compression testing will be performed in accordance with ASTM C39 and City of Los Angeles requirements;
4. Samples of the CLSM will be collected during placement, a minimum of one test (two cylinders) for each 50 cubic yards or fraction thereof;

5. Overexcavation for CLSM placement shall extend laterally beyond the footprint of any proposed footings as required for placement of compacted fill, unless justified otherwise by the soil engineer that footings will have adequate vertical and horizontal bearing capacity.

8.8 Mat Foundation Design - General

8.8.1 Based on the depth of proposed construction, the potential for a methane mitigation system, and potential hydrostatic pressures, it is recommended that proposed structure be supported on a reinforced concrete mat foundation system. A mat foundation system is anticipated to be a very cost-effective foundation system for this project since the pad can remain relatively flat which allows for more efficient construction of waterproofing or a methane system, saving a significant amount of time and labor.

8.8.2 Foundations should be deepened as necessary to extend into satisfactory soils and must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.).

8.8.3 It is anticipated that the mat foundation will impart an average pressure of less than 2,500 psf, with locally higher pressures up to 5,000 psf. The use of a maximum allowable bearing pressure of 5,000 psf is feasible and anticipated differential settlements should be evaluated once the bearing pressure distribution beneath the mat foundation has been evaluated. The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

8.8.4 It is recommended that a modulus of subgrade reaction of 100 pounds per cubic inch (pci) be utilized for the design of the mat foundation bearing in the undisturbed alluvial soils found at the excavation bottom. If the subgrade is stabilized in accordance with the recommendations in the grading section of this report a modulus of subgrade reaction of 200 pci may be utilized. This value is a unit value for use with a 1-foot square footing. The modulus should be reduced in accordance with the following equation when used with larger foundations:

$$K_R = K\left[\frac{B+1}{2B}\right]^2$$

where: K_R = reduced subgrade modulus
K = unit subgrade modulus
B = foundation width (in feet)

8.8.5 The thickness of and reinforcement for the mat foundation should be designed by the project structural engineer.

8.8.6 If the proposed structure is to be designed for full hydrostatic pressure, the recommended floor slab uplift pressure to be used in design would be 62.4(H) in units of psf, where “H” is the height of the water above the bottom of the mat foundation in feet. For design purposes the water table may be assumed to be 10 feet below the existing ground surface. If the proposed structure does not provide sufficient dead load to resist the buoyant forces then uplift mitigation will be required. Such recommendations, if deemed necessary, will be provided under separate cover as the design becomes more finalized.

8.8.7 For seismic design purposes, a coefficient of friction of 0.3 may be utilized between the concrete mat and alluvium or engineered fill without a moisture barrier; 0.45 may be utilized between the concrete mat and stabilized subgrade without a moisture barrier; and 0.15 for slabs underlain by a moisture barrier.

8.8.8 No special subgrade presaturation is required prior to placement of concrete. However, the slab and foundation subgrade should be sprinkled as necessary; to maintain a moist condition as would be expected in any concrete placement.

8.8.9 Waterproofing of subterranean walls and slabs is recommended for this project for any portions of the structure that will be constructed below the groundwater table. 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.

8.8.10 Foundation excavations should be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.), prior to the placement of the methane system, reinforcing steel and concrete to verify that the excavations and exposed soil conditions are consistent with those anticipated. If unanticipated soil conditions are encountered, foundation modifications may be required.

8.8.11 This office should be provided a copy of the final construction plans so that the excavation recommendations presented herein could be properly reviewed and revised if necessary.

8.9 Foundation Settlement

8.9.1 The maximum settlement for a reinforced concrete mat foundation designed with the maximum allowable bearing value of 5,000 psf and deriving support in the recommended bearing materials is expected to be less than 1 inch and occur below the heaviest loaded structural element. A majority of the settlement of the foundation system is expected to occur on initial application of loading; however, minor additional settlements are expected within the first twelve months. The anticipated settlement includes consideration of the removal of 35 feet of soil overburden. Differential settlement between the center and corner of the mat is expected to be less than ½ inch.

8.9.2 Once the design and foundation loading configurations for the proposed structures proceeds to a more finalized plan, the estimated settlements presented in this report should be reviewed and revised, if necessary. Foundation dimensions and allowable bearing pressures may require adjustment to minimize potential differential settlements. The potential for settlement should be reevaluated by this office.

8.10 Lateral Design

8.10.1 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 0.3 may be used with the dead load forces in the competent alluvial soils or in properly compacted engineered fill.

8.10.2 Passive earth pressure for the sides of foundations poured against undisturbed alluvium may be computed as an equivalent fluid having a density of 100 pounds per cubic foot (pcf) with a maximum earth pressure of 1,000 psf (these values have been adjusted for buoyant forces). When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

8.11 Exterior Concrete Slabs-on-Grade

8.11.1 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 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 12 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 one-fourth the slab thickness. The project structural engineer should design construction joints as necessary.

8.11.2 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.

8.11.3 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.

8.12 Retaining Walls Design

8.12.1 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.

8.12.2 Retaining wall foundations may be designed in accordance with the recommendations provided in the *Mat Foundation Design* section of this report (see Section 8.8).

8.12.3 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 is provided as Figure 6.

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	51	59

8.12.4 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.

8.12.5 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.

8.12.6 Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic, or adjacent structures. Recommendations for the incorporation of surcharges are provided in section 8.23 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.

8.12.7 In addition to the recommended earth pressure, the upper 10 feet of the subterranean wall adjacent to the street and parking lot should be designed to resist a uniform lateral pressure of 100 psf, acting as a result of an assumed 300 psf surcharge behind the walls due to normal street traffic. If the traffic is kept back at least 10 feet from the subterranean walls, or a distance from the subterranean walls equal to at least half the wall height, whichever is greater, the traffic surcharge may be neglected.

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

8.13 Dynamic (Seismic) Lateral Forces

8.13.1 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 2016 CBC).

8.13.2 A seismic load of 12 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 2016 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.

8.14 Retaining Wall Drainage

8.14.1 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 7). 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.

8.14.2 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 8). 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.

8.14.3 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.

8.14.4 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.

8.15 Elevator Pit Design

8.15.1 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 *Mat Foundation Design* and *Retaining Wall Design* sections of this report (see Sections 8.8 and 8.12).

8.15.2 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.

8.15.3 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 8.14).

8.15.4 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.

8.16 Elevator Piston

8.16.1 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.

8.16.2 Casing will likely be required in the drilled excavation. 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.

8.16.3 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.

8.17 Temporary Excavations

8.17.1 Excavations on the order of 35 feet in height may be required for excavation and construction of the proposed subterranean levels. The excavations are expected to expose artificial fill and alluvial soils, which may be subject to caving where granular or saturated soils are exposed. Vertical excavations up to 5 feet in height may be attempted where not surcharged by adjacent traffic or structures.

8.17.2 Vertical excavations greater than 5 feet or where surcharged by existing structures will require sloping or shoring measures in order to provide a stable excavation. Where sufficient space is available, temporary unsurcharged embankments could be sloped back at a uniform 1:1 slope gradient or flatter up to a maximum of 10 feet in height. A uniform slope does not have a vertical portion. Where space is limited, shoring measures will be required. *Shoring* data is provided in Section 8.18 of this report.

8.17.3 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.

8.18 Shoring – Soldier Pile Design and Installation

8.18.1 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.

8.18.2 One method of shoring would consist of steel soldier piles, placed in drilled holes and backfilled with concrete. The steel soldier piles may also be installed utilizing high frequency vibration. 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.

8.18.3 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 foundations, methane, and/or adjacent drainage systems.

8.18.4 The proposed soldier piles may be utilized to provide a component of uplift resistance. If required to provide uplift resistance, the shoring piles must be designed as permanent piles. The uplift capacity may be taken as ⅔ of the downward frictional capacity.

8.18.5 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 Wall Design* section of this report (see Section 8.12).

8.18.6 Drilled cast-in-place soldier piles should be placed no closer than 2 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 115 psf per foot (value has been reduced for buoyant forces). 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 soils. Increases in passive pressure may be available at greater depths and Geocon should be contacted to provide an updated value once a preliminary shoring design is available.

8.18.7 Groundwater was encountered during exploration and 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 insure that the tip of the tremie tube is never raised above the surface of the concrete.

8.18.8 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.

8.18.9 Casing may be required if caving may occur in the saturated soils. 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 5 feet. As an alternative, piles may be vibrated into place; however, there is always a risk that excessive vibrations in sandy soils could induce settlements and distress to adjacent offsite improvements. Continuous observation of the drilling and pouring of the piles by the Geotechnical Engineer (a representative of Geocon West, Inc.), is required.

8.18.10 As an alternative, piles may be vibrated into place; however, there is always a risk that excessive vibrations in sandy soils could induce settlements and distress to adjacent offsite improvements. Continuous observation of the drilling and pouring of the piles by the Geotechnical Engineer (a representative of Geocon West, Inc.), is required.

8.18.11 If a vibratory method of solider pile installation is utilized, predrilling may be performed prior to installation of the steel beams. If predrilling is performed, it is recommended that the bore diameter be at least 2 inches smaller than the largest dimension of the pile to prevent excessive loss in the frictional component of the pile capacity. Predrilling should not be conducted below the proposed excavation bottom.

8.18.12 If a vibratory method is utilized, the owner should be aware of the potential risks associated with vibratory efforts, which typically involve inducing settlement within the vicinity of the pile which could result in a potential for damage to existing improvements in the area.

8.18.13 The level of vibration that results from the installation of the piles should not exceed a threshold where occupants of nearby structures are disturbed, despite higher vibration tolerances that a building may endure without deformation or damage. The main parameter used for vibration assessment is peak particle velocity in units of inch per second (in/sec). The acceptable range of peak particle velocity should be evaluated based on the age and condition of adjacent structures, as well as the tolerance of human response to vibration.

8.18.14 Based on Table 19 of the *Transportation and Construction Induced Vibration Guidance Manual* (Caltrans 2013), a continuous source of vibrations (ex. vibratory pile driving) which generates a maximum peak particle velocity of 0.5 in/sec is considered tolerable for modern industrial/commercial buildings and new residential structures. The Client should be aware that a lower value may be necessary if older or fragile structures are in the immediate vicinity of the site.

8.18.15 Vibrations should be monitored and record with seismographs during pile installation to detect the magnitude of vibration and oscillation experienced by adjacent structures. If the vibrations exceed the acceptable range during installation, the shoring contractor should modify the installation procedure to reduce the values to within the acceptable range. Vibration monitoring is not the responsibility of the Geotechnical Engineer.

8.18.16 Geocon does not practice in the field of vibration monitoring. If construction techniques will be implemented, it is recommended that qualified consultant be retained to provide site specific recommendations for vibration thresholds and monitoring.

8.18.17 The frictional resistance between the soldier piles and retained soil may be used to resist the vertical component of the load. The coefficient of friction may be taken as 0.3 based on uniform contact between the steel beam and lean-mix concrete and alluvium. 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 350 psf (value has been reduced for buoyant forces). Increases in frictional resistance may be available at greater depths and Geocon should be contacted to provide an updated value once a preliminary shoring design is available.

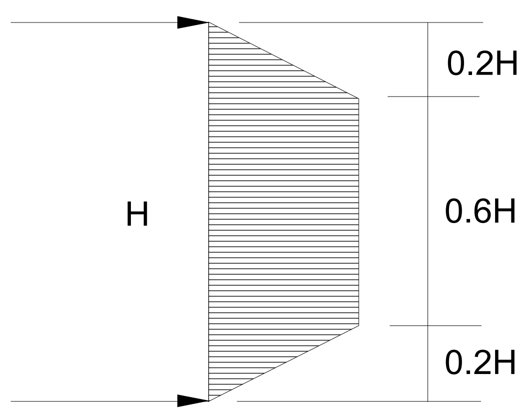
8.18.18 Due to the nature of the site soils, it is expected that continuous lagging between soldier piles will be required. However, it is recommended that the exposed soils be observed by the Geotechnical Engineer (a representative of Geocon West, Inc.), to verify the presence of any cohesive soils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

8.18.19 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.

8.18.20 For the design of shoring, it is recommended that an equivalent fluid pressure based on the following table, be utilized for design. A trapezoidal distribution of lateral earth pressure may be used where shoring will be bracing or tie backs. 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 pressures is provided as Figure 9.

HEIGHT OF SHORING (FEET)	EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (ACTIVE PRESSURE)	EQUIVALENT FLUID PRESSURE (Pounds Per Square Foot per Foot) Active Trapezoidal (Where H is the height of the shoring in feet)
Up to 35	44	28H

Trapezoidal Distribution of Pressure



8.18.21 It is very important to note that active pressures can only be achieved when movement in the soil (earth wall) occurs. If movement in the soil is not acceptable, such as adjacent to an existing structure, an at-rest pressure of 65 pcf should be considered for the design of shoring up to 29 feet in height.

8.18.22 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 slopes, vehicular traffic or adjacent structures and should be designed for each condition. The surcharge pressure should be evaluated in accordance with the recommendations in Section 8.23 of this report.

8.18.23 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, or a distance from the shoring equal to at least half the shoring height, whichever is greater, the traffic surcharge may be neglected.

8.18.24 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.

8.18.25 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.

8.18.26 Due to the depth of 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 even 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.

8.19 Temporary Tie-Back Anchors

8.19.1 Temporary tie-back anchors may be used to resist lateral loads. The owner is responsible for obtaining agreements for installation of temporary tie-backs which extend beyond the property lines. 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.

8.19.2 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. For preliminary design purposes, it is estimated that drilled friction anchors constructed without utilizing post-grouting techniques will develop average skin frictions as follows:

- 5 feet below the top of the excavation – 500 pounds per square foot
- 15 feet below the top of the excavation – 900 pounds per square foot

8.19.3 Depending on the techniques utilized, and the experience of the contractor performing the installation, a maximum allowable friction capacity of 3 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. Higher capacity assumptions may be acceptable, but must be verified by testing.

8.20 Anchor Installation

8.20.1 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.

8.21 Anchor Testing

8.21.1 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.

8.21.2 At least 10 percent of the anchors should be selected for "quick" 200 percent tests and three 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.

8.21.3 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.

8.21.4 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.

8.21.5 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. A representative of this firm should observe the installation and testing of the anchors.

8.22 Internal Bracing

8.22.1 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, poured with the bearing surface normal to rakers inclined at 45 degrees, a bearing value of 2,000 psf in competent alluvial soil, provided the shallowest point of the footing is at least 1 foot below the lowest adjacent grade. 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. The structural engineer should review the shoring plan to determine if the raker footings conflict with the structural foundation system.

8.23 Surcharge from Adjacent Structures and Improvements

8.23.1 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.

8.23.2 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.

8.23.3 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.

8.24 Surface Drainage

8.24.1 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.

8.24.2 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 2016 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.

8.24.3 Positive site drainage should be provided away from structures, pavement, and the tops of slopes to swales or other controlled drainage structures. The building pad and pavement areas should be fine graded such that water is not allowed to pond.

8.24.4 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 an 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.

8.25 Plan Review

8.25.1 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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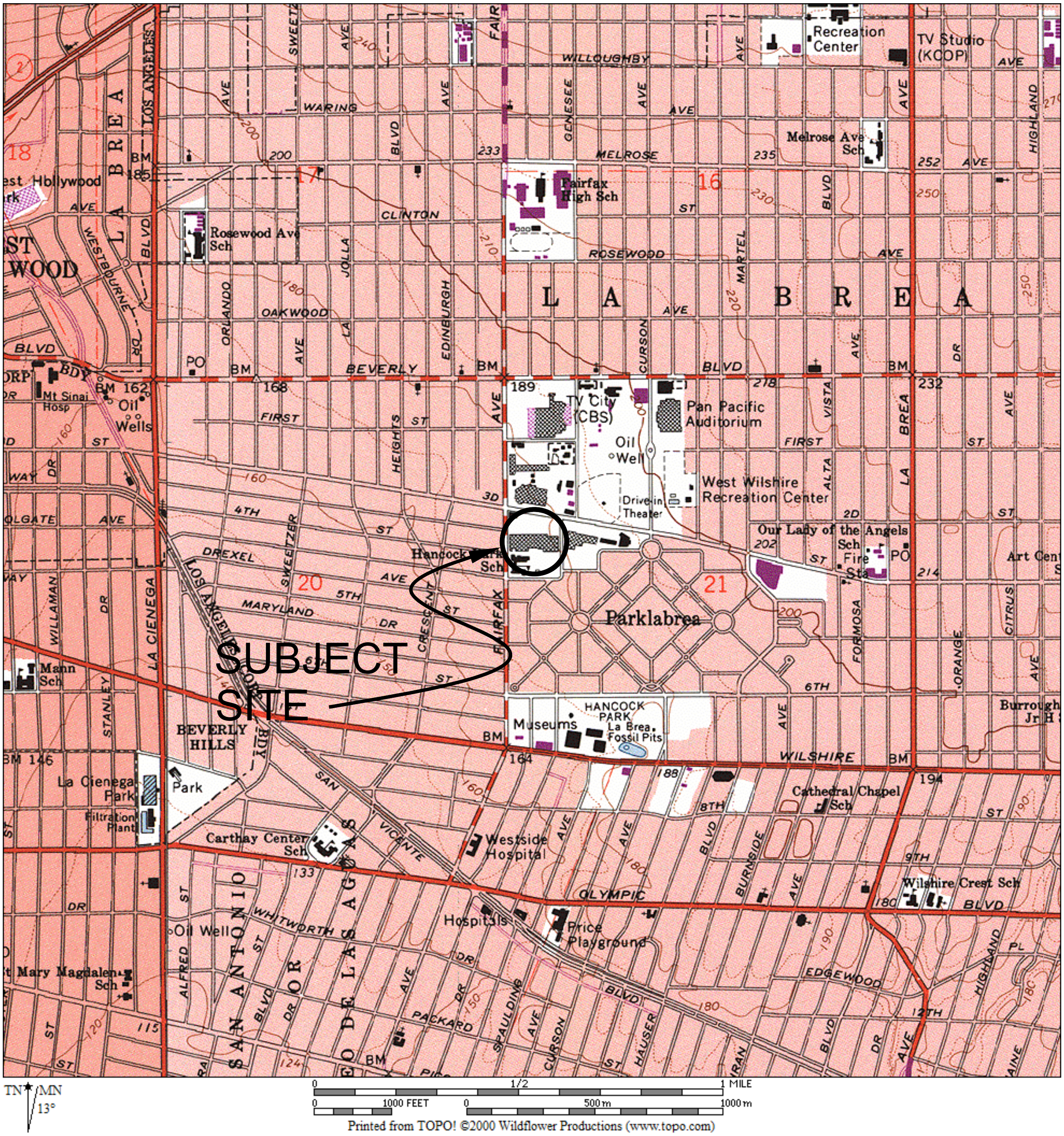

REFERENCE:U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, HOLLYWOOD, CA QUADRANGLE

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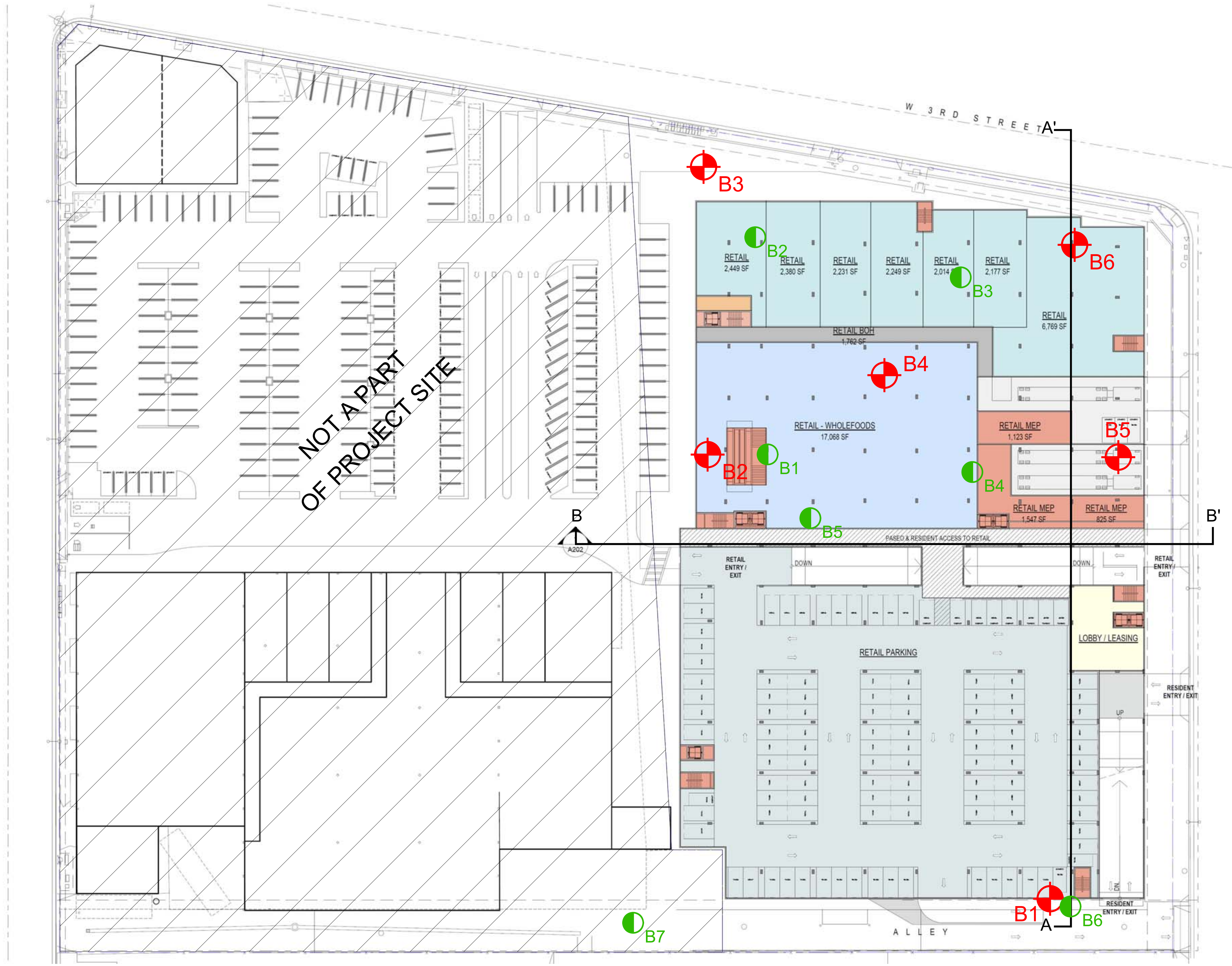
VICINITY MAP

HOLLAND ACQUISITION CO, LLC
3RD STREET AND FAIRFAX AVENUE
LOS ANGELES, CALIFORNIA

NOV 2018	PROJECT NO. A9713-06-01	FIG. 1



LEGEND

B5 — Approximate Location of Boring Geocon, 2018

B7 — Approximate Location of Boring Krazan, 2017

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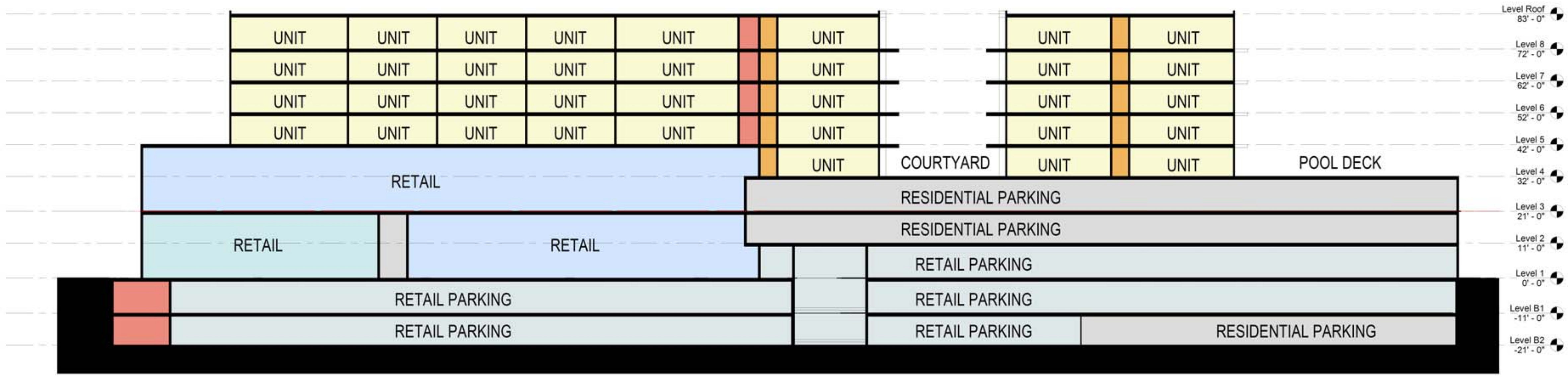
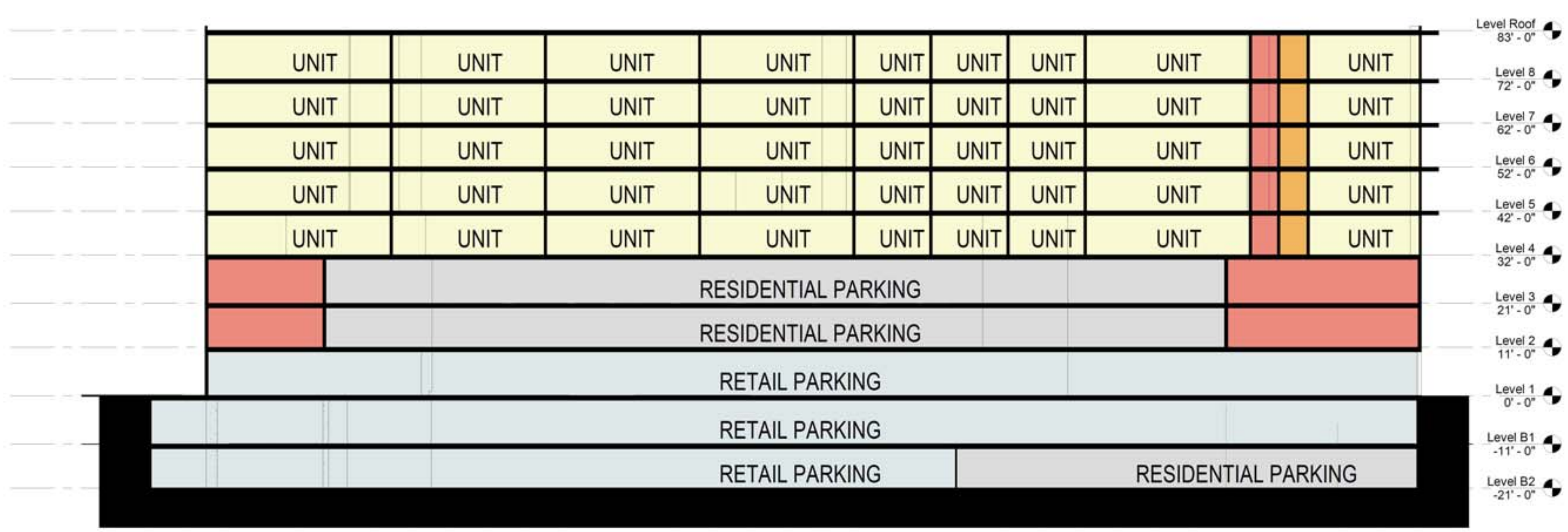
SITE PLAN

HOLLAND ACQUISITION CO, LLC
3RD STREET AND FAIRFAX AVENUE
LOS ANGELES, CALIFORNIA

NOV 2018	PROJECT NO. A9713-06-01	FIG. 2



Section A-A'



Section B-B'

1" = 40' (H&V)

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CROSS SECTION

HOLLAND ACQUISITION CO, LLC
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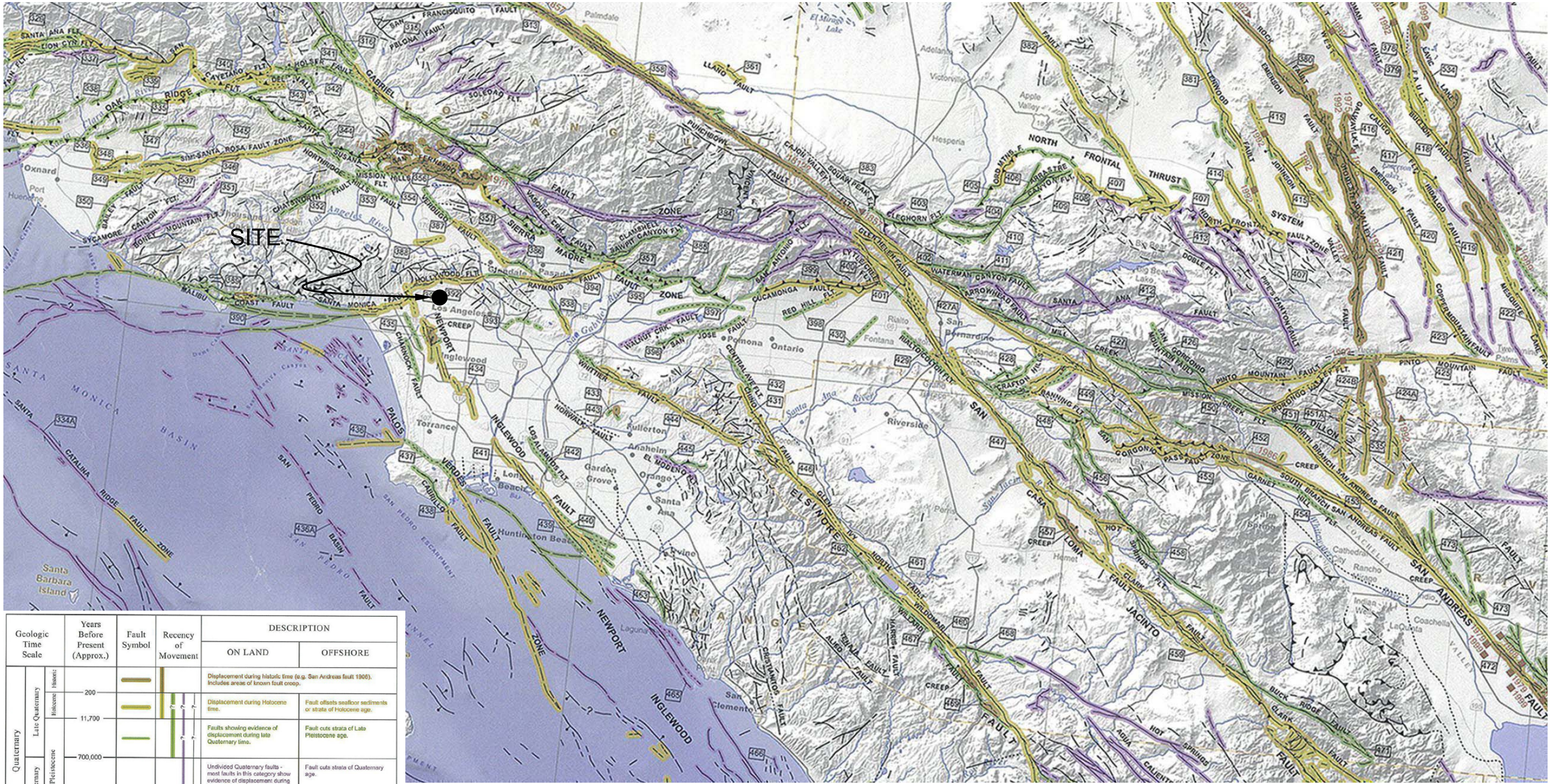
NOV 2018 | PROJECT NO. A9713-06-01 | FIG. 3

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	200			Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
		Holocene	11,700		? ? ?	Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
		Pleistocene	700,000		? ?	Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
	Early Quaternary		1,600,000*		?	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			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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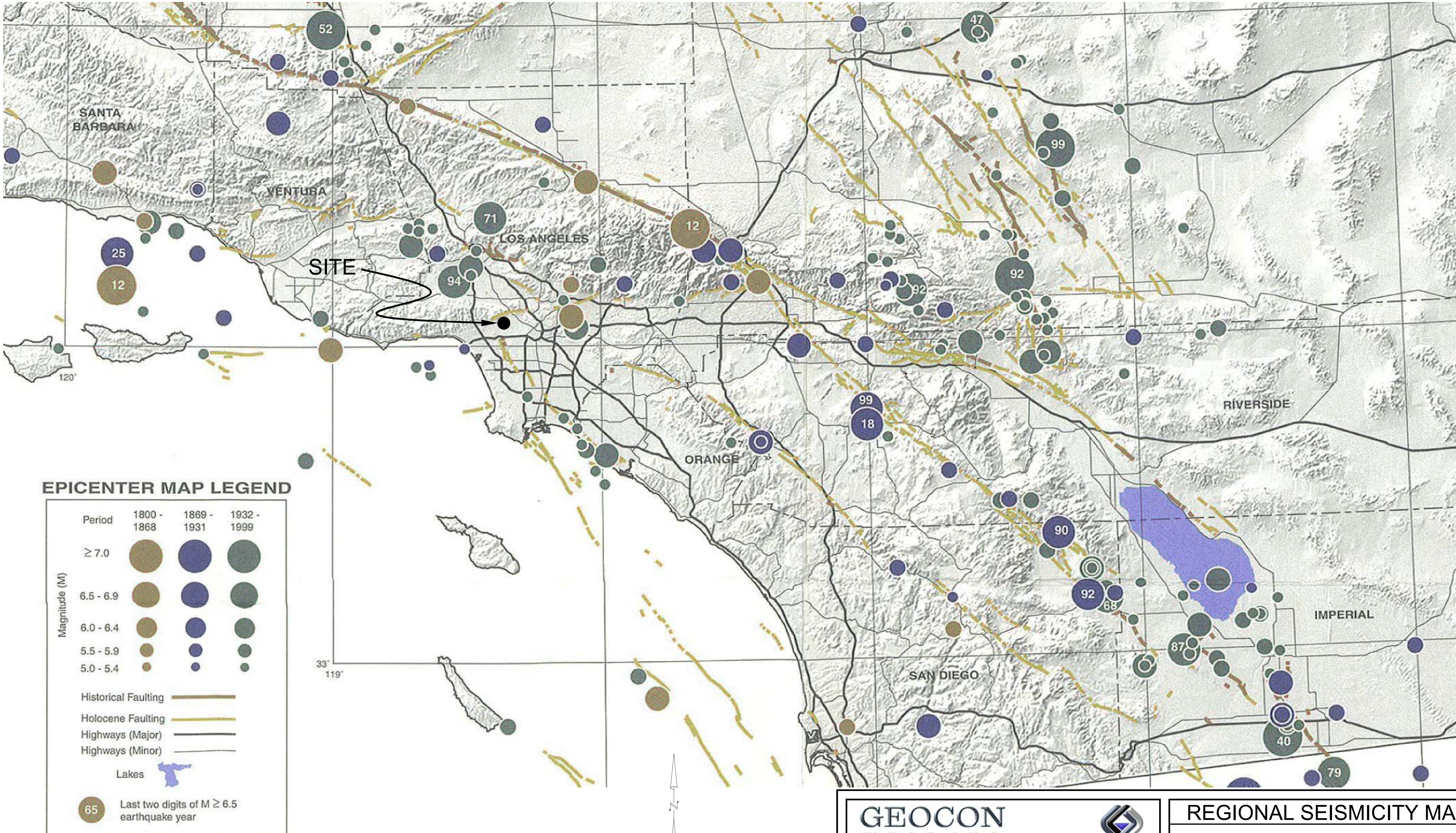
REGIONAL FAULT MAP

HOLLAND ACQUISITION CO, LLC
3RD STREET AND FAIRFAX AVENUE
LOS ANGELES, CALIFORNIA

NOV 2018 | PROJECT NO. A9713-06-01 | FIG. 4



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N

0 20 40 Miles

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REGIONAL SEISMICITY MAP

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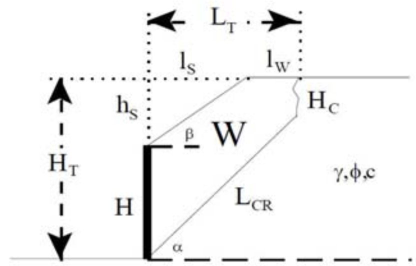
LOS ANGELES, CALIFORNIA

NOV 2018 | PROJECT NO. A9713-06-01 | FIG. 5

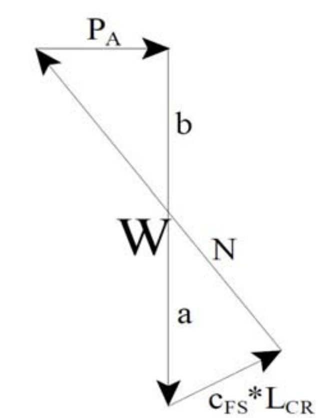
Retaining Wall Design with Transitioned Backfill
(Vector Analysis)

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)	32.0	degrees
Cohesion of Retained Soils	(c)	70.0	psf
Factor of Safety	(FS)	1.50	
Factored Parameters:	(f_{FS})	22.6	degrees
	(c_{FS})	46.7	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	1.3	449	56147.6	40.6	4594.5	51553.1	21232.2
46	1.2	434	54225.7	40.0	4337.9	49887.8	21572.2
47	1.2	419	52366.7	39.3	4105.5	48261.2	21876.4
48	1.2	405	50566.5	38.8	3894.2	46672.3	22146.1
49	1.2	391	48821.5	38.2	3701.5	45120.0	22382.4
50	1.2	377	47128.1	37.6	3525.3	43602.8	22586.5
51	1.2	364	45483.2	37.1	3363.8	42119.4	22759.1
52	1.1	351	43883.8	36.6	3215.3	40668.4	22900.9
53	1.1	339	42327.0	36.1	3078.5	39248.5	23012.6
54	1.1	326	40810.5	35.7	2952.3	37858.2	23094.6
55	1.1	315	39331.6	35.3	2835.5	36496.1	23147.2
56	1.1	303	37888.2	34.8	2727.2	35161.0	23170.7
57	1.1	292	36478.2	34.4	2626.7	33851.5	23165.1
58	1.1	281	35099.6	34.1	2533.2	32566.5	23130.4
59	1.1	270	33750.6	33.7	2446.1	31304.6	23066.5
60	1.1	259	32429.5	33.3	2364.8	30064.7	22973.2
61	1.1	249	31134.5	33.0	2288.8	28845.6	22850.0
62	1.2	239	29864.2	32.7	2217.8	27646.4	22696.4
63	1.2	229	28617.1	32.4	2151.2	26465.9	22511.8
64	1.2	219	27391.9	32.1	2088.7	25303.1	22295.5
65	1.2	209	26187.2	31.8	2030.0	24157.1	22046.4
66	1.2	200	25001.8	31.5	1974.8	23027.0	21763.6
67	1.3	191	23834.5	31.2	1922.8	21911.7	21445.8
68	1.3	181	22684.3	31.0	1873.7	20810.6	21091.7
69	1.3	172	21550.0	30.7	1827.4	19722.6	20699.5
70	1.4	163	20430.7	30.5	1783.5	18647.2	20267.5



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

$P_{A,\,max}$ 23170.7 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 51.5 pcf 58.8 pcf

Design Wall for an Equivalent Fluid Pressure: 51 pcf 59 pcf

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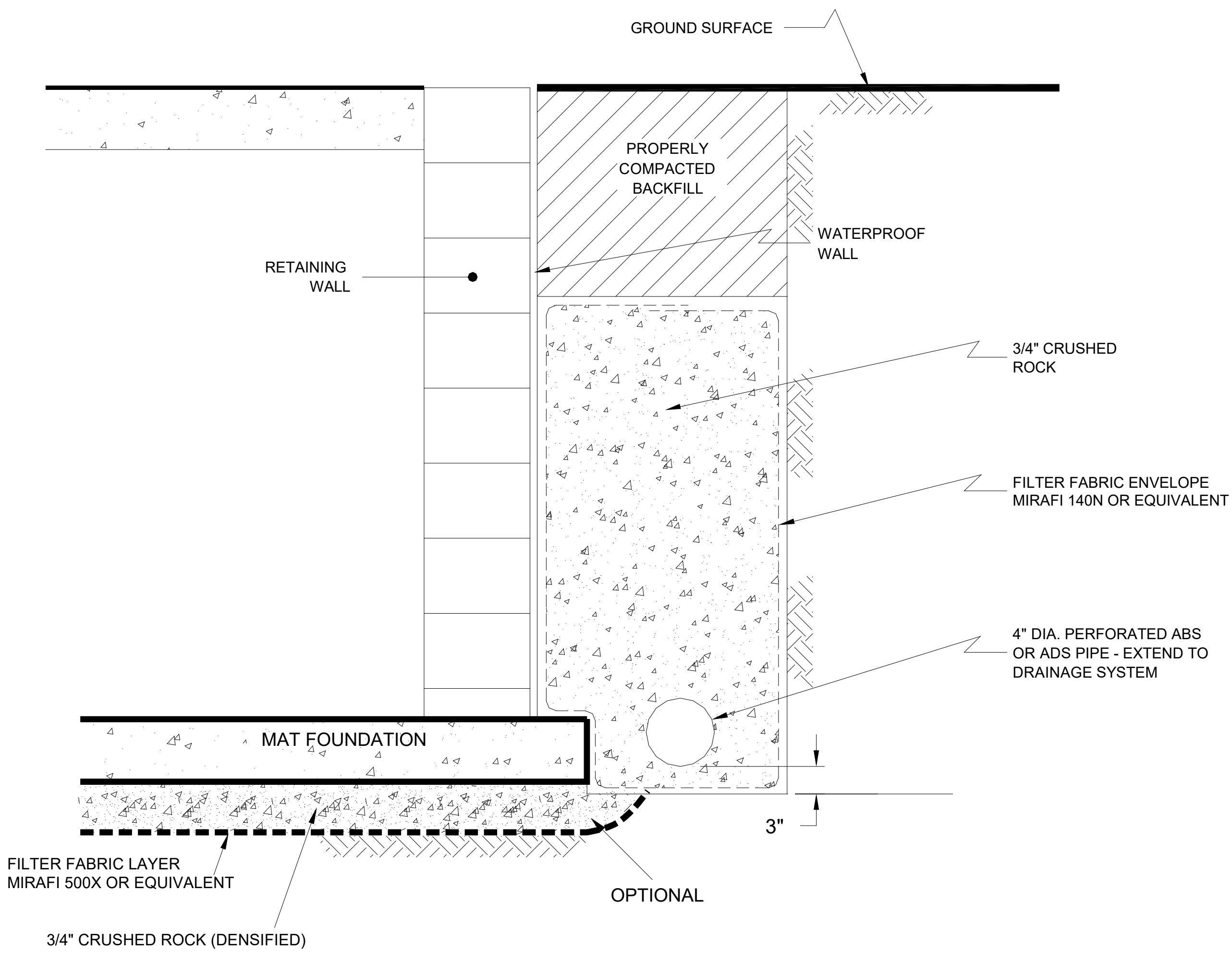
RETAINING WALL PRESSURE CALCULATION

HOLLAND ACQUISITION CO, LLC
3RD STREET AND FAIRFAX AVENUE
LOS ANGELES, CALIFORNIA

Drafted by: JTA	Checked by: NDB	NOV 2018	PROJECT NO. A9713-06-01	FIG. 6



NO SCALE

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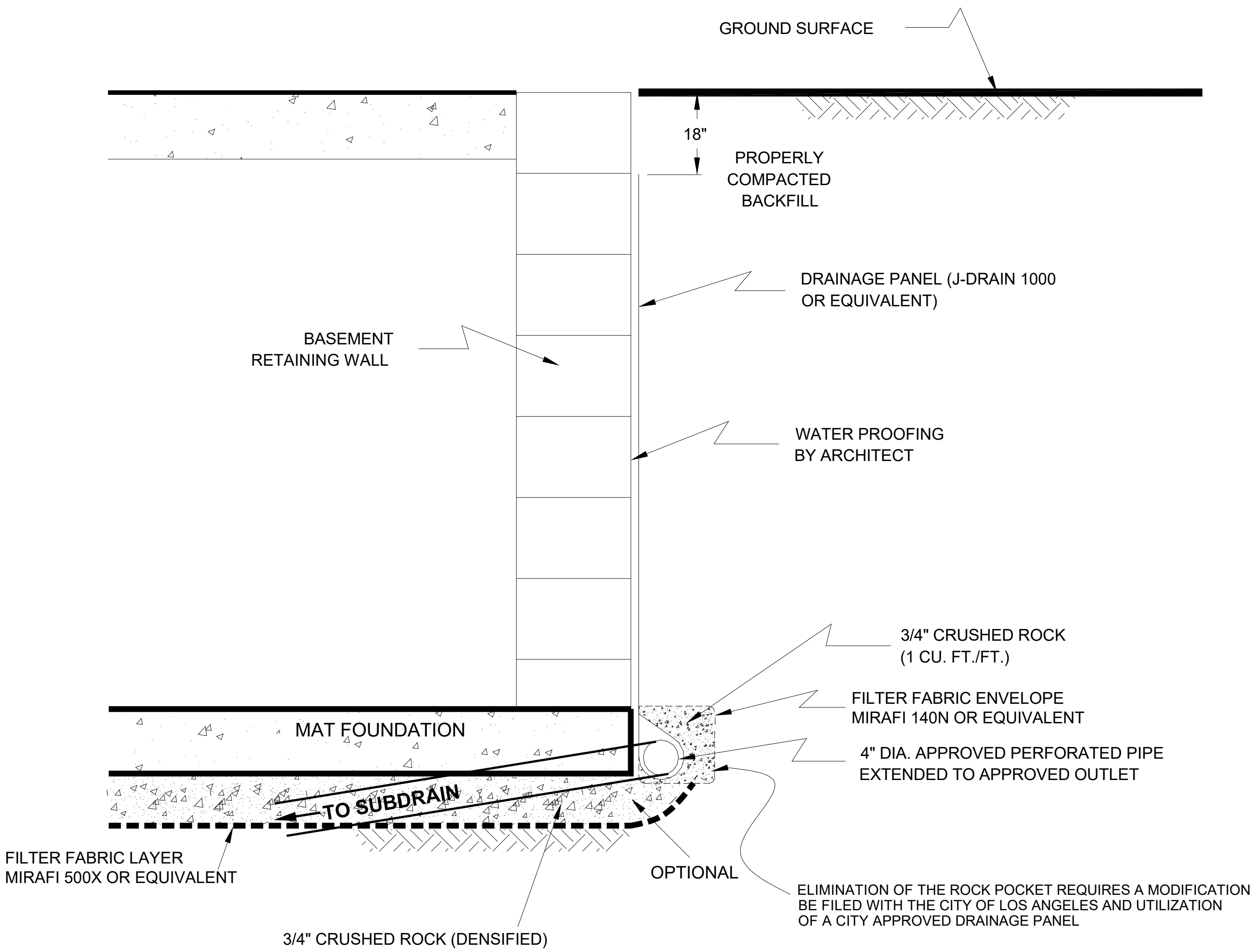
RETAINING WALL DRAIN DETAIL

HOLLAND ACQUISITION CO, LLC
3RD STREET AND FAIRFAX AVENUE
LOS ANGELES, CALIFORNIA

Drafted by: JTA	Checked by: NDB	NOV 2018	PROJECT NO. A9713-06-01	FIG. 7



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

NO SCALE

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RETAINING WALL DRAIN DETAIL

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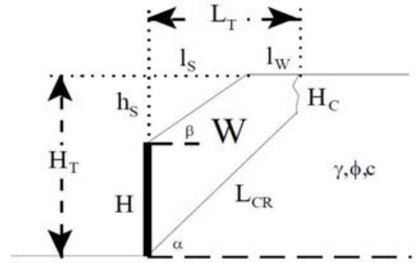
NOV 2018 | PROJECT NO. A9713-06-01 | FIG. 8

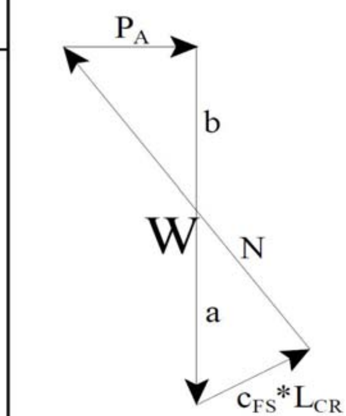
Shoring Design with Transitioned Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	35.00 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)	35.0 feet
Unit Weight of Retained Soils	(g)	125.0 pcf
Friction Angle of Retained Soils	(f)	32.0 degrees
Cohesion of Retained Soils	(c)	70.0 psf
Factor of Safety	(FS)	1.25
Factored Parameters:	(f_{FS})	26.6 degrees
	(c_{FS})	56.0 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	1.8	611	76361.9	47.0	7437.1	68924.8	22981.3
46	1.7	590	73754.2	46.2	6960.2	66794.0	23574.0
47	1.7	570	71230.7	45.6	6534.2	64696.5	24111.5
48	1.6	550	68786.1	44.9	6151.9	62634.2	24596.2
49	1.6	531	66415.7	44.3	5807.4	60608.3	25030.1
50	1.6	513	64114.8	43.6	5495.7	58619.1	25415.0
51	1.5	495	61879.2	43.1	5212.7	56666.5	25752.5
52	1.5	478	59705.1	42.5	4955.0	54750.1	26043.8
53	1.5	461	57588.7	42.0	4719.5	52869.2	26290.2
54	1.5	444	55526.5	41.4	4503.8	51022.7	26492.6
55	1.5	428	53515.5	40.9	4305.6	49209.8	26651.8
56	1.5	412	51552.4	40.5	4123.2	47429.2	26768.4
57	1.5	397	49634.7	40.0	3954.9	45679.8	26842.8
58	1.4	382	47759.5	39.6	3799.2	43960.3	26875.4
59	1.5	367	45924.4	39.1	3654.9	42269.5	26866.2
60	1.5	353	44127.1	38.7	3520.9	40606.1	26815.2
61	1.5	339	42365.3	38.3	3396.4	38968.9	26722.3
62	1.5	325	40637.0	38.0	3280.3	37356.7	26587.1
63	1.5	312	38940.2	37.6	3172.0	35768.3	26409.0
64	1.5	298	37273.1	37.3	3070.7	34202.4	26187.3
65	1.5	285	35633.9	36.9	2975.9	32658.0	25921.3
66	1.6	272	34020.9	36.6	2887.0	31133.9	25609.8
67	1.6	259	32432.5	36.3	2803.6	29629.0	25251.7
68	1.6	247	30867.3	36.0	2725.0	28142.3	24845.4
69	1.7	235	29323.7	35.7	2651.1	26672.7	24389.4
70	1.7	222	27800.4	35.4	2581.2	25219.2	23881.8



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

$P_{A,\ max}$ 26875.4 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 43.9 pcf

Design Shoring for an Equivalent Fluid Pressure: 44 pcf

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SHORING PRESSURE CALCULATION

HOLLAND ACQUISITION CO, LLC

3RD STREET AND FAIRFAX AVENUE

LOS ANGELES, CALIFORNIA

Drafted by: JTA	Checked by: NDB	NOV 2018	PROJECT NO. A9713-06-01	FIG. 9

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

The site was explored on January 2 and 3, 2018, by excavating six 8-inch-diameter borings utilizing a truck-mounted hollow-stem auger drilling machine. The borings were excavated to depths ranging from approximately 15½ to 100½ feet below the existing ground surface. 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 2 ³/₈-inch diameter brass sampler rings to facilitate soil removal and testing. Bulk samples were also obtained.

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 borings are presented on Figures A1 through A6. 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 boring logs were revised based on subsequent laboratory testing. The location of the borings are shown on Figure 2.

BORING 1

ELEV. (MSL.) **--** DATE COMPLETED **1/2/18**

EQUIPMENT **HOLLOW STEM AUGER** BY: **RMA**

DEPTH IN FEET	SAMPLE NO.	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0			**AC: 5.5" BASE: NONE**			
			ARTIFICIAL FILL			
2			Sand, poorly graded, very loose, slightly moist, dark brown with orange mottles, fine- to medium-grained, some clay.			
4						
	B1@5'			6	110.5	17.3
6			**ALLUVIUM**			
		SP	Sand, poorly graded, very loose, slightly moist, dark olive brown, fine- to medium-grained.			
8						
		SM	Silty Sand, loose, slightly moist, brown, fine- to medium-grained.			
10	B1@10'		- yellowish brown	12	118.5	10.7
12	B1@12.5'			10	108.2	23.3
14		ML	Sandy Silt, soft, slightly moist, yellowish brown with grayish brown mottles, fine-grained.			
	B1@15'			11	106.9	16.1
16		SP	Sand, poorly graded, medium dense, slightly moist, yellowish brown, fine- to medium-grained.			
18	B1@17.5'			23	100.1	14.8
		ML	Sandy Silt, stiff, slightly moist, grayish brown, trace clay.			
20	B1@20'			32	109.3	15.9
22						
24			- dark yellowish brown			
	B1@25'			32	103.0	22.9
26						
28						
			- gray			

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

A9713-06-01 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.

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PROJECT NO. A9713-06-01

BORING 1

ELEV. (MSL.) **--** DATE COMPLETED **1/2/18**

EQUIPMENT **HOLLOW STEM AUGER** BY: **RMA**

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'			ML		22	106.6	19.3
35	B1@35'			SC	Clayey Sand, medium dense, slightly moist, gray, fine-grained.	26	88.3	35.1
39					Clay, firm, slightly moist, light gray, plastic.			
40	B1@40'			CL		14	95.6	29.1
44.5					Sandy Silt, firm, slightly moist, light gray, fine-grained.			
45	B1@45'			ML		14	98.9	26.3
49					Sand, poorly graded, medium dense, wet, light gray, fine- to medium-grained, some silt.			
50	B1@50'			SP		46	119.4	13.7
54.5					Sand with Silt, medium dense, wet, light gray, fine- to medium-grained.			
55	B1@55'			SP-SM		44	113.0	19.1

A9713-06-01 BORING LOGS.GPJ

Figure A1,
Log of Boring 1, Page 2 of 4

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

BORING 1

ELEV. (MSL.) -- DATE COMPLETED **1/2/18**

EQUIPMENT **HOLLOW STEM AUGER** BY: **RMA**

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@60'			SP-SM		40	112.2	15.5
65	B1@65'			SM	Silty Sand, medium dense, wet, gray, fine- to medium-grained.	47	116.0	11.3
70	B1@70'					37	105.4	25.9
75	B1@75'					46	111.6	16.7
80	B1@80'					37	107.8	16.6
85	B1@85'				- grayish brown, fine-grained	28	112.1	14.8

Figure A1,
Log of Boring 1, Page 3 of 4

A9713-06-01 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

BORING 1

ELEV. (MSL.) -- DATE COMPLETED 1/2/18

EQUIPMENT HOLLOW STEM AUGER BY: RMA

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@90'			SM	- very dense, dark gray	50 (6")	119.8	16.0
92								
94								
	B1@95'				- light gray	50 (6")	106.4	17.4
96								
98								
100	B1@100'				- medium dense, fine- to medium-grained	52	107.3	18.7

Total depth of boring: 100.5 feet
Fill to 5.5 feet.
Groundwater encountered at 30 feet.
Backfilled with soil cuttings and tamped.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.

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

A9713-06-01 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.

PROJECT NO. A9713-06-01

BORING 2

ELEV. (MSL.) -- DATE COMPLETED 1/2/18

EQUIPMENT HOLLOW STEM AUGER BY: RMA

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**AC: 5" BASE: NONE** **ARTIFICIAL FILL** Silt, soft, slightly moist, brown.			
5	B2@5'			SM	**ALLUVIUM** Silty Sand, medium dense, slightly moist, grayish brown, fine- to medium-grained, trace clay.	25	100.6	25.1
10	B2@10'				- loose, no clay	13	95.5	24.9
11				SP	Sand, poorly graded, medium dense, slightly moist, grayish brown, fine- to medium-grained, friable.			
12.5	B2@12.5'					51	--	--
14					- gray, trace silt			
15	B2@15'					30	112.3	9.0
17.5	B2@17.5'				- loose, some coarse-grained	14	103.5	19.0
20	B2@20'		▼		- silt layer, dark brown - medium dense, some fine gravel	21	104.3	26.6
25	B2@25'			SP	Sand, poorly graded, medium dense, slightly moist, brown, fine-grained, trace clay.	21	116.6	16.6

A9713-06-01 BORING LOGS.GPJ

Figure A2,
Log of Boring 2, Page 1 of 4

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

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 2 ELEV. (MSL.) -- DATE COMPLETED 1/2/18 EQUIPMENT HOLLOW STEM AUGER BY: RMA	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
30	B2@30'			SM	Silty Sand, medium dense, slightly moist, dark gray, fine-grained, trace clay.	30	105.1	23.0
32								
34								
36	B2@35'				- increase in silt content	30	98.1	30.2
38								
40	B2@40'			SM	Silty Sand, medium dense, slightly moist, light gray, fine-grained, trace medium- to coarse-grained.	50	112.4	19.4
42								
44								
46	B2@45'				- grayish brown, fine-grained, trace clay	36	101.7	22.4
48								
50	B2@50'				- gray	36	96.9	26.9
52								
54								
56	B2@55'				- some clay	35	100.0	27.5
58								

Figure A2,
Log of Boring 2, Page 2 of 4

A9713-06-01 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.

BORING 2

ELEV. (MSL.) -- DATE COMPLETED 1/2/18

EQUIPMENT **HOLLOW STEM AUGER** BY: **RMA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
60	B2@60'			SM	- dense, trace clay, trace fine gravel (to 0.75")	64	109.7	18.1
65	B2@65'			SM	- no clay	67	165.4	14.2
69				SP	Sand, poorly graded, medium dense, wet, gray, fine- to medium-grained, trace silt.			
70	B2@70'			SP		30	107.6	20.3
75	B2@75'			SP	- very dense	50 (6")	121.1	12.6
80	B2@80'			SP		50 (6")	108.4	18.4
85	B2@85'			SP	- medium dense	29	103.5	22.9

Figure A2,
Log of Boring 2, Page 3 of 4

A9713-06-01 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

BORING 2

ELEV. (MSL.) -- DATE COMPLETED 1/2/18

EQUIPMENT HOLLOW STEM AUGER BY: RMA

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
90	B2@90'			SP		36	109.6	19.3
92								
94								
	B2@95'					33	--	--
96								
98								
100	B2@100'					43	109.8	18.9

Total depth of boring: 100.5 feet
Fill to 5 feet.
Groundwater encountered at 20 feet.
Backfilled with soil cuttings and tamped.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.

Figure A2,
Log of Boring 2, Page 4 of 4

A9713-06-01 BORING LOGS.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

PROJECT NO. A9713-06-01

BORING 3

ELEV. (MSL.) -- DATE COMPLETED 1/3/18

EQUIPMENT HOLLOW STEM AUGER BY: RMA

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**AC: 4" BASE: 4"** **ARTIFICIAL FILL** Sandy Silt, stiff, slightly moist, brown, fine-grained, some clay.			
2	B3@2.5'					30	112.3	17.8
4								
6	B3@5'			SM	**ALLUVIUM** Silty Sand, medium dense, slightly moist, yellowish brown, fine-grained.	43	115.6	15.1
8								
10	B3@10'			ML	Sandy Silt, firm, slightly moist, grayish brown, fine-grained, trace clay.	13	108.3	19.1
12	B3@12.5'			ML	Silt, firm, slightly moist, light gray.	17	104.2	23.3
14								
16	B3@15'			CL	Silty Clay, firm, slightly moist, grayish brown.	21	101.8	25.0
18	B3@17.5'			ML	Clayey Silt, stiff, slightly moist, grayish brown.	27	98.7	25.1
20	B3@20'			ML	Sandy Silt, stiff, slightly moist, grayish brown, fine-grained.	27	102.6	23.9
22								
24								
26	B3@25'		▼	SP	Sand, poorly graded, medium dense, slightly moist, gray, fine- to medium-grained, trace silt.	42	117.2	16.8
28								

Figure A3,
Log of Boring 3, Page 1 of 2

A9713-06-01 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

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**BORING 3** ELEV. (MSL.) **--** DATE COMPLETED **1/3/18** EQUIPMENT **HOLLOW STEM AUGER** BY: **RMA**	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
30	B3@30'			SP		24	107.6	20.7

Total depth of boring: 30.5 feet
Fill to 5 feet.
Groundwater encountered at 27 feet.
Backfilled with soil cuttings and tamped.
Asphalt patched.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.

Figure A3,
Log of Boring 3, Page 2 of 2

A9713-06-01 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

BORING 4

ELEV. (MSL.) -- DATE COMPLETED 1/3/18

EQUIPMENT HOLLOW STEM AUGER BY: RMA

DEPTH IN FEET	SAMPLE NO.	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				**AC: 4" BASE: 6"** **ARTIFICIAL FILL** Sandy Silt, soft, slightly moist, dark brown, fine- to medium-grained.			
2	B4@2.5'				7	109.4	17.3
5	B4@5'		SM	**ALLUVIUM** Silty Sand, medium dense, slightly moist, dark grayish brown, fine- to medium-grained, trace fine gravel.	29	121.0	15.0
9			SP	Sand, poorly graded, medium dense, slightly moist, dark brown, fine- to medium-grained, trace silt.			
10	B4@10'				34	116.3	7.3
12.5	B4@12.5'			- grayish brown	32	104.6	14.4
14			SM	Silty Sand, loose, slightly moist, grayish brown, fine-grained.			
15	B4@15'				15	100.4	21.4
17			ML	Sandy Silt, firm, slightly moist, light gray, fine-grained.			
17.5	B4@17.5'				17	104.1	19.4
20	B4@20' BULK 20-25'		SM	Silty Sand, medium dense, slightly moist, grayish brown, fine-grained.	19	112.6	17.7
25	B4@25'	▼		- fine- to medium-grained, trace fine gravel	29	110.9	17.0

Figure A4,
Log of Boring 4, Page 1 of 2

A9713-06-01 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

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**BORING 4** ELEV. (MSL.) **--** DATE COMPLETED **1/3/18** EQUIPMENT **HOLLOW STEM AUGER** BY: **RMA**	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
30	B4@30'			SM	- fine-grained, no gravel	36	119.2	15.0
					Total depth of boring: 30.5 feet Fill to 5 feet. Groundwater encountered at 25 feet. Backfilled with soil cuttings and tamped. Asphalt patced. *Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.			

Figure A4,
Log of Boring 4, Page 2 of 2

A9713-06-01 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

BORING 5

ELEV. (MSL.) -- DATE COMPLETED 1/3/18

EQUIPMENT HOLLOW STEM AUGER BY: RMA

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**AC: 4" BASE: 6"** **ARTIFICIAL FILL** Silty Sand, loose, slightly moist, dark grayish brown with light brown mottles, fine- to medium-grained.			
5	B5@5'				**ALLUVIUM** Silty Sand, medium dense, slightly moist, dark brown, fine- to medium-grained, trace clay.	34	111.2	17.7
9				SM	- increase in silt content			
10	B5@10'					24	113.5	14.0
15	B5@15'			ML	Silt, stiff, slightly moist, grayish brown, trace clay.	35	113.3	18.5
19					Silty Sand, medium dense, grayish brown, fine-grained.			
20	B5@20'			SM		21	111.5	17.6
24				SP	Sand, poorly graded, medium dense, slightly moist, light gray, fine-grained, trace clay.			
25	B5@25'					25	115.0	17.5

Total depth of boring: 25.5 feet
Fill to 5 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.
Asphalt patched.
*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.

Figure A5,
Log of Boring 5, Page 1 of 1

A9713-06-01 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

BORING 6

ELEV. (MSL.) -- DATE COMPLETED 1/3/18

EQUIPMENT HOLLOW STEM AUGER BY: RMA

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**AC: 4" BASE: 4"** **ARTIFICIAL FILL** Silty Sand, loose, slightly moist, dark brown, fine- to medium-grained, trace clay.			
5	B6@5'					17	108.9	14.3
				SP	**ALLUVIUM** Sand, poorly graded, medium dense, slightly moist, grayish brown, fine- to medium-grained, trace coarse-grained, trace silt.			
10	B6@10'			SM	Silty Sand, medium dense, slightly moist, grayish brown, fine-grained, trace clay.	23	113.5	5.6
15	B6@15'					21	109.4	19.4

Total depth of boring: 15.5 feet
Fill to 5.5 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.
Asphalt patched.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.

Figure A6,
Log of Boring 6, Page 1 of 1

A9713-06-01 BORING LOGS.GPJ

SAMPLE SYMBOLS
- ... SAMPLING UNSUCCESSFUL
- ... STANDARD PENETRATION TEST
- ... DRIVE SAMPLE (UNDISTURBED)
- ... DISTURBED OR BAG SAMPLE
- ... CHUNK SAMPLE
- ... WATER TABLE OR SEEPAGE

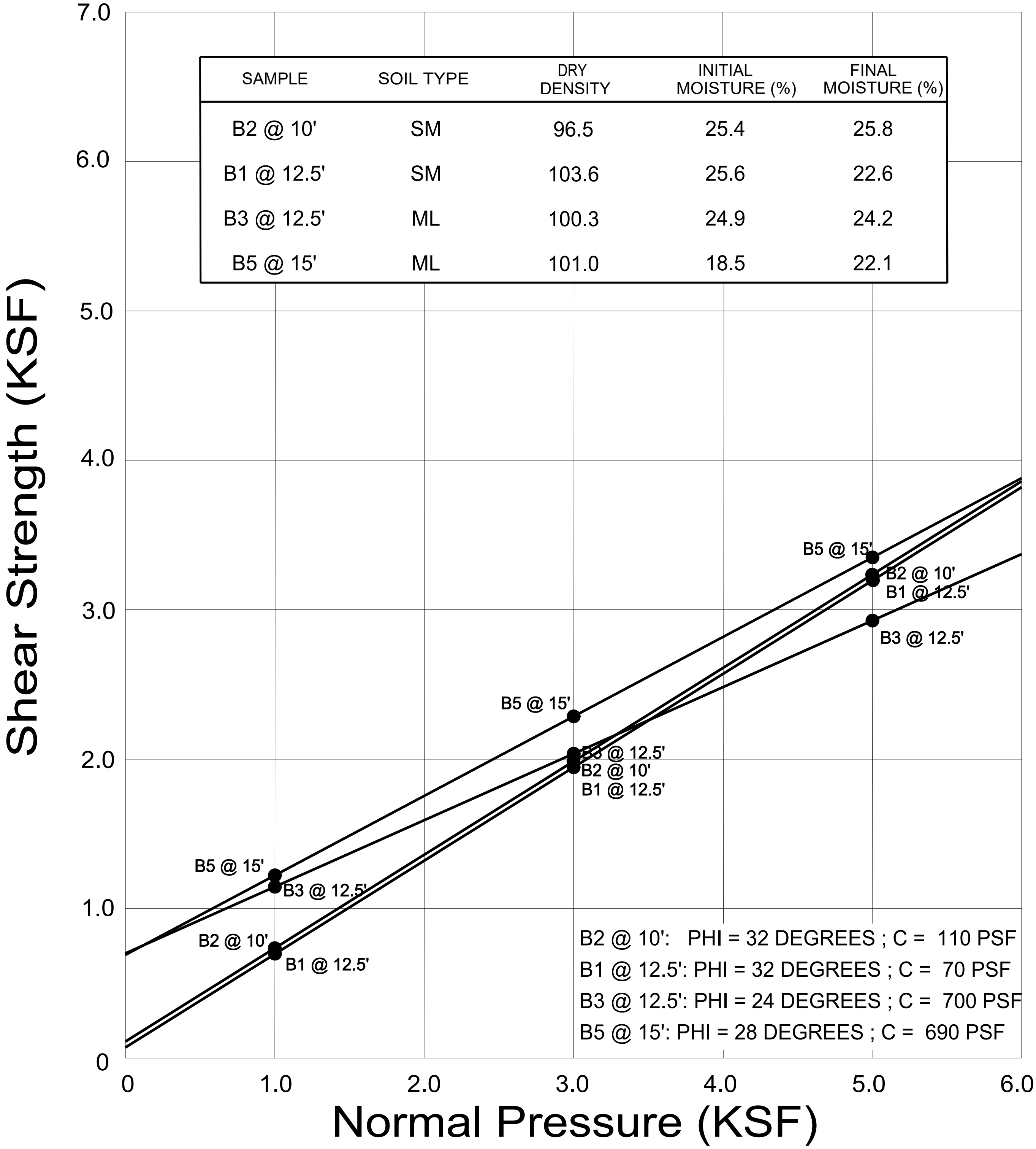
NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

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, corrosivity, in-place dry density and moisture content. The results of the laboratory tests are summarized in Figures B1 through B8. The in-place dry density and moisture content of the samples tested are presented on the boring logs, Appendix A.



SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2 @ 10'	SM	96.5	25.4	25.8
B1 @ 12.5'	SM	103.6	25.6	22.6
B3 @ 12.5'	ML	100.3	24.9	24.2
B5 @ 15'	ML	101.0	18.5	22.1

● Direct Shear, Saturated

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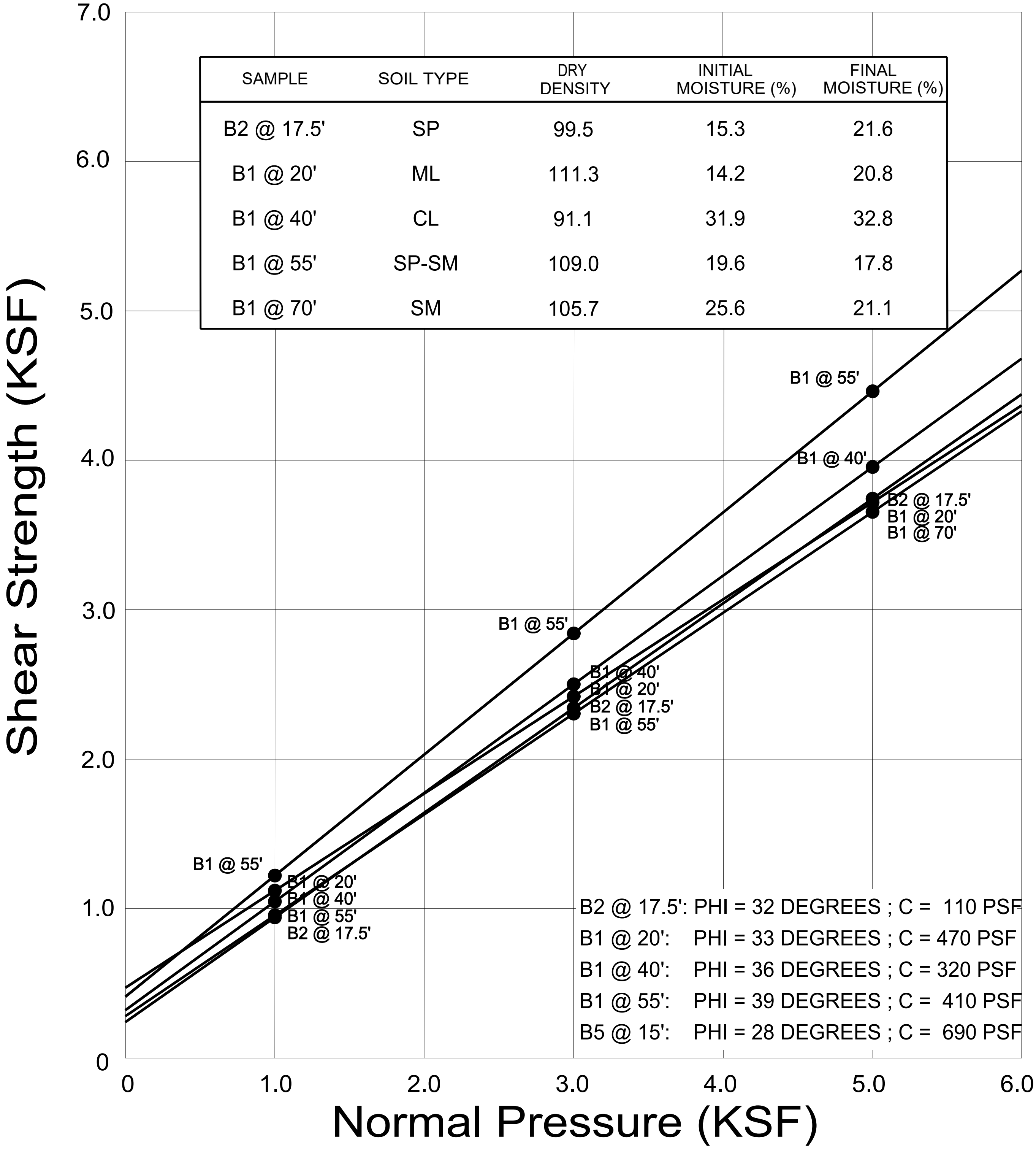
DIRECT SHEAR TEST RESULTS

HOLLAND ACQUISITION CO, LLC
3RD STREET AND FAIRFAX AVENUE
LOS ANGELES, CALIFORNIA

DRAFTED BY: JTA	CHECKED BY: NDB	NOV 2018	PROJECT NO. A9713-06-01	FIG. B1



SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2 @ 17.5'	SP	99.5	15.3	21.6
B1 @ 20'	ML	111.3	14.2	20.8
B1 @ 40'	CL	91.1	31.9	32.8
B1 @ 55'	SP-SM	109.0	19.6	17.8
B1 @ 70'	SM	105.7	25.6	21.1

● Direct Shear, Saturated

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DIRECT SHEAR TEST RESULTS

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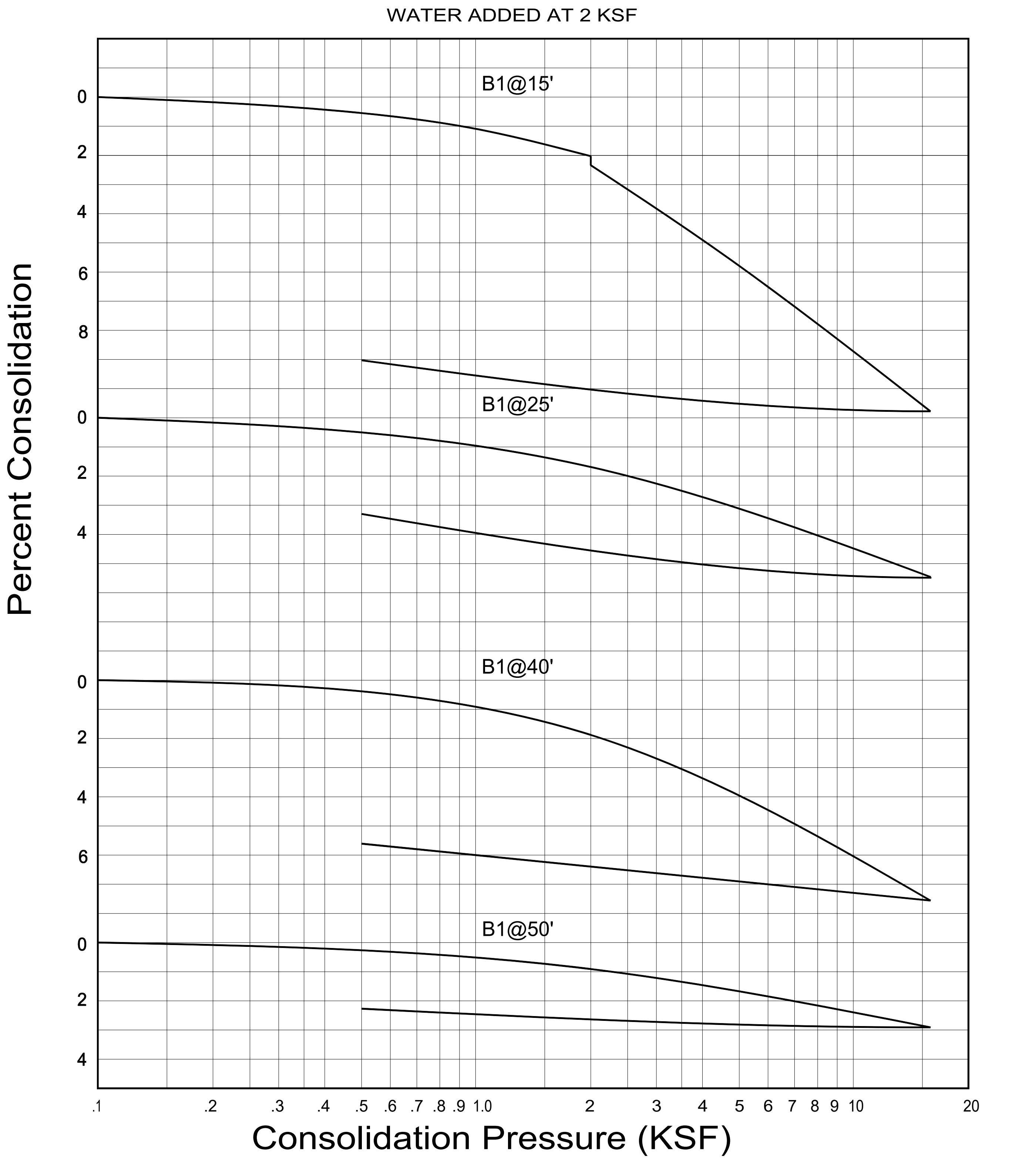
LOS ANGELES, CALIFORNIA

DRAFTED BY: JTA	CHECKED BY: NDB	NOV 2018	PROJECT NO. A9713-06-01	FIG. B2



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CONSOLIDATION TEST RESULTS

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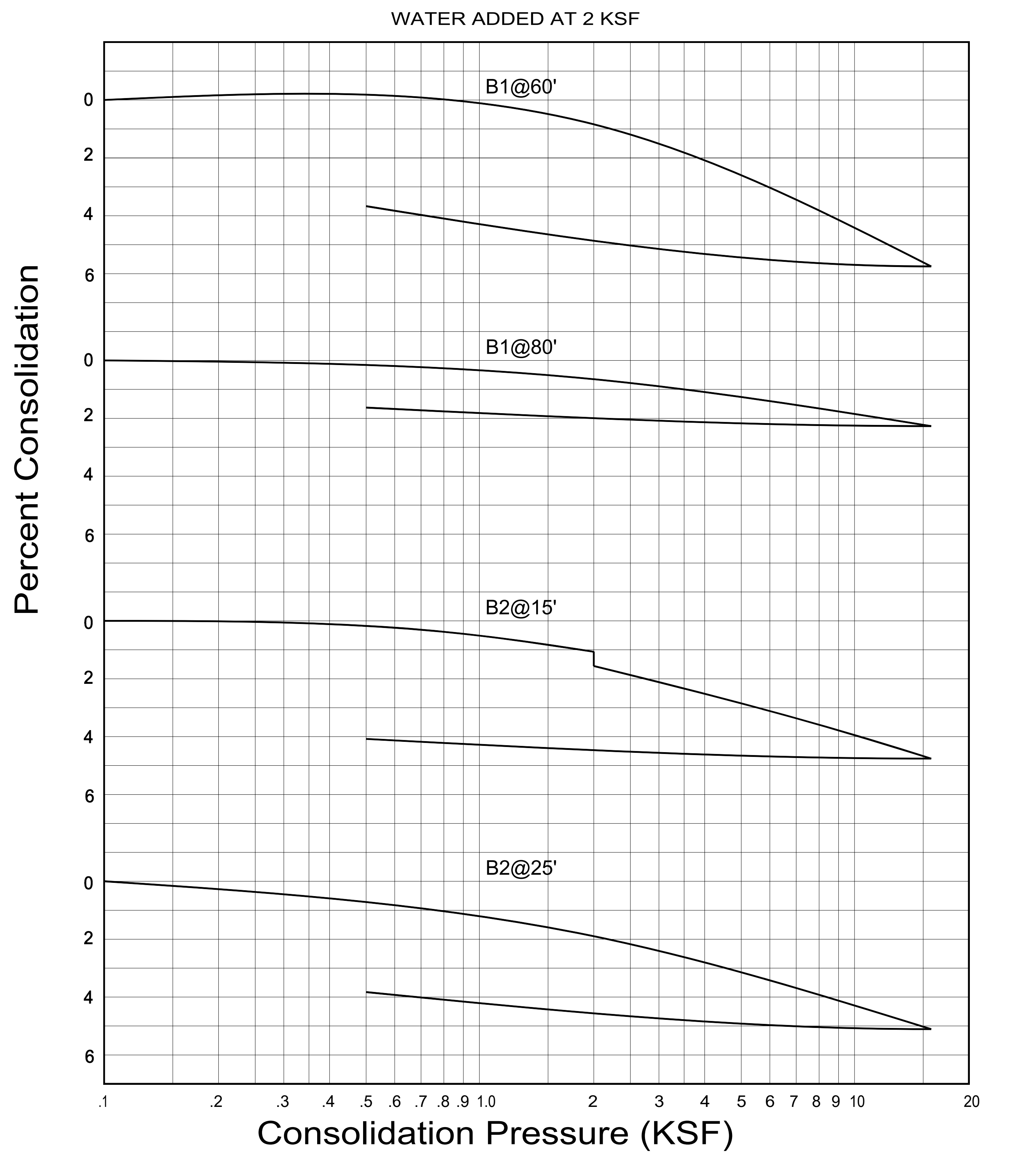
3RD STREET AND FAIRFAX AVENUE
LOS ANGELES, CALIFORNIA

Drafted by: JTA	Checked by: NDB	NOV 2018	PROJECT NO. A9713-06-01	FIG. B3



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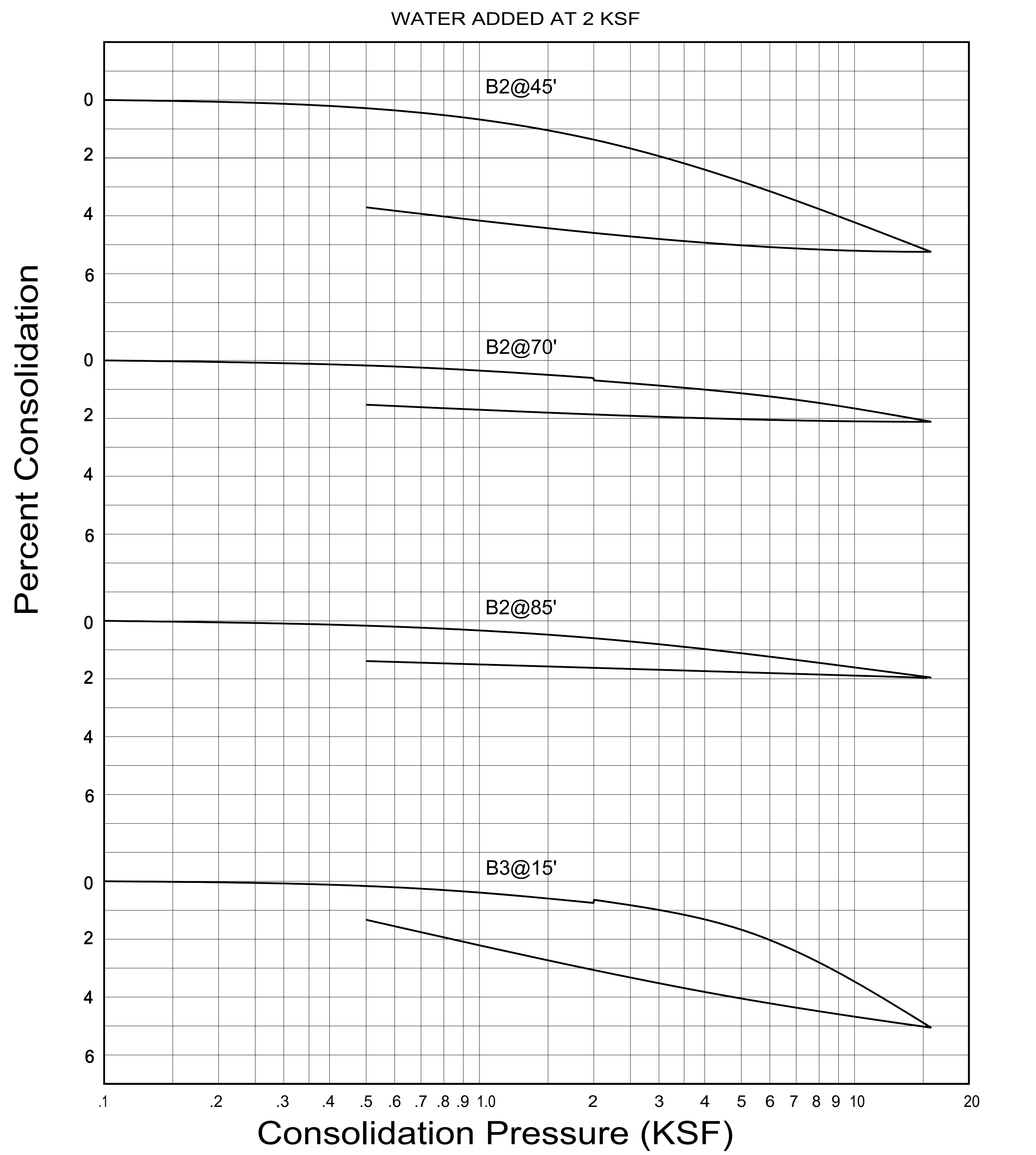
CONSOLIDATION TEST RESULTS

HOLLAND ACQUISITION CO, LLC
3RD STREET AND FAIRFAX AVENUE
LOS ANGELES, CALIFORNIA

NOV 2018	PROJECT NO. A9713-06-01	FIG. B4



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CONSOLIDATION TEST RESULTS

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3RD STREET AND FAIRFAX AVENUE

LOS ANGELES, CALIFORNIA

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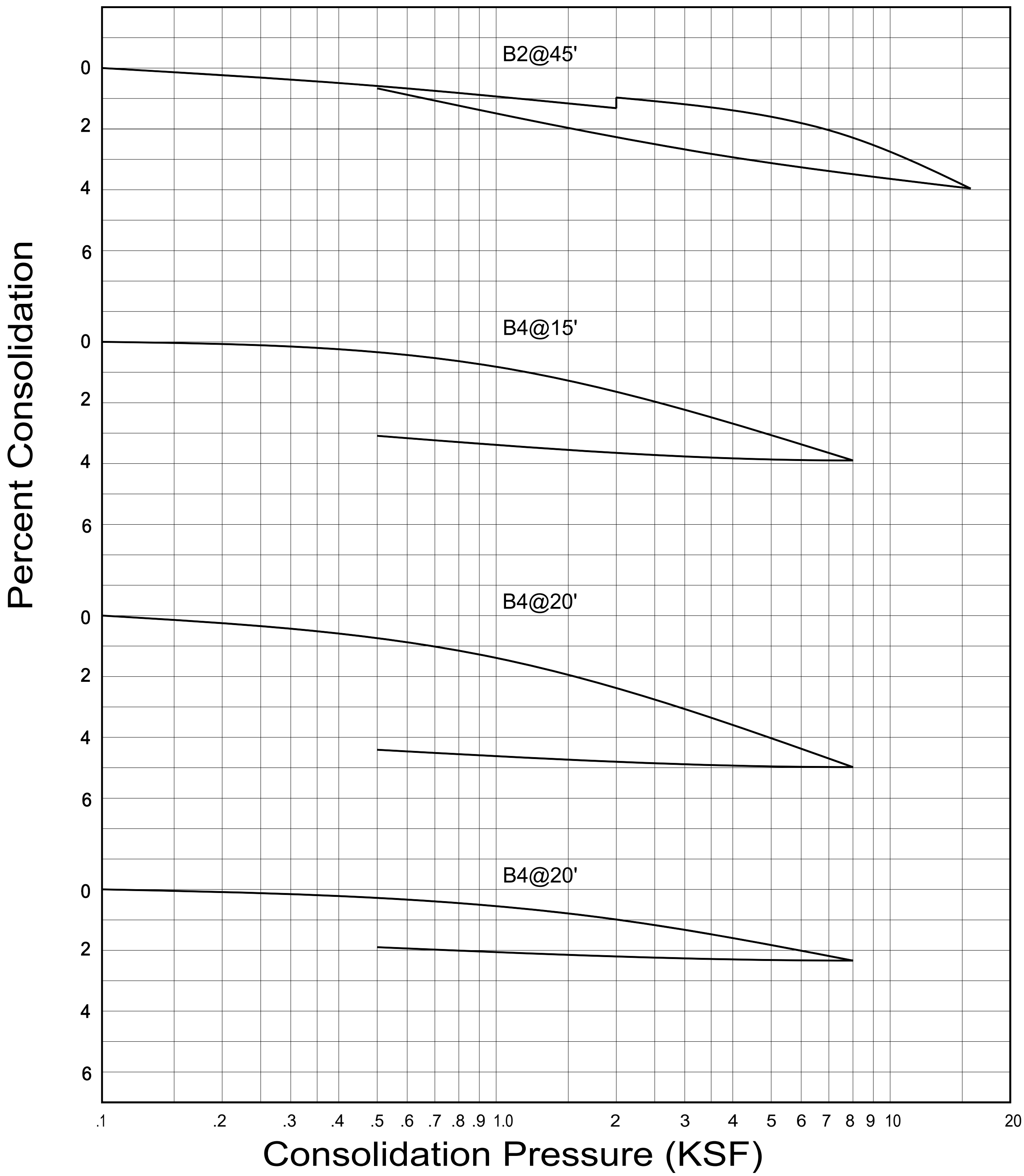
WATER ADDED AT 2 KSF



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Drafted by: JTA	Checked by: NDB

CONSOLIDATION TEST RESULTS

HOLLAND ACQUISITION CO, LLC
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LOS ANGELES, CALIFORNIA

NOV 2018	PROJECT NO. A9713-06-01	FIG. B6

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				
B3 @ 0-5'	8.0	14.6	116.0	17	Very Low	Non-Expansive

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

** Reference: 2016 California Building Code, Section 1803.5.3

SUMMARY OF LABORATORY MAXIMUM DENSITY AND
AND OPTIMUM MOISTURE CONTENT TEST RESULTS
ASTM D 1557-12

Sample No.	Soil Description	Maximum Dry Density (pcf)	Optimum Moisture (%)
B3 @ 0-5'	Dark Brown Silty Sand	131.0	8.5

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LABORATORY TEST RESULTS

HOLLAND ACQUISITION CO, LLC
3RD STREET AND FAIRFAX AVENUE
LOS ANGELES, CALIFORNIA

Drafted by: JTA	Checked by: NDB	NOV 2018	PROJECT NO. A9713-06-01	FIG. B7

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

Sample No.	**pH**	**Resistivity (ohm centimeters)**
B4 @ 20-25'	9.14	1800 (Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS EPA NO. 325.3

Sample No.	Chloride Ion Content (%)
B4 @ 20-25'	0.012

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

Sample No.	Water Soluble Sulfate (% SO_4)	Sulfate Exposure*
B4 @ 20-25'	0.002	Negligible

* Reference: 2016 California Building Code, Section 1904.3 and ACI 318-11 Section 4.3.

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CORROSIVITY TEST RESULTS

HOLLAND ACQUISITION CO, LLC
3RD STREET AND FAIRFAX AVENUE
LOS ANGELES, CALIFORNIA

Drafted by: JTA	Checked by: NDB	NOV 2018	PROJECT NO. A9713-06-01	FIG. B8

APPENDIX C

APPENDIX C

PRIOR GEOTECHNICAL REPORT

PRELIMINARY GEOTECHNICAL ENGINEERING INVESTIGATION PROPOSED MIXED USE DEVELOPMENT SEC OF 3RD & FAIRFAX AVENUE LOS ANGELES, CALIFORNIA

PROJECT NO. 112-17043
JULY 31, 2017

Prepared for:

MR. JOHN HAYES
REGENCY CENTERS, INC.
915 WILSHIRE BOULEVARD, SUITE 2200
LOS ANGELES, CALIFORNIA 90017

Prepared by:

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(951) 273-1011

TABLE OF CONTENTS

Krazan & ASSOCIATES, INC.

GEOTECHNICAL ENGINEERING • ENVIRONMENTAL ENGINEERING
CONSTRUCTION TESTING & INSPECTION

July 31, 2017

KA Project No. 112-17043

**PRELIMINARY GEOTECHNICAL
ENGINEERING INVESTIGATION
PROPOSED MIXED USE DEVELOPMENT
SOUTHEAST CORNER OF WEST 3RD STREET AND FAIRFAX AVENUE
LOS ANGELES, CALIFORNIA**

INTRODUCTION

This report presents the results of our Preliminary Geotechnical Engineering Investigation for the Proposed Mixed Use Development to be located at the southeast corner of West 3rd Street and Fairfax Avenue, in the City of Los Angeles, California. Discussions regarding site conditions are presented herein, together with conclusions and recommendations pertaining to site preparation, Engineered Fill, utility trench backfill, drainage and landscaping, foundations, concrete floor slabs and exterior flatwork, retaining walls, and soil cement reactivity.

A site plan showing the approximate boring locations is presented following the text of this report. A description of the field investigation, boring logs, and the boring log legend are presented in Appendix A. Appendix A also contains a description of the laboratory testing phase of this study, along with the laboratory test results. Appendices B and C contain guides to earthwork and pavement specifications. When conflicts in the text of the report occur with the general specifications in the appendices, the recommendations in the text of the report have precedence.

PURPOSE AND SCOPE

This investigation was conducted to evaluate the soil and groundwater conditions at the subject site, to make geotechnical engineering recommendations for use in design of specific construction elements, and to provide criteria for site preparation and Engineered Fill construction.

Our scope of services was outlined in our proposal dated June 2, 2017 (KA Proposal No. G17053CAC) and included the following:

- A site reconnaissance by a member of our engineering staff to evaluate the surface conditions at the project site.
- A field investigation consisting of drilling eleven (11) borings to depths ranging from approximately 4 to 80 feet for evaluation of the subsurface conditions at the project site.

- Performance of laboratory tests on representative soil samples obtained from the borings to evaluate the physical and index properties of the subsurface soils.

- Performance of two (2) infiltration tests at the subject site in order to determine an estimated infiltration rates for the near surface soil conditions.

- Evaluation of the data obtained from the investigation and an engineering analysis to provide recommendations for use in the project design and preparation of construction specifications.

- Preparation of this report summarizing the results, conclusions, recommendations, and findings of our investigation.

Environmental services, such as a chemical analysis of soil and groundwater for possible environmental contaminates, were not in our scope of services.

PROPOSED CONSTRUCTION

We understand that design of the proposed development is currently underway and as such, structural load information and other final details pertaining the structure are unavailable. On a preliminary basis, it is understood the development will consist of construction of a Mixed Use Residential and Commercial Development with a footprint of approximately 73,000 square feet. The proposed building is understood to include up to 6-stories above grade and up to 3 levels of subterranean parking. The proposed building is understood to be a wood and/or steel framed structure. The proposed subterranean parking is anticipated to be Portland cement and/or masonry construction. Footing loads are anticipated to be moderate. It is anticipated that the structures will utilize conventional shallow foundations or mat foundation systems and a concrete slab-on-grade. Additional improvements may include remodeling of existing structures located at the subject site, underground utilities, and pavement rehabilitation.

In the event, these structural or grading details are inconsistent with the final design criteria, the Soils Engineer should be notified so that we may update this writing as applicable.

SITE LOCATION AND SITE DESCRIPTION

The subject site is rectangular in shape and encompasses an area of approximately 7.3 acres. The site is located at the southeast corner of West 3rd Street and Fairfax Avenue in the City of Los Angeles, California.

The site is bound to the north by West 3rd Street and the Farmers Market beyond, to the east by South Ogden Drive and a retail center beyond, to the south by Hancock Park Elementary School, and to the west by South Fairfax Avenue and some retail beyond. Presently, the site is occupied by commercial, retail, restaurants, bank buildings, and a grade level parking lot.

Trees and shrubs are located within isolated landscape planters along the edge of the existing structure and in between parking stalls. Buried utilities are located throughout and along the perimeter of the site. The site is relatively flat and level with no major changes in grade.

GEOLOGIC SETTING

The subject property is located within the Los Angeles Coastal Plain, which is situated between the Santa Monica Mountains to the northwest, the San Gabriel Mountains to the northeast, the San Bernardino Mountains to the east, and the Pacific Ocean to the west and south. Unconsolidated materials found in the vicinity of the subject site are generally composed of alluvial deposits derived from the surrounding mountain ranges to the northwest, north, and northeast of the Los Angeles Coastal Plain. Sediments currently at or near the surface are believed to be of Quaternary Age (2 million years old or younger). Deposits encountered on the subject site during exploratory drilling are discussed in detail in this report.

The subject site occupies the westerly extent of the La Brea Plain. The La Brea Plain is a broad, slightly elevated, and dissected surface underlain by coalescing Quaternary age alluvial fan and flood plain deposits. These alluvial sediments were deposited on the underlying Tertiary-age shallow marine sedimentary bedrock formations. Faulting and folding of the bedrock over millions of years has formed structural traps where petroleum deposits have accumulated in anticlinal folds and along fault blocks. Several oil and gas fields developed within this portion of the Los Angeles Basin, including the nearby Salt Lake and South Salt Lake fields.

The oil deposits are found at depths exceeding about 1,000 feet. Crude oil and methane gas leaking from the petroleum deposits has migrated towards the ground surface through fractures and faults in the bedrock, permeating into the overlying alluvium. Upon reaching shallower depths, the lighter petroleum components are altered by evaporation and biologic processes resulting in a more viscous remnant tar deposit, such as those exposed at the La Brea Tar Pits east of the subject site.

The property is located on the Los Angeles Basin (Morton, et al., 1974). Southern California is seismically active and will experience future earthquakes that will affect the project site. The earthquakes are predominately generated by periodic slip along the northwesterly trending faults associated with the San Andreas Fault system and the east-west trending faults along the northern margin of the Los Angeles Basin. In addition to these probable earthquake sources, recent earthquakes in the region have occurred on previously unknown faults having no surface expression (1987 Whittier Narrows and the 1994 Northridge earthquakes). The Seismic hazard most likely to impact the site is groundshaking due to a large earthquake on one of the major active regional faults. The Santa Monica fault is the nearest active fault to the site, and is located approximately 0.8 miles to the north. The Hollywood, Newport-Inglewood, and Puente Hills faults are located approximately 2.1, 2.6, and 3.0 miles from the site, respectively. Secondary hazards of earthquakes include rupture, seiche, landslides, liquefaction, and subsidence. Since there are no known faults within the immediate area, ground rupture from surface faulting is not anticipated to be a potential hazard for the site. Seiche and landslides are not considered potential hazards for the site either. The area in consideration shows no mapped faults on-site according to maps prepared by the California Division of Mines and Geology (now known as the California Geologic Survey) and published by the International Conference of Building Officials (ICBO). No evidence of surface faulting was observed on the property during our reconnaissance.

FIELD AND LABORATORY INVESTIGATION

Subsurface soil conditions were explored by drilling eleven (11) borings to depths ranging from approximately 4 to 80 feet below existing site grade, using a truck-mounted drill rig. The approximate boring and bulk sample locations are shown on the site plan. During drilling operations, penetration tests were performed at regular intervals to evaluate the soil consistency and to obtain information regarding the engineering properties of the subsurface soils. Soil samples were retained for laboratory testing. The soils encountered were continuously examined and visually classified in accordance with the Unified Soil Classification System. A more detailed description of the field investigation is presented in Appendix A.

Laboratory tests were performed on selected soil samples to evaluate their physical characteristics and engineering properties. The laboratory testing program was formulated with emphasis on the evaluation of natural moisture, density, gradation, shear strength, consolidation potential, expansion potential, and moisture-density relationships of the materials encountered. In addition, chemical tests were performed to evaluate the corrosivity of the soils to buried concrete and metal. Details of the laboratory test program and results of the laboratory tests are summarized in Appendix A. This information, along with the field observations, was used to prepare the final boring logs in Appendix A.

SOIL PROFILE AND SUBSURFACE CONDITIONS

Based on our findings, the subsurface conditions encountered at the boring locations appear typical of those found in the geologic region of the site. The areas of the site not occupied by the existing structures are covered with approximately 4 inches of asphalt pavement underlain by approximately 6 inches of discernable base material. Below the existing pavements, stiff to very stiff sandy clay was encountered to depths of up to 19 feet below existing site grades. Below the near surface sandy clay soil, medium dense to dense silty sand was encountered from depths of approximately 14 feet below site grade to approximately 29 feet below site grade. Below the silty sand, medium dense to dense poorly-graded sand was encountered from depths of approximately 19 feet below site grade to approximately 24 feet below site grade. Below the poorly-graded sand, medium stiff to hard sandy silt was encountered from depths of approximately 24 feet below site grade to approximately 32 feet below site grade. Below the sandy silt, stiff to hard sandy clay was encountered from depths of approximately 29 feet below site grade to approximately 44 feet below site grade. Below the sandy clay, medium dense clayey sand was encountered from depths of approximately 43 feet below site grade to approximately 54 feet below site grade. Finally, below the clayey sand, medium dense asphalt sand was encountered from depths of approximately 53 feet below site grade to the maximum depth explored, 80 feet below site grade.

Penetration resistance, measured by the number of blows required to drive a Modified California sampler or Standard Penetration Test (SPT) ranged from 11 to 75 blows per foot. Dry densities ranged from approximately 102 to 124 pcf. Representative samples of the near surface soils consolidated approximately 1.0 to 2.0 percent under a 2 ksf load when saturated. Representative soil samples of near the surface had angles of internal friction of 27 and 28. Representative soil samples from 30 feet below site grades had angles of internal friction of 26 and 27 with a cohesion of 100 psf. Representative near surface samples were found to have expansive indices of 55 and 58.

For additional information about the soils encountered, please refer to the logs of borings in Appendix A.

GROUNDWATER

Test boring locations were checked for the presence of groundwater during and immediately following the drilling operations. Free groundwater was encountered at a depth of approximately 20 feet below existing site grade. Information provided by the California Geological Survey indicates that the historical high groundwater depth within the project site vicinity has been found on the order of 10 feet below existing site grades.

It should be recognized that water table elevations may fluctuate with time, being dependent upon seasonal precipitation, irrigation, land use and climatic conditions, as well as other factors. Therefore, water level observations at the time of the field investigation may vary from those encountered during the construction phase of the project. The evaluation of such factors is beyond the scope of this report.

SOIL LIQUEFACTION

Soil liquefaction is a state of soil particle suspension caused by a complete loss of strength when the effective stress drops to zero. Liquefaction normally occurs under saturated conditions in soils such as sand in which the strength is purely frictional. However, liquefaction has occurred in soils other than clean sand. Liquefaction usually occurs under vibratory conditions such as those induced by a seismic event.

To evaluate the liquefaction potential of the site, the following items were evaluated:

1) Soil type
2) Groundwater depth
3) Relative density
4) Initial confining pressure
5) Intensity and duration of groundshaking

The soils encountered within a depth of 80 feet on the project site predominately consist of medium stiff to hard sandy clay and sandy silt, medium dense to very dense clayey sand and silty sand, and dense to very dense sand and asphalt sand. Moderate cohesion strength is associated with the clayey soils. Groundwater was encountered below the site at a depth of approximately 20 feet during subsurface exploration. Available groundwater depth mapping, as well as our experience in the area, indicates that historically groundwater has been located at depths on the order of 10 feet below grade in the general vicinity of the site.

The subject site is not located in an area designated by the State of California as a Seismic Hazard Zone. Due to the cohesive nature of the near surface soil encountered at the subject site and the relative consistency of the underlying soil, liquefaction is not considered to be a significant concern for the subject site.

FAULT RUPTURE HAZARD ZONES

The Alquist-Priolo Geologic Hazard Zones Act went into effect in March, 1973. Since that time, the Act has been amended 11 times (Hart, 2007). The purpose of the Act, as provided in California Geologic Survey (CGS) Special Publication 42 (SP 42), is to "prohibit the location of most structures for human occupancy across the traces of active faults and to mitigate thereby the hazard of fault-rupture." The Act was renamed the Alquist-Priolo Earthquake Fault Zoning Act in 1994, and at that time, the originally designated "Special Studies Zones" was renamed the "Earthquake Fault Zones."

The subject site is located on the Hollywood Quadrangle, Earthquake Fault Zone Map, dated November 6, 2014. The nearest significant active fault is the Santa Monica Fault Zone, which is located approximately 0.8 miles from the site. The area in consideration shows no mapped faults on-site according to maps prepared by the California Geologic Survey. No evidence of surface faulting was observed on the property during our reconnaissance.

SEISMIC HAZARDS ZONES

In 1990, the California State Legislature passed the Seismic Hazard Mapping Act to protect public safety from the effects of strong shaking, liquefaction, landslides, or other ground failure, and other hazards caused by earthquakes. The Act requires that the State Geologist delineate various seismic hazard zones on Seismic Hazard Zones Maps. Specifically, the maps identify areas where soil liquefaction and earthquake-induced landslides are most likely to occur. A site-specific geotechnical evaluation is required prior to permitting most urban developments within the mapped zones. The Act also requires sellers of real property within the zones to disclose this fact to potential buyers. The subject site is located on the Hollywood Quadrangle, Seismic Hazard Zones Map, dated March 25, 1999. The subject site is not located in an area designated by the State of California as a Seismic Hazard Zone.

OTHER HAZARDS

Rockfall, Landslide, Slope Instability, and Debris Flow: The subject site is relatively flat and level. It is our understanding that there are no significant slopes proposed as part of the proposed development. Provided the recommendations presented in this report are implemented into the design and construction of the anticipated development, rockfalls, landslides, slope instability, and debris flows are not anticipated to pose a hazard to the subject site.

Seiches: Seiches are large waves generated within enclosed bodies of water. The site is not located in close proximity to any lakes or reservoirs. As such, seiches are not anticipated to pose a hazard to the subject site.

Tsunamis: Tsunamis are tidal waves generated by fault displacement or major ground movement. The site is several miles from the ocean. As such, tsunamis are not anticipated to pose a hazard to the subject site.

Hydroconsolidation: The near surface soils encountered at the subject site were found to be stiff to very stiff. The underlying native soils were found to be medium dense to dense. Provided the recommendations in this report are incorporated into the design and construction of the proposed development, hydroconsolidation is not anticipated to be a significant concern for the subject site.

SEISMIC SETTLEMENT

The native soils within the project site are not conducive to hydro-collapse due to the relatively dense soil conditions, low void ratio and moderate to high penetration resistance measured. Any loose fill materials at the site could be vulnerable to hydro-collapse. However, the hazard can be mitigated by following the design and construction recommendations of current and future Geotechnical Engineering Investigation Reports.

The proposed development will include grading of the subject site and surrounding areas to construct a relatively level site with subsurface parking. Retaining walls will be used in the proposed construction. Groundwater has historically been encountered at depths greater than 10 feet below existing site grade. Provided the planned grading complies with the current code requirements and the recommendations of current and future Geotechnical Engineering Investigation reports, the site will not likely be subject to lateral spreading hazards.

SOIL CORROSIVITY

Corrosion tests were performed to evaluate the soil corrosivity to the buried structures. The tests consisted of sulfate content, chloride content, and resistivity and the results of the tests are included as follows:

Parameter	Results	Test Method
Resistivity	3,100 ohm-cm	CA 643
Sulfate	160 ppm	CA 417
Chloride	139 ppm	CA 422
pH	6.9	EPA 9045C

EXPANSION POTENTIAL

The results of laboratory testing performed on near surface soil samples collected from the subject site indicate the expansion potential of the sandy clay soils were 55 and 58 and therefore classified as possessing a "medium" expansion potential. The California Building Code (CBC) defines soil with an expansion index greater than 20 as expansive. The near surface soils present at the subject site generally possess expansion potentials in excess of 20 and therefore should be considered expansive.

OIL WELLS

According to the data collected from the California Division of Oil, Gas & Geothermal Resources website, the subject site is located within the limits of the Salt Lake Oil Field. There are 2 reportedly abandoned wells on-site in the existing parking lot area. One well is located at the west portion of the site numbered 102 and the other well is located on the east portion of the site numbered 99. Both wells were reportedly operated by Chevron. No other abandoned or active oil wells are displayed within the area of the subject site. Although the likelihood of encountering an abandoned oil well is low, mitigation is recommended in the event an oil well is encountered.

METHANE GAS

The subject site is located within an area of known shallow methane accumulation. Information regarding methane gas mitigation should be obtained by consulting with an expert professional.

PETROLEUM IMPACTED GROUNDWATER

Shallow groundwater is present at the subject site. Groundwater will need to be collected during dewatering associated with the excavation of the underground parking basement. Due to the potential presence of asphalt sands and potential for impacted groundwater, extracted groundwater should be chemically analyzed in order to determine the appropriate treatment or disposal methods.

SUBSIDENCE

Subsidence of the ground surface can be caused by the removal of groundwater and/or petroleum from the subsurface. If in sufficient volumes, the extraction the pore fluid can cause permanent collapse of the pore space due to consolidation and potentially damage structural improvements.

The subject site is within the limits of the Salt Lake Oil Field, however evidence of subsidence in the vicinity of the subject site was not found due to the extraction of petroleum. Likewise, subsidence due to the extraction of free groundwater was not found.

Temporary dewatering is anticipated during the excavation and construction of the underground parking basement. Groundwater extracted during this temporary dewatering will be relatively small volumes to produce a localized drawdown around the excavation. The relatively stiff/dense soils below the proposed parking basement are unlikely to settle from the temporary dewatering. Therefore, the subsidence related to groundwater removal is not considered a significant impact to the subject site.

CONCLUSIONS AND RECOMMENDATIONS

Based on the findings of our field and laboratory investigations, along with previous geotechnical experience in the project area, the following is a summary of our evaluations, conclusions, and recommendations.

Administrative Summary

In brief, the subject site and soil conditions, with the exception to the relatively shallow groundwater, medium expansion potential of the on-site clayey soils, and existing development, appear to be conducive to the development of the project.

The organic-free, on-site, upper native soils are predominately sandy clay and clay. The underlying silty sand and sandy silt soils that do not contain clay will be suitable for reuse as Non-Expansive Engineered Fill provided they are cleansed of excessive organics and debris. The clayey soils will not be suitable for reuse as Non-Expansive Engineered Fill. The clayey soils should be at or above optimum moisture-content during mixing operations. These clayey soils will be suitable for reuse as General Engineered Fill, within flexible pavement areas, structural areas supported by structural slabs, and below 24 inches from finished grade in building areas, provided they are cleansed of excessive organics, debris, and moisture-conditioned to at least 2 percent above optimum moisture-content. It is recommended that additional testing be performed on the on-site soils and fill material to evaluate the physical and index properties prior to reuse as Engineered Fill.

The clayey soils appear to have a high swell potential. The clayey soils in their present condition present a minor to moderate hazard to construction in terms of possible post-construction movement of slab-on-grade construction. To reduce potential soil movement related to the swell potential of the clayey soils, it is recommended that slab-on-grade and exterior flatwork areas be supported by at least 24 inches of Non-Expansive Engineered Fill. The fill material should be a well-graded silty sand or sandy silt soil. A clean sand or very sandy soil is not acceptable for this purpose. A sandy soil will allow the surface water to drain into the expansive soils below, which may result in soil swelling. The replacement soils and/or upper 24 inches of Imported Fill soils should meet the specifications as described under the subheading Engineered Fill. The replacement soils should extend 5 feet beyond the perimeter of slab-on-grade areas. The non-expansive replacement soils should be compacted to at least 90 percent of the relative compaction based on ASTM Test Method D1557. The exposed native soils in the excavation should not be allowed to dry out and should be kept continually moist, prior to backfilling. In addition, it is recommended that slab-on-grade, continuous footings and slabs be nominally reinforced to reduce cracking and vertical off-set.

The site is presently occupied by an existing commercial plaza with as asphalt paved parking lot. Fill may be present at the site. Any fill soil encountered should be excavated and stockpiled so that the native soils can be properly prepared. The clayey soils encountered at the site will not be suitable for reuse as Non-Expansive Engineered Fill. However, these clayey soils will be suitable for reuse as General Engineered Fill, provided they are cleansed of excessive organics and debris, and are moisture-conditioned to a minimum of 2 percent above optimum moisture-content. Prior to fill placement Krazan & Associates, Inc. should inspect the bottom of the excavation to verify no additional removal will be required.

Trees and shrubs are located within the project site. Tree and shrub removal operations should include roots greater than ½ inch in diameter. The resulting excavations should be backfilled with Engineered Fill compacted to a minimum of 90 percent of maximum dry density based on ASTM Test Method D1557.

After completion of the recommended site preparation and over-excavation, the site should be suitable for shallow footing support. The proposed structure footings may be designed utilizing an allowable bearing pressure of 2,500 psf for dead-plus-live loads. Footings should have a minimum embedment of 24 inches. As an alternative, the proposed structure may be designed utilizing a mat or structural slab system.

Foundations supported at deeper elevations may require the use of a structural mat foundation. Deeper excavations will likely require dewatering and soil stabilization in order to address saturated soil conditions and provide for a stable foundation bearing grade.

For preliminary purposes, an allowable bearing pressure of 1,000 pounds per square foot may be used for design of the slab. For preliminary modeling purposes a vertical modulus of subgrade reaction (Kv1), also referred to as a soil spring, of 30 pounds per square inch per inch may be used for long term conditions. An increased modulus of 40 pounds per square inch per inch may be used for short term loading to evaluate punching shear at columns and walls. The slab design should ultimately limit slab bending or arching in the lightly loaded mid-slab areas between load bearing columns and walls. Based on the preliminary nature of the project design and a lack of formal design documents, these values should be considered preliminary and should be reevaluated during final design. The values should be reevaluated in order to determine soil support values appropriate for the actual design conditions.

Walls retaining horizontal backfill and capable of deflecting a minimum of 0.1 percent of its height at the top may be designed using an equivalent fluid active pressure of 47 pounds per square foot per foot of depth. Walls that are incapable of this deflection or walls that are fully constrained against deflection may be designed for an equivalent fluid at-rest pressure of 68 pounds per square foot per foot of depth. Expansive soils should not be used for backfill against walls. The wedge of non-expansive backfill material should extend from the bottom of each retaining wall outward and upward at a slope of 2:1 (horizontal to vertical) or flatter. The stated lateral earth pressures do not include the effects of hydrostatic water pressures generated by infiltrating surface water that may accumulate behind the retaining walls; or loads imposed by construction equipment, foundations, or roadways. All of the above earth pressures are unfactored and are, therefore, not inclusive of factors of safety.

Groundwater Influence on Structures/Construction

During our field investigation, groundwater was encountered at depths of approximately 20 feet below existing grade. Based on the anticipated depth of construction, groundwater is anticipated to impact the proposed construction, dewatering techniques should be implemented during the excavation and construction of the proposed structures. Also, very moist soils were encountered at the subject site and should be anticipated during construction.

Historic groundwater levels are reported at depths on the order of 10 feet below existing site grade. Therefore, dewatering and/or waterproofing will be required should structures or excavations approach or extend below this depth. If groundwater is encountered, our firm should be consulted prior to dewatering the site. Installation of a standpipe piezometer is suggested prior to construction should groundwater levels be a concern. The Contractor should refer to the soil boring logs in Appendix A for available information regarding groundwater levels at specific locations.

In addition to the groundwater level, if earthwork is performed during or soon after periods of precipitation, the subgrade soils may become saturated, pump, or not respond to densification techniques. Typical remedial measures include discing and aerating the soil during dry weather; mixing the soil with dryer materials; removing and replacing the soil with an approved fill material; or mixing the soil with an approved lime or cement product. Our firm should be consulted prior to implementing remedial measures to observe the unstable subgrade conditions and provide appropriate recommendations.

Site Preparation

General site clearing should include removal of vegetation; existing utilities; structures including foundations; basement walls and floors; existing stockpiled soil; trees and associated root systems; rubble; rubbish; and any loose and/or saturated materials. With the construction of a subterranean structure throughout the majority of the site and with the required excavation estimated to extend to a depth of approximately 30 feet below grade, we expect the remnants of any prior development, will be removed during the excavation of the site. The same is true for the root structures of the existing trees and any near-surface organic-laden soils. In the event that previously unidentified debris pits or underground utilities are encountered, those objects should be removed in their entirety. Any abandoned underground utilities that are exposed and found to extend into adjacent properties should be capped.

The site is presently occupied by an existing commercial plaza. Portions of the site are covered by concrete and asphaltic concrete pavement. Fill may be present at the site. Any fill soil encountered should be excavated and stockpiled so that the native soils can be properly prepared. The clayey soils encountered at the site will not be suitable for reuse as Non-Expansive Engineered Fill. However, these clayey soils will be suitable for reuse as General Engineered Fill, provided they are cleansed of excessive organics and debris, and are moisture-conditioned to a minimum of 2 percent above optimum moisture-content. Prior to fill placement Krazan & Associates, Inc. should inspect the bottom of the excavation to verify no additional removal will be required.

Existing structures are located within the site. Associated with these structures are buried structures such as utilities. Demolition activities should include proper removal of any buried structures. Any surface or buried structures, such as utilities or loosely backfilled excavations, encountered during construction should be properly removed and the resulting excavations backfilled. After demolition activities, it is recommended that these disturbed soils be removed and/or recompacted. Excavations, depressions, or soft and pliant areas extending below planned, finished subgrade levels should be cleaned to firm, undisturbed soil and backfilled with Engineered Fill. In general, any septic tanks, debris pits, cesspools, or similar structures should be entirely removed. Concrete footings should be removed to an equivalent depth of at least 3 feet below proposed footing elevations or as recommended by the Soils Engineer. Any other buried structures should be removed in accordance with the recommendations of the Soils Engineer. The resulting excavations should be backfilled with Engineered Fill.

Trees and shrubs are located within the project site. Tree and shrub removal operations should include roots greater than ½ inch in diameter. The resulting excavations should be backfilled with Engineered Fill compacted to a minimum of 90 percent of the maximum dry density based on ASTM Test Method D1557.

Following stripping, fill removal, and demolition activities, it is recommended that at a minimum, the upper 24 inches of exposed subgrade soils beneath the building pad areas be excavated, worked until uniform and free from large clods, moisture-conditioned to a minimum of 2 percent above optimum moisture-content, and recompacted to a minimum of 90 percent of maximum dry density based on ASTM Test Method D1557. In addition, remedial grading should be performed to a minimum of 24 inches below proposed foundation bearing grades. Within the pavement and exterior flatwork areas, the exposed subgrade should be excavated to a depth of 8 inches, worked until uniform and free from large clods and moisture-conditioned to a minimum of 2 percent above optimum moisture-content and recompacted to a minimum of 90 percent of the maximum dry density based on ASTM Test Method D1557. Prior to backfilling, the bottom of the excavation should be proof-rolled and observed by Krazan & Associates, Inc. to verify stability. This compaction effort should stabilize the upper soils and locate any unsuitable or pliant areas not found during our field investigation.

In areas where slab-on-grade construction will be utilized, it is recommended that the upper 24 inches of soil within proposed slab-on-grade and exterior flatwork areas consist of Non-Expansive Engineered Fill. The fill placement serves two functions: 1) it provides a uniform amount of soil which will more evenly distribute the soil pressures and 2) it reduces moisture-content fluctuation in the clayey material beneath the building area. The non-expansive fill material should be a well-graded silty sand or sandy silt soil. A clean sand or very sandy soil is not acceptable for this purpose. A sandy soil will allow the surface water to drain into the expansive clayey soil below, which may result in soil swelling. Imported Fill should be approved by the Soils Engineer prior to placement. The fill should be placed as specified as Engineered Fill.

The upper soils, during wet winter months, become very moist due to the absorptive characteristics of the soil. Earthwork operations performed during winter months may encounter very moist unstable soils, which may require removal to grade a stable building foundation. Project site winterization consisting of placement of aggregate base and protecting exposed soils during the construction phase should be performed.

Historic groundwater levels are reported at depths on the order of 10 feet below existing site grade. Therefore, dewatering and/or waterproofing will be required should structures or excavations approach or extend below this depth. If groundwater is encountered, our firm should be consulted prior to dewatering the site. Installation of a standpipe piezometer is suggested prior to construction should groundwater levels be a concern. The Contractor should refer to the soil boring logs in Appendix A for available information regarding groundwater levels at specific locations.

A representative of our firm should be present during all site clearing and grading operations to test and observe earthwork construction. This testing and observation is an integral part of our service, as acceptance of earthwork construction is dependent upon compaction and stability of the material. The Soils Engineer may reject any material that does not meet compaction and stability requirements. Further recommendations of this report are predicated upon the assumption that earthwork construction will conform to recommendations set forth in this section and the Engineered Fill section.

Engineered Fill

The organic-free, on-site, upper native soils are predominately sandy clay and clay. The underlying silty sand and sandy silt soils that do not contain clay will be suitable for reuse as Non-Expansive Engineered Fill provided they are cleansed of excessive organics and debris. The clayey soils will not be suitable for reuse as Non-Expansive Engineered Fill. The clayey soils should be at or above optimum moisture-content during mixing operations. These clayey soils will be suitable for reuse as General Engineered Fill, within flexible pavement areas, structural areas supported by structural slabs, and below 24 inches from finished grade in building areas, provided they are cleansed of excessive organics, debris, and moisture-conditioned to at least 2 percent above optimum moisture-content. It is recommended that additional testing be performed on the on-site soils and fill material to evaluate the physical and index properties prior to reuse as Engineered Fill.

The asphaltic concrete will not be suitable for reuse as Engineered Fill within the proposed building pad. However, fill intermixed with crushed asphaltic concrete may be used in paved areas provided they are cleansed of excessive organics, debris, and fragments larger than 4 inches in maximum dimension.

The preferred materials specified for Engineered Fill are suitable for most applications with the exception of exposure to erosion. Project site winterization and protection of exposed soils during the construction phase should be the sole responsibility of the Contractor, since he has complete control of the project site at that time.

Imported Non-Expansive Fill should consist of a well-graded, slightly cohesive, fine silty sand or sandy silt soil, with relatively impervious characteristics when compacted. This material should be approved by the Soils Engineer prior to use and should typically possess the following characteristics:

Percent Passing No. 200 Sieve	20 to 50
Plasticity Index	10 maximum
UBC Standard 29-2 Expansion Index	15 maximum

Fill soils should be placed in lifts approximately 6 inches thick, moisture-conditioned to a minimum of 2 percent above optimum moisture-context, and compacted to achieve at least 90 percent of the maximum density based on ASTM D1557. Additional lifts should not be placed if the previous lift did not meet the required dry density or if soil conditions are not stable.

Drainage and Landscaping

We understand that civil design practices typically include measures to address storm water quality issues. These often include the use of swales to allow for infiltration of the water into the site and to some extent bioremediation of the runoff water as it flows through the swale. Given the anticipated very low permeability of the clay soils at the site, relatively shallow groundwater, the limited exposed surface area, as well as the proposed subterranean structures, little to no infiltration will occur after wetting of the site. The storm water system should be designed for 100 percent run-off over the entire site.

The ground surface should slope away from building pad and pavement areas toward appropriate drop inlets or other surface drainage devices. In accordance with Section 1803 of the 2016 California Building Code, it is recommended that the ground surface adjacent to foundations be sloped a minimum of 5 percent for a minimum distance of 10 feet away from structures, or to an approved alternative means of drainage conveyance. Swales used for conveyance of drainage and located within 10 feet of foundations should be sloped a minimum of 2 percent. Impervious surfaces, such as pavement and exterior concrete flatwork, within 10 feet of building foundations should be sloped a minimum of 2 percent away from the structure. Drainage gradients should be maintained to carry all surface water to collection facilities and off-site. These grades should be maintained for the life of the project.

Slots or weep holes should be placed in drop inlets or other surface drainage devices in pavement areas to allow free drainage of adjoining base course materials. Cutoff walls should be installed at pavement edges adjacent to vehicular traffic areas. These walls should extend to a minimum depth of 12 inches below pavement subgrades to limit the amount of seepage water that can infiltrate the pavements. Where cutoff walls are undesirable subgrade drains can be constructed to transport excess water away from planters to drainage interceptors. If cutoff walls can be successfully used at the site, construction of subgrade drains is considered unnecessary.

Subsurface Drainage

With the presence of shallow groundwater and the proposed depth of the structure, implementation of a subsurface drainage system may not be practical. The structure should be designed with full waterproofing and to resist hydrostatic pressures. Retaining walls extending below the groundwater level should be fully waterproofed and designed to resist both soil and hydrostatic pressures, as well as any applicable surcharge loads. If the final depth of the proposed structure approaches the depth to groundwater, floor slabs should also be waterproofed and be able to resist hydrostatic pressures. Waterproofing of the slab-on-grade (mat-slab foundation as discussed below) should consist of positive side waterproofing (located below the slab). This type of water proofing typically requires the placement of a waste or rat slab (nominal 2-inch concrete section) over the base of the excavation. The waterproofing membrane is then installed followed by a second waste or rat slab to protect the membrane. The membrane is wrapped up the sides of the foundation and is then lapped by the waterproofing membrane installed at the basement walls.

Temporary Excavation Stability

All excavations should comply with the current requirements of Occupational Safety and Health Administration (OSHA). All cuts greater than 4 feet in depth should be sloped or shored. Temporary excavations should be sloped at 1½:1 (horizontal to vertical) or flatter, up to a maximum depth of 8 feet, and at 2:1 (horizontal to vertical) to a maximum depth of 12 feet. Heavy construction equipment, building materials, excavated soil, and vehicular traffic should not be allowed within five feet of the top (edge) of the excavation. Where sloped excavations are not feasible due to site constraints, the excavations will require shoring. The design of the shoring system is normally the responsibility of the contractor or shoring designer, and therefore, is outside the scope of this report. The design of the temporary shoring should take into account lateral pressures exerted by the adjacent soil, and, where anticipated, surcharge loads due to adjacent buildings and any construction equipment or traffic expected to operate alongside the excavation.

The excavation/shoring recommendations provided herein are based on soil characteristics derived from our test borings within the area. Variations in soil conditions will likely be encountered during the excavations. Krazan & Associates, Inc. should be afforded the opportunity to provide field review to evaluate the actual conditions and account for field condition variations, not otherwise anticipated in the preparation of this recommendation.

Utility Trench Backfill

Utility trenches should be excavated according to accepted engineering practice following OSHA (Occupational Safety and Health Administration) standards by a Contractor experienced in such work. The responsibility for the safety of open trenches should be borne by the Contractor. Traffic and vibration adjacent to trench walls should be minimized; cyclic wetting and drying of excavation side slopes should be avoided. Depending upon the location and depth of some utility trenches, groundwater flow into open excavations could be experienced; especially during or following periods of precipitation.

Utility trench backfill placed in or adjacent to buildings and exterior slabs should be compacted to at least 90 percent of the maximum dry density based on ASTM Test Method D1557. The utility trench backfill placed in pavement areas should be compacted to at least 90 percent of the maximum dry density based on ASTM Test Method D1557. Pipe bedding should be in accordance with pipe manufacturer's recommendations.

The Contractor is responsible for removing all water-sensitive soils from the trench regardless of the backfill location and compaction requirements. The Contractor should use appropriate equipment and methods to avoid damage to the utilities and/or structures during fill placement and compaction.

Foundations - Conventional

After completion of the recommended site preparation and over-excavation, the site should be suitable for shallow footing support within a depth of up to 10 feet below site grades. The proposed structures may be supported on a shallow foundation system bearing on a minimum of 2 feet of Engineered Fill. Spread and continuous footings can be designed for the following maximum allowable soil bearing pressures:

Load	Allowable Loading
Dead Load Only	1,875 psf
Dead-Plus-Live Load	2,500 psf
Total Load, Including Wind or Seismic Loads	3,320 psf

The footings should have a minimum embedment depth of 24 inches below pad subgrade (soil grade) or adjacent exterior grade, whichever is lower. Footings should have a minimum width of 18 inches, regardless of load. Shallow foundation systems should be designed to tolerate the anticipated static and seismic settlement. The actual foundation design should be performed by the project structural engineer.

The footings should have a minimum embedment depth of 24 inches below pad subgrade (soil grade) or adjacent exterior grade, whichever is lower. Footings should have a minimum width of 18 inches, regardless of load. The actual design of foundations should be performed by the project structural engineer. Shallow foundation systems should be designed to tolerate the anticipated static and seismic settlement.

Foundations supported at deeper elevations may require the use of a structural mat foundation. Deeper excavations will likely require dewatering and soil stabilization in order to address saturated soil conditions and provide for a stable foundation bearing grade.

The footing excavations should not be allowed to dry out any time prior to pouring concrete. It is recommended that footings be reinforced by at least one No. 4 reinforcing bar in both top and bottom. The actual design of foundations should be performed by the project structural engineer.

Resistance to lateral footing displacement can be computed using an allowable friction factor of 0.25 acting between the base of foundations and the supporting subgrade. Lateral resistance for footings can alternatively be developed using an allowable equivalent fluid passive pressure of 200 pounds per cubic foot acting against the appropriate vertical footing faces. The frictional and passive resistance of the soil may be combined without reduction in determining the total lateral resistance. An 1/3 increase in the value above may be used for short duration, wind, or seismic loads. All of the above earth pressures are unfactored and are, therefore, not inclusive of factors of safety.

The total static movement is not expected to exceed ¾ inch. Differential static movement should be less than ½ inch. Most of the static settlement is expected to occur during construction, as the loads are applied. However, additional post-construction movement may occur if the foundation soils are flooded or saturated.

Foundations –Structural Mat Slab

The potential for structural damage as a result of differential settlement due to the potential effects of soil consolidation associated with applied structural loads can be reduced by supporting the building on a very stiff structural mat-slab foundation. The foundation should be designed to distribute the building loads uniformly onto the supporting subgrade. By designing a relatively stiff mat, the settlement of the structure will be relatively uniform. The foundation should be designed to be sufficiently rigid to prevent the introduction of excess stresses in the superstructure above the foundation.

The use of a sufficiently stiff to rigid structural mat-slab foundation will mitigate abrupt differential settlement but will not negate building settlement (total settlement). Where both total and differential settlements of the structure are to be fully mitigated a deep foundation system or extensive ground improvement would be required. Deep foundations might include driven piles, geopiers, or auger cast piles. Ground improvement could potentially include compaction grouting. Should a deep foundation or ground improvement option be desired, we should be contacted to discuss the options and to assist you with the preparation of additional geotechnical recommendations for use in site development. In the event that ground improvement or deep foundations are desired, additional subsurface exploration and laboratory testing of soil samples may be required.

Support of structures with a mat-slab foundation is a method used to aid in controlling differential settlement of structures over weak soils. The foundation distributes high point loads and line loads over a much broader area resulting in significantly reduced stresses and a more uniform loading condition over the building area. This reduces the differential settlement of walls and columns that would be expected when supported by dissimilarly loaded footings and footings of differing sizes, and can result in less total settlement of the superstructure when supported by the structural slab. The slab also provides increase confinement for sands below the surface reducing the potential for abrupt loss of support of foundation elements due to sand boils where shallow liquefiable sands are present.

The slab foundation should be designed to resist both bending and punching shear associated with the structural loads and design live loads. With the potential for arching or bending of the slab foundation to occur as a result of differential settlement, we recommend that the slab be designed to span over localized areas of settlement and to act as a cantilevered beam to support the perimeter of the building should localized settlement occur in areas of the perimeter.

For preliminary purposes, an allowable bearing pressure of 1,000 pounds per square foot may be used for design of the slab. For preliminary modeling purposes a vertical modulus of subgrade reaction (Kv1), also referred to as a soil spring, of 30 pounds per square inch per inch may be used for long term conditions. An increased modulus of 40 pounds per square inch per inch may be used for short term loading to evaluate punching shear at columns and walls. The slab design should ultimately limit slab bending or arching in the lightly loaded mid-slab areas between load bearing columns and walls. Based on the preliminary nature of the project design and a lack of formal design documents, these values should be considered preliminary and should be reevaluated during final design. The values should be reevaluated in order to determine soil support values appropriate for the actual design conditions.

Floor Slabs and Exterior Flatwork

To reduce post-construction soil movement beneath floor slabs and exterior flatwork, it is recommended that mitigation measures be performed. For conventional slab-on-grade, it is recommended that the upper 24 inches of soil consist of Non-Expansive Engineered Fill.

Concrete slab-on-grade floors should be underlain by a water vapor retarder. The water vapor retarder should be installed in accordance with ASTM Specification E 1643-98. According to ASTM Guidelines, the water vapor retarder should consist of vapor retarder sheeting underlain by a minimum of 3 inches of compacted, clean, gravel of ¾-inch maximum size. To aid in concrete curing an optional 2 to 4 inches of granular fill may be placed on top of the vapor retarder. The granular fill should consist of damp clean sand with at least 10 to 30 percent of the sand passing the 100 sieve. The sand should be free of clay, silt, or organic material. Rock dust which is manufactured sand from rock crushing operations is typically suitable for the granular fill. This granular fill material should be compacted.

It is recommended that the concrete slabs be reinforced with at least No. 3 reinforcing bars, placed at 18 inches on center in each direction within the slabs middle third, to reduce crack separation and possible vertical offset at the cracks. Thicker floor slabs with increased concrete strength and reinforcement should be designed wherever heavy concentrated loads, heavy equipment, or machinery is anticipated.

The exterior floors should be poured separately in order to act independently of the walls and foundation system. Exterior finish grades should be sloped a minimum of 2 percent away from all interior slab areas to preclude ponding of water adjacent to the structures. All fills required to bring the building pads to grade should be Engineered Fills.

Moisture within the structure may be derived from water vapors, which were transformed from the moisture within the soils. This moisture vapor can travel through the vapor membrane and penetrate the slab-on-grade. This moisture vapor penetration can affect floor coverings and produce mold and mildew in the structure. To minimize moisture vapor intrusion, it is recommended that a vapor retarder be installed in accordance with ASTM guidelines. It is recommended that the utility trenches within the structure be compacted, as specified in our report, to minimize the transmission of moisture through the utility trench backfill. Special attention to the immediate drainage and irrigation around the building is recommended. Positive drainage should be established away from the structure and should be maintained throughout the life of the structure. Ponding of water should not be allowed adjacent to the structure. Over-irrigation within landscaped areas adjacent to the structure should not be performed. In addition, ventilation of the structure (i.e. ventilation fans) is recommended to reduce the accumulation of interior moisture.

Shoring and Excavation Stability

The locations of the planned subterranean structure relative to existing developments, including houses, apartment buildings, commercial structures, and roadways, that surround the subject site will need to be evaluated for possible impacts of the excavations on these structures. Due to the close proximity of several of these structures to the property lines, the relatively small setback of the structure from the property lines, as well as the expected depth of the excavation, sloping back of the excavation walls is not feasible and shoring will be required.

The design of the shoring system is normally the responsibility of the contractor or shoring designer, and therefore, is outside the scope of this report. However, the logs of borings presented with this report may be used for factual data such as soil types encountered at the location of each particular boring and at the indicated depths for a preliminary assessment of shoring requirements. Interpolation between the exploratory borings is at the user's own risk. Design work for shoring systems should be performed by an engineer with expertise in shoring systems. The design of the temporary shoring should take into account lateral pressures exerted by the adjacent soil, and, where present, surcharge loads due to adjacent embankments, buildings and any construction equipment or traffic expected to operate alongside the excavation.

Shoring on the sides of the excavation can be provided by means of a cantilever or restrained soldier beam or soldier pile and lagging wall. Lateral load resistance can be mobilized through the use of passive pressures on members that extend below the bottom of the excavation or interior bracing. Shoring must be designed with sufficient rigidity or must be supported by struts (bracing) to prevent deflection where in close proximity to structures. If the shoring is allowed to deflect, as occurs where designed as a cantilevered wall, settlement of the area behind the shoring will occur.

Whenever excavation is made adjacent to existing streets, utilities and structures, there is the potential for movement. The existing structures should be inspected and documented to preclude claims for damage or settlement that are not associated with the construction of the planned development. A monitoring program should be established so excessive movement is detected early. The monitoring program should include optical surveying of the shoring and adjacent streets and buildings to detect any horizontal or vertical movement.

Lateral Earth Pressures and Retaining Walls

Walls retaining horizontal backfill and capable of deflecting a minimum of 0.1 percent of its height at the top may be designed using an equivalent fluid active pressure of 47 pounds per square foot per foot of depth. Walls that are incapable of this deflection or walls that are fully constrained against deflection may be designed for an equivalent fluid at-rest pressure of 68 pounds per square foot per foot of depth. Expansive soils should not be used for backfill against walls. The wedge of non-expansive backfill material should extend from the bottom of each retaining wall outward and upward at a slope of 2:1 (horizontal to vertical) or flatter. The stated lateral earth pressures do not include the effects of hydrostatic water pressures generated by infiltrating surface water that may accumulate behind the retaining walls; or loads imposed by construction equipment, foundations, or roadways. All of the above earth pressures are unfactored and are, therefore, not inclusive of factors of safety.

To simulate the effect of earthquake loading on retaining walls, the walls may be evaluated based on an active lateral soil pressure calculated using an equivalent fluid weight of 58 pounds per cubic foot plus a horizontal seismic surcharge line force of 35H pounds per square foot of wall. The resultant of the lateral soil pressure should be applied at H/3 above the wall base and the resultant of the seismic surcharge force should be applied at a height of 0.6H above the wall base. For the purpose of this report, "H" is defined as the vertical height from the base of the wall to the ground surface above.

During grading and backfilling operations adjacent to any walls, heavy equipment should not be allowed to operate within a lateral distance of 5 feet from the wall, or within a lateral distance equal to the wall height, whichever is greater, to avoid developing excessive lateral pressures. Within this zone, only hand operated equipment ("whackers," vibratory plates, or pneumatic compactors) should be used to compact the backfill soils.

Retaining and/or below grade walls should be drained with either perforated pipe encased in free-draining gravel or a prefabricated drainage system. The gravel zone should have a minimum width of 12 inches wide and should extend upward to within 12 inches of the top of the wall. The upper 12 inches of backfill should consist of native soils, concrete, asphaltic concrete or other suitable backfill to minimize surface drainage into the wall drain system. The aggregate should conform to Class II permeable materials graded in accordance with the CalTrans Standard Specifications (May 2006). Prefabricated drainage systems, such as Miradrain®, Enkadrain®, or an equivalent substitute, are acceptable alternatives in lieu of gravel provided they are installed in accordance with the manufacturer's recommendations. If a prefabricated drainage system is proposed, our firm should review the system for final acceptance prior to installation.

Drainage pipes should be placed with perforations down and should discharge in a non-erosive manner away from foundations and other improvements. The pipes should be placed no higher than 6 inches above the heel of the wall in the center line of the drainage blanket and should have a minimum diameter of 4 inches. Collector pipes may be either slotted or perforated. Slots should be no wider than 1/8 inch in diameter, while perforations should be no more than ¼ inch in diameter. If retaining walls are less than 6 feet in height, the perforated pipe may be omitted in lieu of weep holes on 4 feet maximum spacing. The weep holes should consist of 4-inch diameter holes (concrete walls) or unmortared head joints (masonry walls) and not be higher than 18 inches above the lowest adjacent grade. Two 8-inch square overlapping patches of geotextile fabric (conforming to the CalTrans Standard Specifications for "edge drains") should be affixed to the rear wall opening of each weep hole to retard soil piping.

It is recommended that any uncertified fill material encountered within pavement areas, be removed and/or recompacted. The fill material should be moisture-conditioned to near optimum moisture-content and recompacted to a minimum of 90 percent of the maximum dry density based on ASTM Test Method D1557. As an alternative, the Owner may elect not to recompact the existing fill within paved areas. However, the Owner should be aware that the paved areas may settle which may require annual maintenance. At a minimum, it is recommended that the upper 12 inches of subgrade soil be moisture-conditioned as necessary and recompacted to a minimum of 90 percent of the maximum dry density based on ASTM Test Method D1557.

R-Value Test Results and Pavement Design

One bulk soil sample was obtained from the project site for R-Value testing at the location shown on the attached site plan. The sample was tested in accordance with the State of California Materials Manual Test Designation 301. Results of the test are as follows:

Sample	Depth	Description	R-Value at Equilibrium
1	0-24"	Sandy Clay (CL)	35

The test results are moderate and indicate good subgrade support characteristics under dynamic traffic loads. The following table shows the recommended pavement sections for various traffic indices.

Traffic Index	Asphaltic Concrete	Class II Aggregate Base*	Compacted Subgrade**
4.0	2.0"	5.0"	12.0"
4.5	2.5"	5.0"	12.0"
5.0	2.5"	6.0"	12.0"
5.5	3.0"	6.0"	12.0"
6.0	3.0"	6.0"	12.0"
6.5	3.5"	7.0"	12.0"
7.0	4.0"	8.0"	12.0"
7.5	4.0"	8.0"	12.0"

**** 95% compaction based on ASTM Test Method D1557 or CAL 216***
***** 95% compaction based on ASTM Test Method D1557 or CAL 216***

If traffic indices are not available, an estimated (typical value) index of 4.5 may be used for light automobile traffic and an index of 7.0 may be used for light truck traffic. Following grading operations, it is recommended additional R-Value testing be performed to verify the design R-Value.

Seismic Parameters – 2016 California Building Code

The Site Class per Table 1613.5.2, of the 2016 California Building Code (2016 CBC) is based upon the site soil conditions. It is our opinion that Site Class D is most consistent with the subject site soil conditions. For seismic design of the structures based on the seismic provisions of the 2016 CBC, we recommend the following parameters:

Seismic Item	Value	CBC Reference
Site Class	D	Table 1613.5.2
Site Coefficient Fa	1.000	Table 1613.5.3 (1)
Ss	1.947	Figure 1613.5 (3)
SMS	1.947	Section 1613.5.3
SDS	1.298	Section 1613.5.4
Site Coefficient Fv	1.500	Table 1613.5.3 (2)
S1	0.814	Figure 1613.5 (4)
SM1	1.221	Section 1613.5.3
SD1	0.814	Section 1613.5.4

Infiltration Testing

The shallow soil conditions present at the subject site were evaluated by drilling shallow borings in the vicinity of the infiltration test. The borings drilled at the site indicated the subsurface soil conditions consisted of sandy clays.

Infiltration rates were determined using the results of open borehole infiltration testing performed at the subject site. Infiltration testing performed on the near surface sandy clay soil indicate infiltration rates of approximately 0.68 and 0.91 inch per hour. Based on the very low infiltration rates as well as relatively shallow historic groundwater elevations, the subsurface conditions encountered at the site may not be conducive to infiltration. Detailed results of the percolation tests and infiltration rates are attached in tabular format.

The soil percolation rates are based on tests conducted with clean water. The infiltration rates may vary with time as a result of soil clogging from water impurities. A factor of safety should be incorporated into the design of the percolation system to compensate for these factors as determined appropriate by the designer. In addition, periodic maintenance consisting of clearing the bottom of the system of clogged soils should be expected.

It is recommended that the location of the infiltration systems not be closer than ten feet (10') as measured laterally from the edge of the adjacent property line, ten feet (10') from the outside edge of any foundation and five (5') from the edge of any right-of way to the outside edges of the infiltration system.

If the infiltration location is within ten feet (10') of the proposed foundation, it is recommended that this infiltration system should be impervious from the finished ground surface to a depth that will achieve a diagonal distance of a minimum of ten feet (10') below the bottom of the closest footing in the project.

Soil Cement Reactivity

Excessive sulfate in either the soil or native water may result in an adverse reaction between the cement in concrete (or stucco) and the soil. HUD/FHA and CBC have developed criteria for evaluation of sulfate levels and how they relate to cement reactivity with soil and/or water.

A soil sample was obtained from the site and tested in accordance with State of California Materials Manual Test Designation 417. The sulfate concentration detected from the soil sample indicated a moderate sulfate exposure value as established by HUD/FHA and CBC. Therefore, it is recommended that concrete in contact with soil utilize Type II Cement and have a minimum compressive strength of 4,000 psi.

Electrical resistivity testing of the soil indicates that the onsite soils may have a moderate potential for metal loss from electrochemical corrosion process. A qualified corrosion engineer should be consulted regarding the corrosion effects of the onsite soils on underground metal utilities.

Compacted Material Acceptance

Compaction specifications are not the only criteria for acceptance of the site grading or other such activities. However, the compaction test is the most universally recognized test method for assessing the performance of the Grading Contractor. The numerical test results from the compaction test cannot solely be used to predict the engineering performance of the compacted material. Therefore, the acceptance of compacted materials will also be dependent on the stability of that material. The Soils Engineer has the option of rejecting any compacted material regardless of the degree of compaction if that material is considered to be unstable or if future instability is suspected. A specific example of rejection of fill material passing the required percent compaction is a fill which has been compacted with an in-situ moisture-content significantly less than optimum moisture. This type of dry fill (brittle fill) is susceptible to future settlement if it becomes saturated or flooded.

Testing and Inspection

A representative of Krazan & Associates, Inc. should be present at the site during the earthwork activities to confirm that actual subsurface conditions are consistent with the exploratory fieldwork. This activity is an integral part of our service, as acceptance of earthwork construction is dependent upon compaction testing and stability of the material. This representative can also verify that the intent of these recommendations is incorporated into the project design and construction. Krazan & Associates, Inc. will not be responsible for grades or staking, since this is the responsibility of the Prime Contractor.

LIMITATIONS

Soils Engineering is one of the newest divisions of Civil Engineering. This branch of Civil Engineering is constantly improving as new technologies and understanding of earth sciences advance. Although your site was analyzed using the most appropriate and most current techniques and methods, undoubtedly there will be substantial future improvements in this branch of engineering. In addition to advancements in the field of Soils Engineering, physical changes in the site, either due to excavation or fill placement, new agency regulations, or possible changes in the proposed structure after the soils report is completed may require the soils report to be professionally reviewed. In light of this, the owner should be aware that there is a practical limit to the usefulness of this report without critical review. Although the time limit for this review is strictly arbitrary, it is suggested that 2 years be considered a reasonable time for the usefulness of this report.

Foundation and earthwork construction is characterized by the presence of a calculated risk that soil and groundwater conditions have been fully revealed by the original foundation investigation. This risk is derived from the practical necessity of basing interpretations and design conclusions on limited sampling of the earth. The recommendations made in this report are based on the assumption that soil conditions do not vary significantly from those disclosed during our field investigation. If any variations or undesirable conditions are encountered during construction, the Soils Engineer should be notified so that supplemental recommendations may be made.

The conclusions of this report are based on the information provided regarding the proposed construction. If the proposed construction is relocated or redesigned, the conclusions in this report may not be valid. The Soils Engineer should be notified of any changes so the recommendations may be reviewed and re-evaluated.

This report is a Geotechnical Engineering Investigation with the purpose of evaluating the soil conditions in terms of foundation design. The scope of our services did not include any Environmental Site Assessment for the presence or absence of hazardous and/or toxic materials in the soil, groundwater, or atmosphere; or the presence of wetlands. Any statements, or absence of statements, in this report or on any boring log regarding odors, unusual or suspicious items, or conditions observed, are strictly for descriptive purposes and are not intended to convey engineering judgment regarding potentially hazardous and/or toxic assessment.

The geotechnical engineering information presented herein is based upon professional interpretation utilizing standard engineering practices and a degree of conservatism deemed proper for this project. It is not warranted that such information and interpretation cannot be superseded by future geotechnical engineering developments. We emphasize that this report is valid for the project outlined above and should not be used for any other sites.

If you have any questions, or if we may be of further assistance, please do not hesitate to contact our office at (951) 273-1011.

Respectfully submitted,
KRAZAN & ASSOCIATES, INC.

James Kellogg
Managing Engineer
RGE No. 2902/RCE No. 65092

Jorge A. Pelayo, EIT
Staff Engineer

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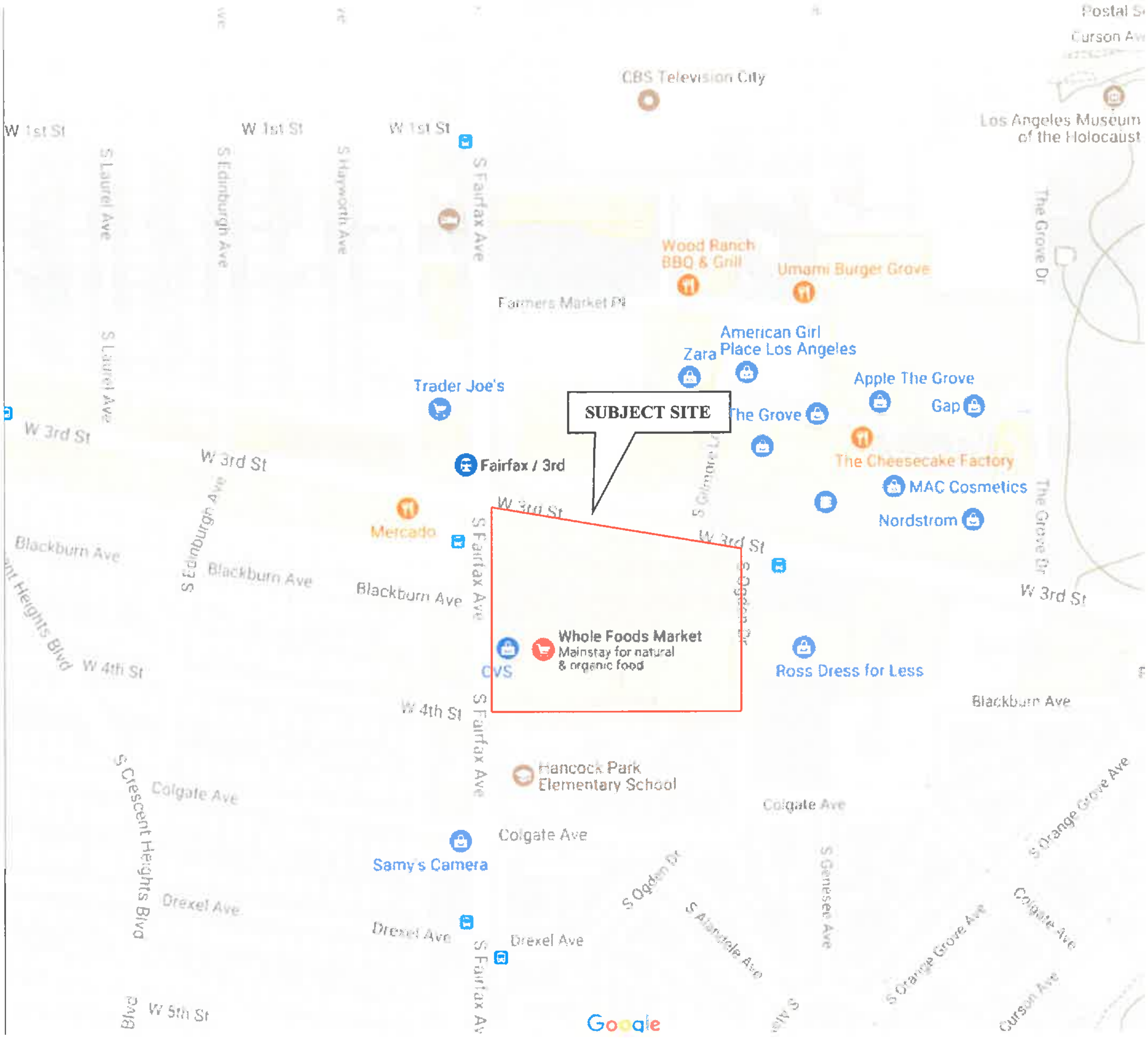
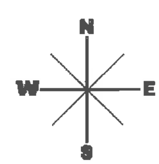
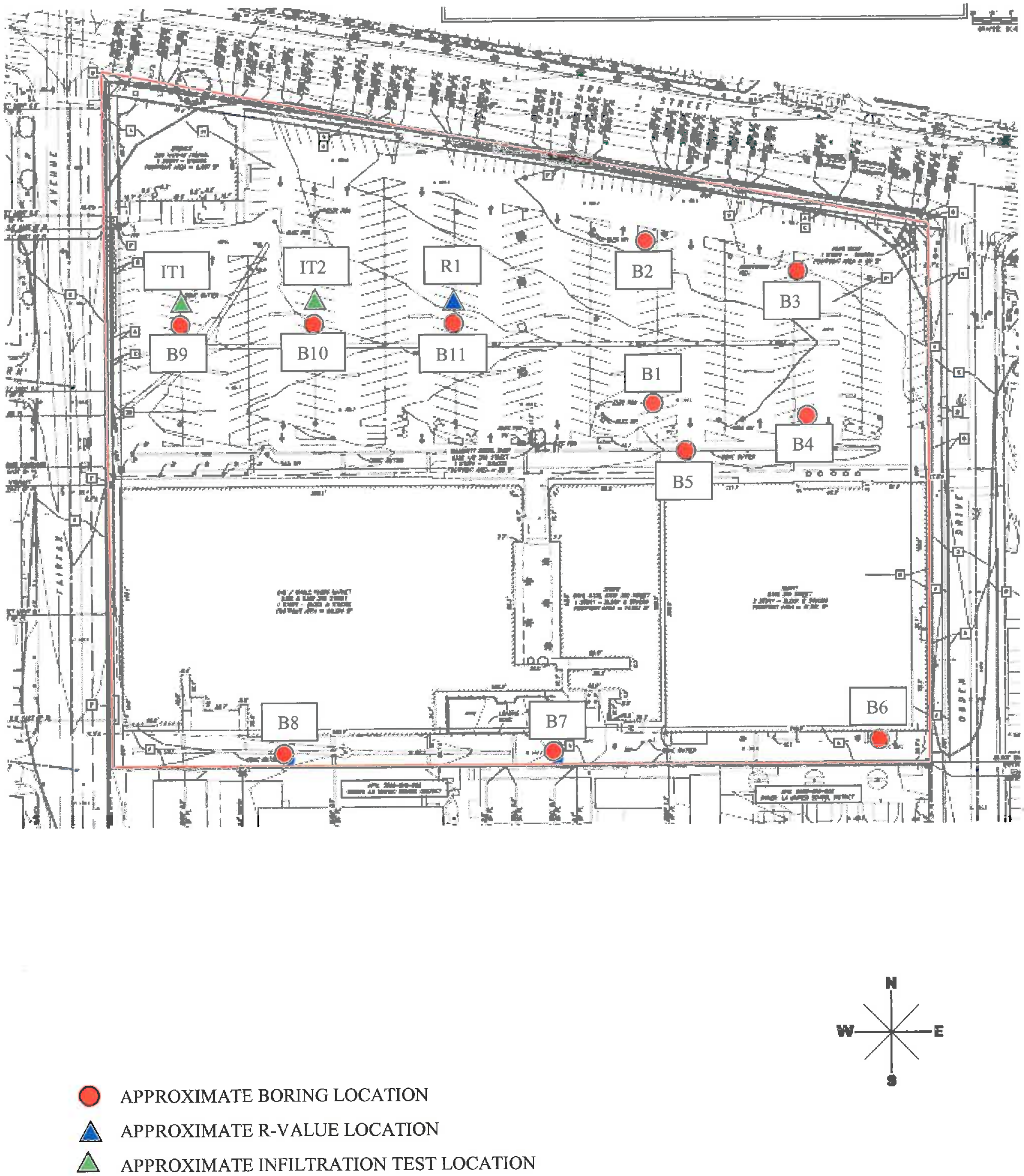
Figures

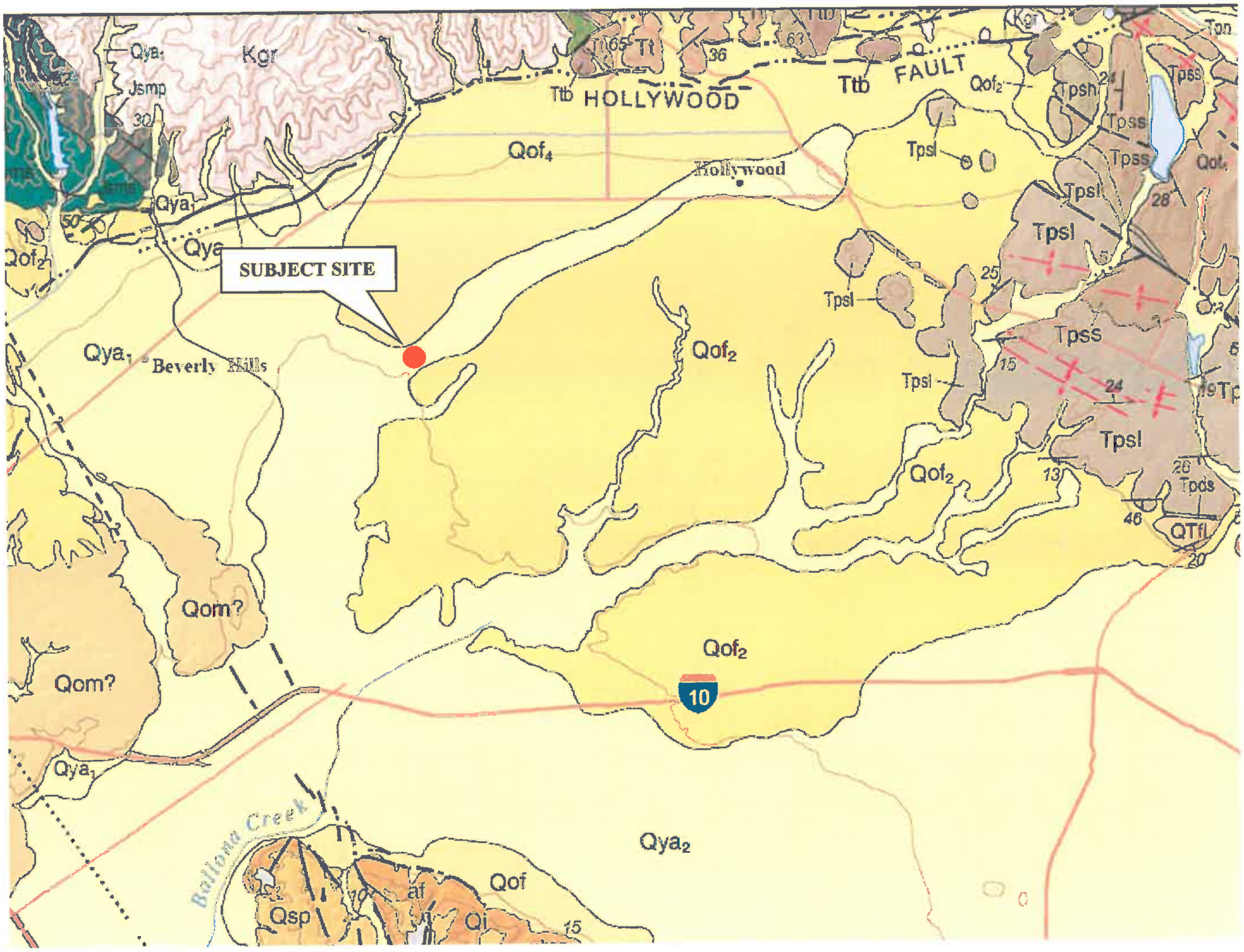




VICINITY MAP	**Scale:** NTS	**Date:** June 2017
PROPOSED MIXED USE DEVELOPMENT SEC 3rd & FAIRFAX LOS ANGELES, CALIFORNIA	**Drawn by:** SK	**Approved by:** JK
		Figure No. 1



SITE MAP	Scale: NTS	Date: June 2017
PROPOSED MIXED USE DEVELOPMENT SEC 3rd & FAIRFAX LOS ANGELES, CALIFORNIA	Drawn by: SK	Approved by: JK
		Figure No. 2

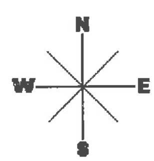


Old alluvium

Qoa	Undivided
Qoa$_3$	Unit 3
Qoa$_2$	Unit 2
Qoa$_1$	Unit 1

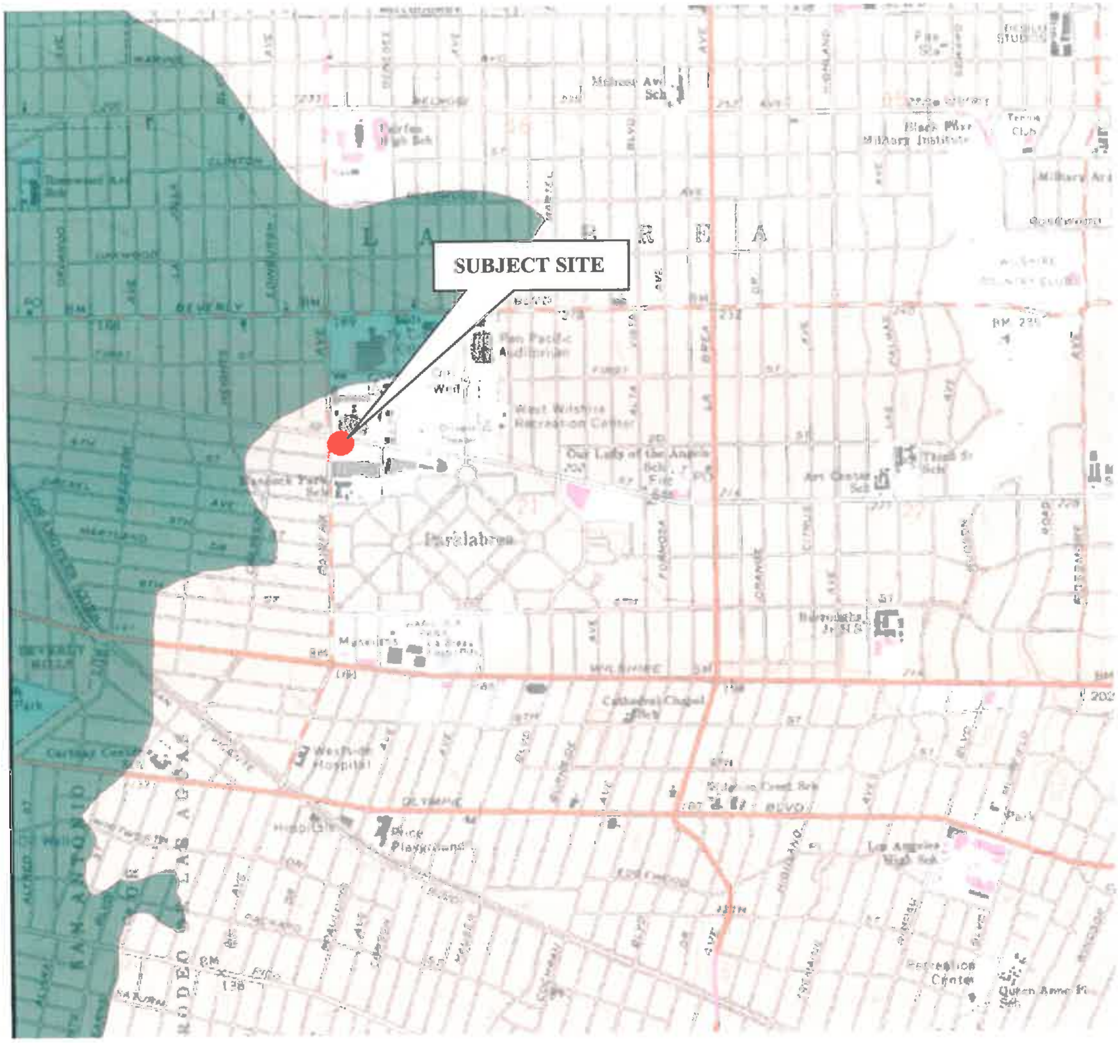
Old alluvial fan deposits

Qof	Undivided
Qof$_4$	Unit 4
Qof$_3$	Unit 3
Qof$_2$	Unit 2
Qof$_1$	Unit 1



Source: USGS Geologic Map of the Los Angeles 30' X 60' Quadrangle

GEOLOGIC MAP	**Scale:** NTS	**Date:** June 2017
PROPOSED MIXED USE DEVELOPMENT SEC 3rd & FAIRFAX LOS ANGELES, CALIFORNIA	**Drawn by:** SK	**Approved by:** JK
		Figure No. 3



MAP EXPLANATION

ALQUIST-PRIOLO EARTHQUAKE FAULT ZONES

Earthquake Fault Zones
Zone boundaries are delineated by straight-line segments; the boundaries define the zone encompassing active faults that constitute a potential hazard to structures from surface faulting or fault creep such that avoidance as described in Public Resources Code Section 2621.5(a) would be required.

Active Fault Traces
Faults considered to have been active during Holocene time and to have potential for surface rupture: Solid Line in Black or Red where Accurately Located; Long Dash in Black or Solid Line in Purple where Approximately Located; Short Dash in Black or Solid Line in Orange where Inferred; Dotted Line in Black or Solid Line in Rose where Concealed; Query (?) indicates additional uncertainty.

1906 C ? ?

SEISMIC HAZARD ZONES

Liquefaction Zones
Areas where historical occurrence of liquefaction, or local geological, geotechnical and ground water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Earthquake-Induced Landslide Zones
Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Source: State of California Seismic Hazards Map, Hollywood Quadrangle

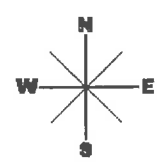
N W E S

LIQUEFACTION HAZARD ZONE MAP	**Scale:** NTS	**Date:** June 2017	
PROPOSED MIXED USE DEVELOPMENT SEC 3rd & FAIRFAX LOS ANGELES, CALIFORNIA	**Drawn by:** SK	**Approved by:** JK	
		Figure No. 4	

Legend:

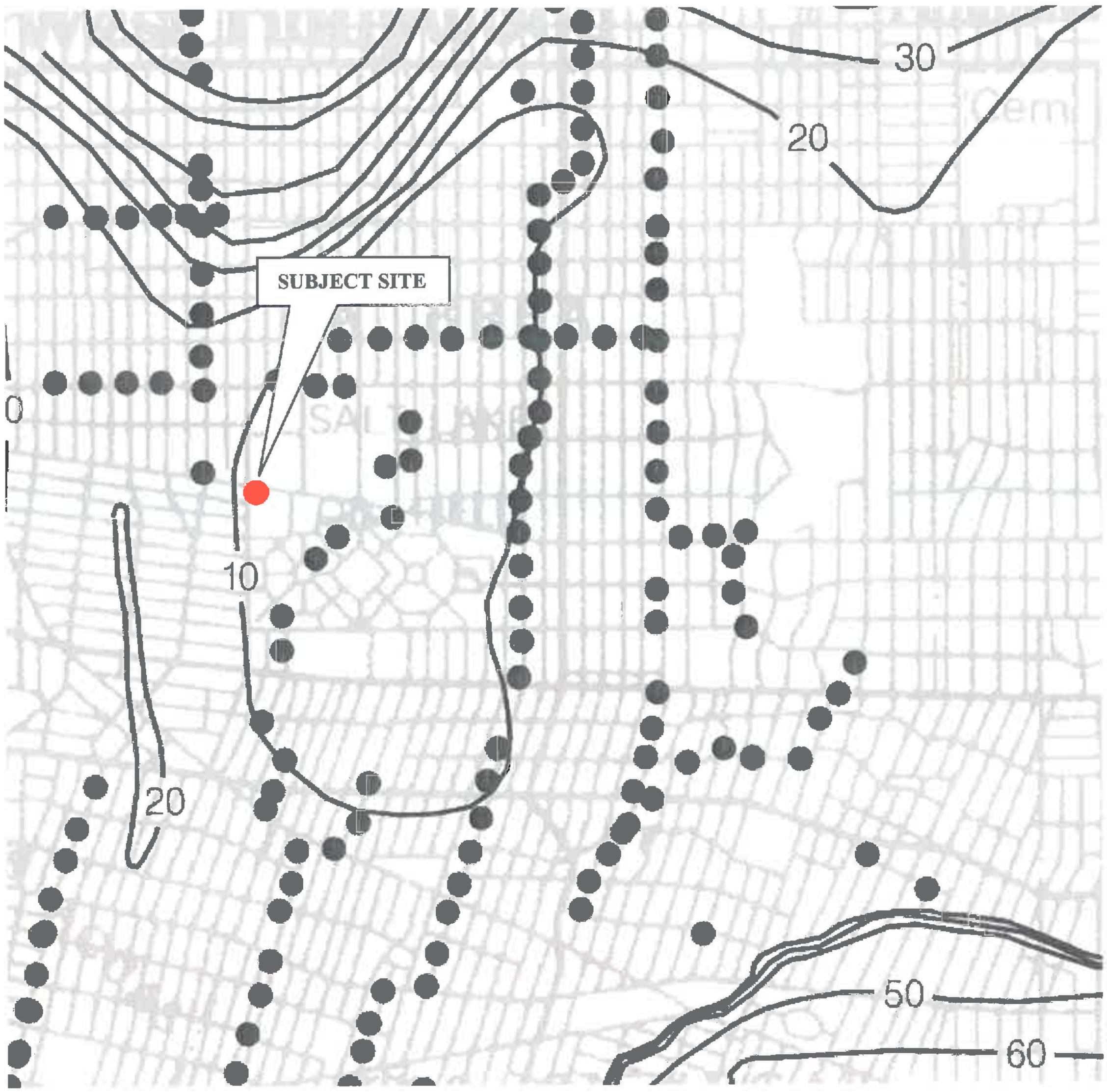
Well Types:

- New
- Active Producer
- Active Injector
- Dry Hole
- Plugged
- Geothermal
- Notice & Permit
- Enhanced Oil Recovery



Source: Map retrieved on 07/03/17 at Division of Oil, Gas & Geothermal Resources Well Finder

OIL WELL MAP	**Scale:** NTS	**Date:** June 2017	
PROPOSED MIXED USE DEVELOPMENT SEC 3rd & FAIRFAX LOS ANGELES, CALIFORNIA	**Drawn by:** SK	**Approved by:** JK	
		Figure No. 5	

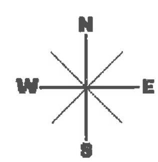


EXPLANATION

20 — Contour interval depth (in feet) to historic high groundwater.

10 — Depth (in feet) to historic high groundwater within defined area.

● Borehole Location



Source: State of California Seismic Hazards Map, Hollywood Quadrangle

HISTORICAL HIGH GROUNDWATER	**Scale:** NTS	**Date:** June 2017	
PROPOSED MIXED USE DEVELOPMENT SEC 3rd & FAIRFAX LOS ANGELES, CALIFORNIA	**Drawn by:** SK	**Approved by:** JK	
		Figure No. 6	

Appendix A

Log of Borings
&
Laboratory Testing

APPENDIX A

FIELD AND LABORATORY INVESTIGATIONS

Field Investigation

The field investigation consisted of a surface reconnaissance and a subsurface exploratory program. Eleven 6½-inch diameter exploratory borings were advanced. The boring locations are shown on the attached site plan.

The soils encountered were logged in the field during the exploration and with supplementary laboratory test data are described in accordance with the Unified Soil Classification System.

Modified standard penetration tests and standard penetration tests were performed at selected depths. This test represents the resistance to driving a 2½-inch and 1½-inch diameter split barrel sampler, respectively. The driving energy was provided by a hammer weighing 140 pounds falling 30 inches. Relatively undisturbed soil samples were obtained while performing this test. Bag samples of the disturbed soil were obtained from the auger cuttings. The modified standard penetration tests are identified in the sample type on the boring logs with a full shaded in block. The standard penetration tests are identified in the sample type on the boring logs with one-half of the block shaded. All samples were returned to our Fresno laboratory for evaluation.

Laboratory Investigation

The laboratory investigation was programmed to determine the physical and mechanical properties of the foundation soil underlying the site. Test results were used as criteria for determining the engineering suitability of the surface and subsurface materials encountered.

In-situ moisture content, dry density, consolidation, direct shear, and sieve analysis tests were completed for the undisturbed samples representative of the subsurface material. Expansion index and R-value tests were completed for select bag samples obtained from the auger cuttings. These tests, supplemented by visual observation, comprised the basis for our evaluation of the site material.

The logs of the exploratory borings and laboratory determinations are presented in this Appendix.

UNIFIED SOIL CLASSIFICATION SYSTEM

UNIFIED SOIL CLASSIFICATION AND SYMBOL CHART

COARSE-GRAINED SOILS (more than 50% of material is larger than No. 200 sieve size.)		
	Clean Gravels (Less than 5% fines)	
GRAVELS More than 50% of coarse fraction larger than No. 4 sieve size	GW	Well-graded gravels, gravel-sand mixtures, little or no fines
	GP	Poorly-graded gravels, gravel-sand mixtures, little or no fines
	Gravels with fines (More than 12% fines)	
	GM	Silty gravels, gravel-sand-silt mixtures
	GC	Clayey gravels, gravel-sand-clay mixtures
	Clean Sands (Less than 5% fines)	
SANDS 50% or more of coarse fraction smaller than No. 4 sieve size	SW	Well-graded sands, gravelly sands, little or no fines
	SP	Poorly graded sands, gravelly sands, little or no fines
	Sands with fines (More than 12% fines)	
	SM	Silty sands, sand-silt mixtures
	SC	Clayey sands, sand-clay mixtures
FINE-GRAINED SOILS (50% or more 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 of 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 50% or greater	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

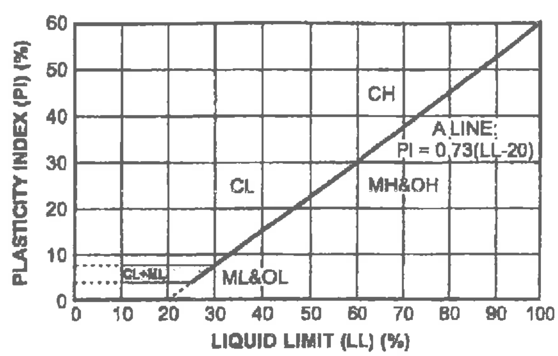
CONSISTENCY CLASSIFICATION

Description	Blows per Foot
Granular Soils	
Very Loose	< 5
Loose	5 – 15
Medium Dense	16 – 40
Dense	41 – 65
Very Dense	> 65
Cohesive Soils	
Very Soft	< 3
Soft	3 – 5
Firm	6 – 10
Stiff	11 – 20
Very Stiff	21 – 40
Hard	> 40

GRAIN SIZE CLASSIFICATION

Grain Type	*Standard Sieve Size*	*Grain Size in Millimeters*
Boulders	Above 12 inches	Above 305
Cobbles	12 to 13 inches	305 to 76.2
Gravel	3 inches to No. 4	76.2 to 4.76
Coarse-grained	3 to ¾ inches	76.2 to 19.1
Fine-grained	¾ inches to No. 4	19.1 to 4.76
Sand	No. 4 to No. 200	4.76 to 0.074
Coarse-grained	No. 4 to No. 10	4.76 to 2.00
Medium-grained	No. 10 to No. 40	2.00 to 0.042
Fine-grained	No. 40 to No. 200	0.042 to 0.074
Silt and Clay	Below No. 200	Below 0.074

PLASTICITY CHART



Log of Boring B1

Project: Mixed Use Development

Client: Regency Centers, Inc.

Location: SEC 3rd & Fairfax, Los Angeles, CA

Depth to Water> 20 Feet

Initial: 20 Feet

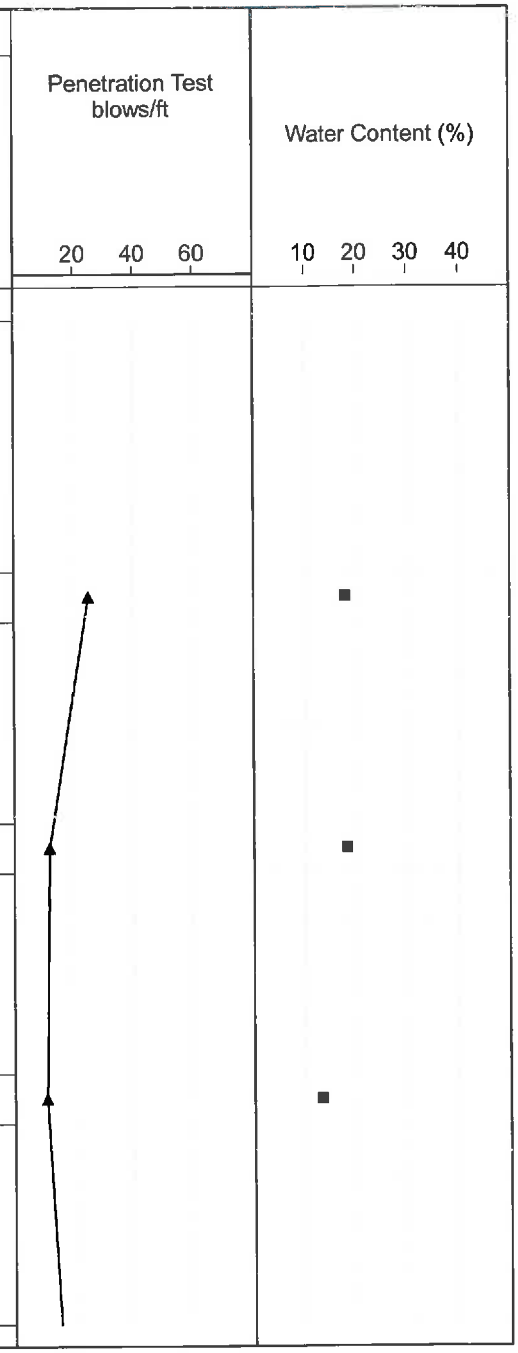
Project No: 112-17043

Figure No.: A-1

Logged By: Jorge Pelayo

At Completion: 20 Feet

SUBSURFACE PROFILE			SAMPLE			
Depth (ft)	Symbol	Description	Dry Density (pcf)	Moisture (%)	Type	Blows/ft.
0		Ground Surface ASPHALT PAVING = 4 inches AGGREGATE BASE = 6 inches				
1–13		***SANDY CLAY (CL)*** Stiff to very stiff, fine-grained; black-brown, moist				
5			111.2	18.0		25
10			105.4	18.3		12
13–19		***SILTY SAND (SM)*** Medium dense, medium- to fine-grained with trace CLAY; black-brown, moist				
15				13.2		11
20		∇				



Drill Method: Hollow Stem

Drill Rig: CME 75

Driller: Jorge Pelayo

Krazan and Associates

Drill Date: 6-8-17

Hole Size: 6½ Inches

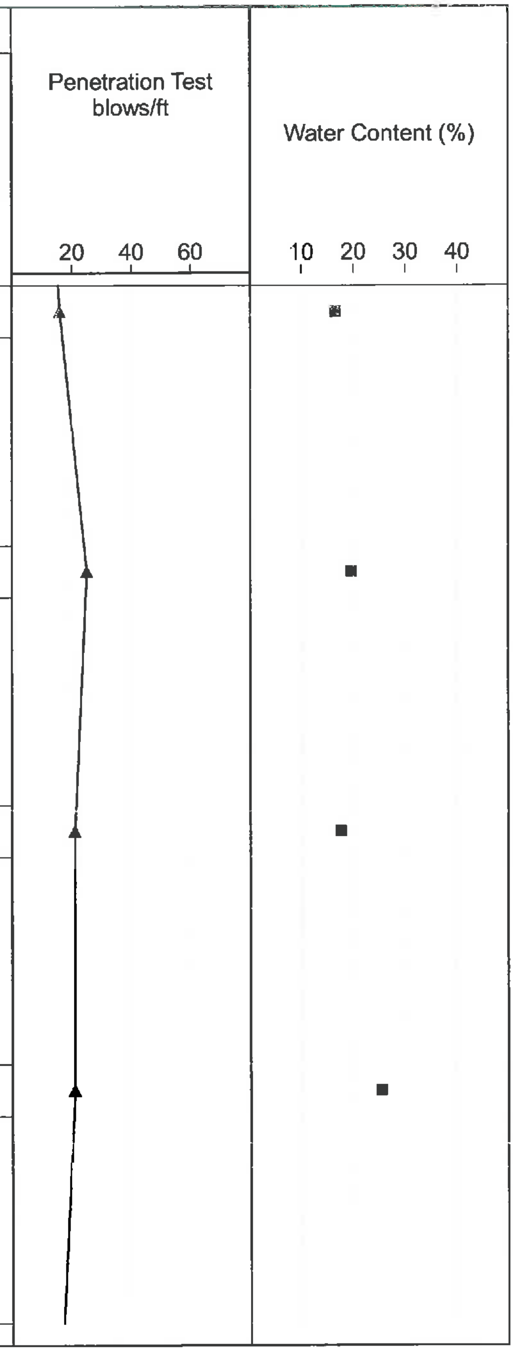
Elevation: 80 Feet

Log of Boring B1

Project: Mixed Use Development
Project No: 112-17043
Client: Regency Centers, Inc.
Figure No.: A-1
Location: SEC 3rd & Fairfax, Los Angeles, CA
Logged By: Jorge Pelayo
Depth to Water> 20 Feet **Initial:** 20 Feet **At Completion:** 20 Feet

SUBSURFACE PROFILE		SAMPLE			
Depth (ft)	Description	Dry Density (pcf)	Moisture (%)	Type	Blows/ft.
20–24	***SAND (SP)*** Medium dense, coarse- to fine-grained, poorly graded; gray, wet		16.6		16
24–29	***SANDY SILT (ML)*** Very stiff, medium- to fine-grained; brown, wet		19.7		25
29–40	***SANDY CLAY (CL)*** Stiff to very stiff, fine-grained; dark gray, wet	112.2	17.6		21
		110.1	25.5		21

Depth (ft): 22, 24, 26, 28, 30, 32, 34, 36, 38, 40



Drill Method: Hollow Stem
Drill Date: 6-8-17
Drill Rig: CME 75
Hole Size: 6½ Inches
Driller: Jorge Pelayo
Elevation: 80 Feet

Krazan and Associates

Log of Boring B1

Project: Mixed Use Development
Project No: 112-17043

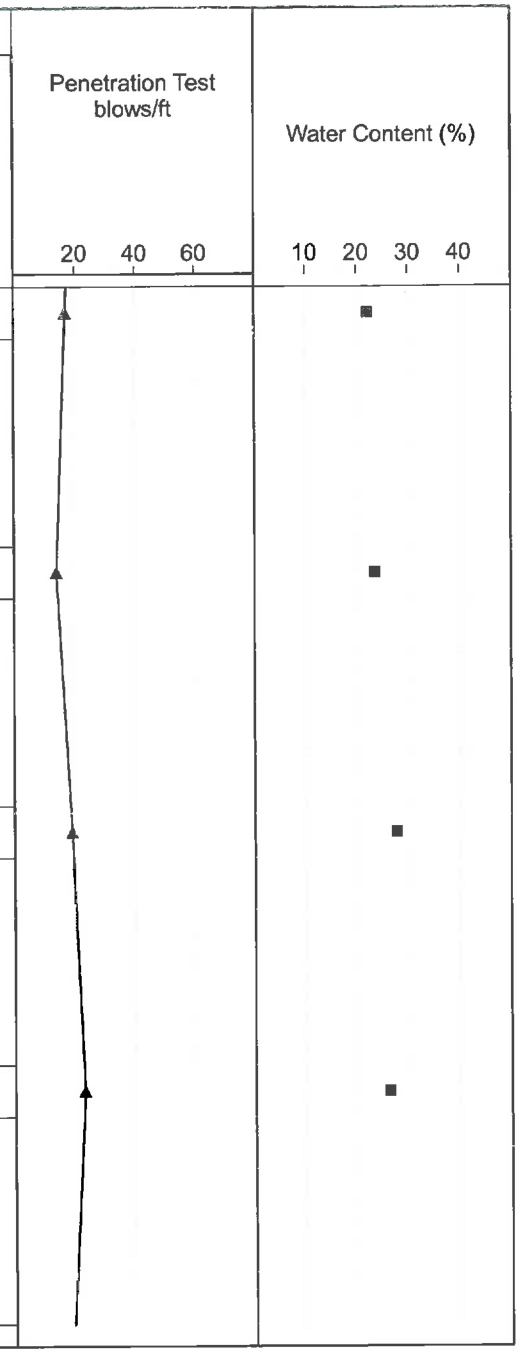
Client: Regency Centers, Inc.
Figure No.: A-1

Location: SEC 3rd & Fairfax, Los Angeles, CA
Logged By: Jorge Pelayo

Depth to Water> 20 Feet
Initial: 20 Feet
At Completion: 20 Feet

SUBSURFACE PROFILE			SAMPLE			
Depth (ft)	Symbol	Description	Dry Density (pcf)	Moisture (%)	Type	Blows/ft.
40–44				22.2		17
44–54		***CLAYEY SAND (SC)*** Meduim dense, medium- to fine-grained; dark gray, wet		23.7		14
				27.8		19
54–60		***SAND (SP)*** Medium dense, fine-grained, ASPHALT SAND, with TAR, poorly graded; black, wet		26.4		23

Depth markings: 42, 44, 46, 48, 50, 52, 54, 56, 58, 60



Drill Method: Hollow Stem
Drill Date: 6-8-17

Drill Rig: CME 75
Krazan and Associates
Hole Size: 6½ Inches

Driller: Jorge Pelayo
Elevation: 80 Feet

Log of Boring B1

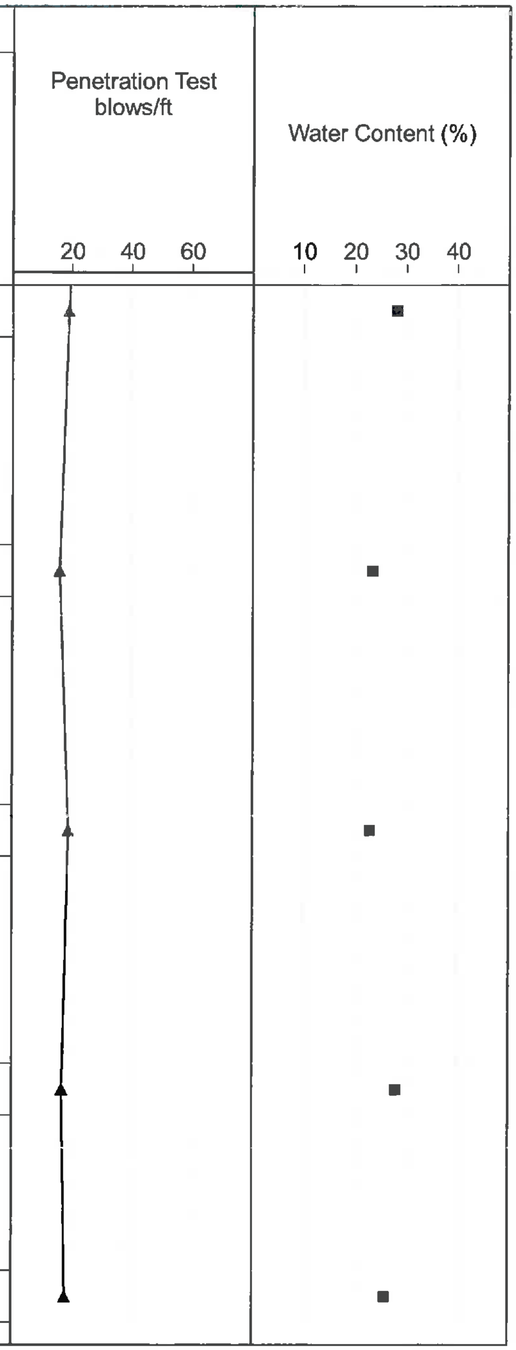
Project: Mixed Use Development
Project No: 112-17043

Client: Regency Centers, Inc.
Figure No.: A-1

Location: SEC 3rd & Fairfax, Los Angeles, CA
Logged By: Jorge Pelayo

Depth to Water> 20 Feet
Initial: 20 Feet
At Completion: 20 Feet

SUBSURFACE PROFILE			SAMPLE			
Depth (ft)	Symbol	Description	Dry Density (pcf)	Moisture (%)	Type	Blows/ft.
				28.2		19
62						
64						
				23.5		16
66						
68						
70						
				23.0		19
72						
74						
				28.1		17
76						
78		Water encountered at 21 feet Boring backfilled with soil cuttings				
				26.0		18
80						



Drill Method: Hollow Stem
Drill Date: 6-8-17

Drill Rig: CME 75
Krazan and Associates
Hole Size: 6½ Inches

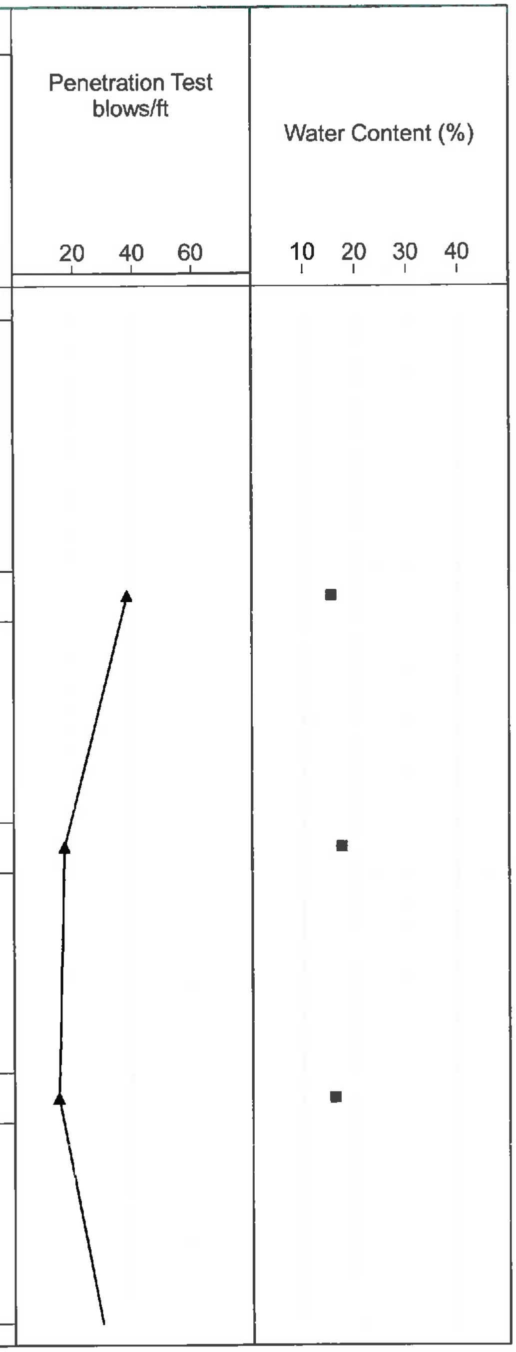
Driller: Jorge Pelayo
Elevation: 80 Feet

Log of Boring B2

Project: Mixed Use Development
Client: Regency Centers, Inc.
Location: SEC 3rd & Fairfax, Los Angeles, CA
Depth to Water> 21 Feet **Initial:** 21 Feet

Project No: 112-17043
Figure No.: A-2
Logged By: Jorge Pelayo
At Completion: 21 Feet

SUBSURFACE PROFILE			SAMPLE			
Depth (ft)	Symbol	Description	Dry Density (pcf)	Moisture (%)	Type	Blows/ft.
		Ground Surface				
0		ASPHALT PAVING = 4 inches AGGREGATE BASE = 6 inches				
1–14		***SANDY CLAY (CL)*** Stiff to very stiff, medium- to fine-grained; gray, moist				
5			111.0	15.4		38
10			109.5	17.4		17
14–19		***SILTY SAND (SM)*** Medium dense, fine-grained; gray, moist				
15				16.1		15
20						



Drill Method: Hollow Stem
Drill Rig: CME 75
Driller: Jorge Pelayo

Krazan and Associates

Drill Date: 6-8-17
Hole Size: 6½ Inches
Elevation: 40 Feet

Log of Boring B2

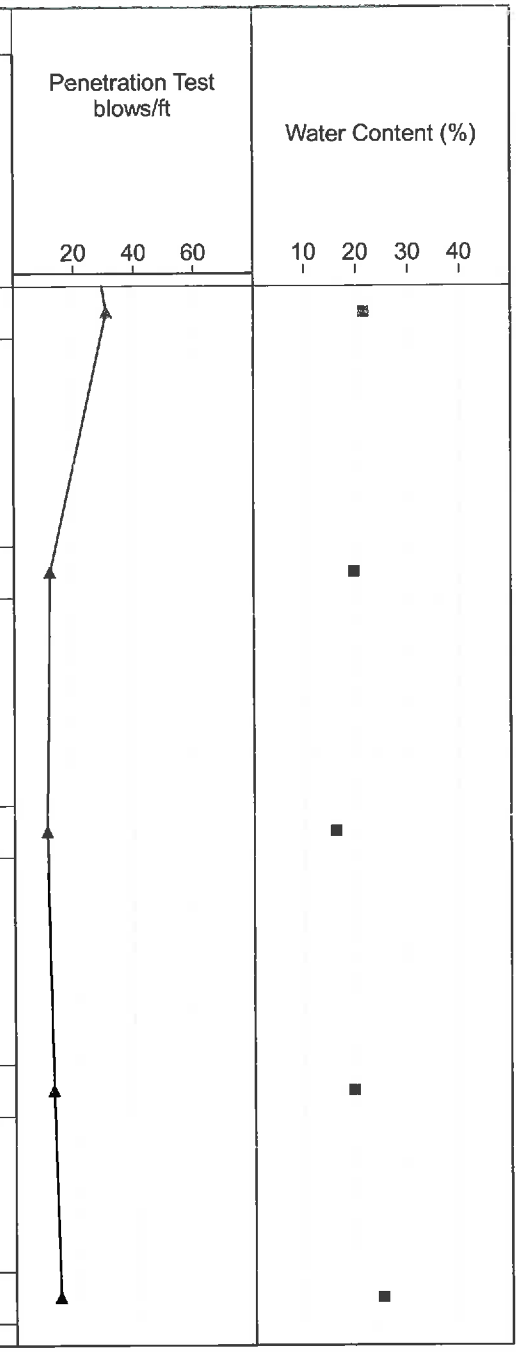
Project: Mixed Use Development
Project No: 112-17043

Client: Regency Centers, Inc.
Figure No.: A-2

Location: SEC 3rd & Fairfax, Los Angeles, CA
Logged By: Jorge Pelayo

Depth to Water> 21 Feet
Initial: 21 Feet
At Completion: 21 Feet

SUBSURFACE PROFILE			SAMPLE			
Depth (ft)	Symbol	Description	Dry Density (pcf)	Moisture (%)	Type	Blows/ft.
21		***SAND (SP)*** Dense, coarse- to fine-grained, poorly graded; gray, wet		21.6		31
24		***SANDY SILT (ML)*** Medium stiff to stiff, medium- to fine-grained; gray with brown, wet				
25.5				19.7		12
29		***SANDY CLAY (CL)*** Stiff to very stiff, fine-grained; dark gray, wet				
30.5				16.3		11
35.5				19.6		13
38		Water encountered at 21 feet Boring backfilled with soil cuttings				
39.5				25.1		15
40						



Drill Method: Hollow Stem
Drill Date: 6-8-17

Drill Rig: CME 75
Krazan and Associates
Hole Size: 6½ Inches

Driller: Jorge Pelayo
Elevation: 40 Feet

Log of Boring B3

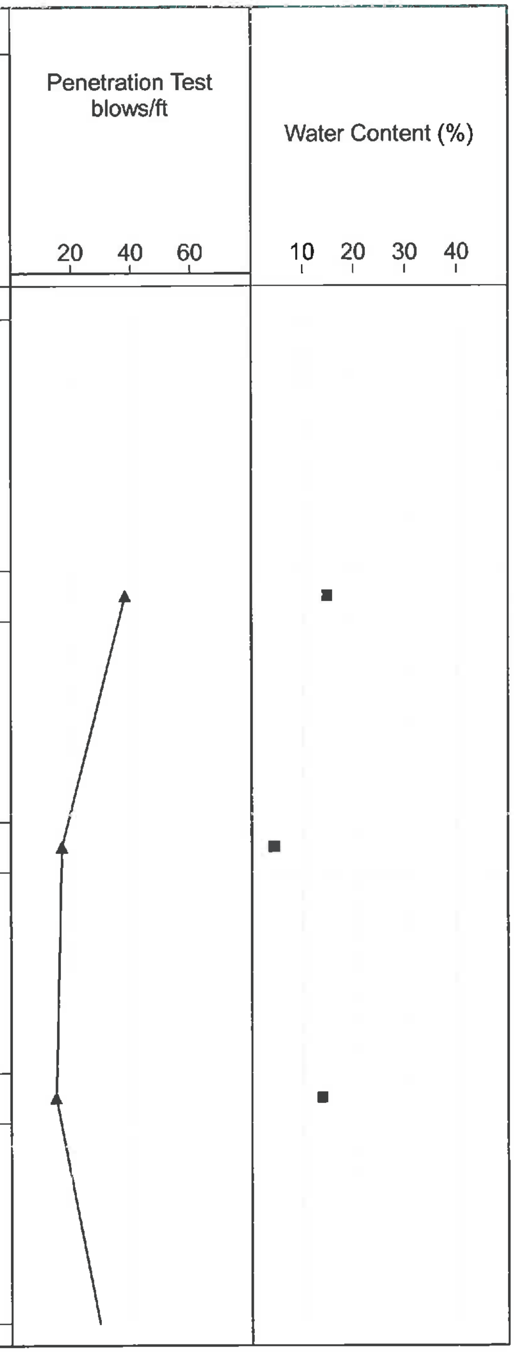
Project: Mixed Use Development
Project No: 112-17043

Client: Regency Centers, Inc.
Figure No.: A-3

Location: SEC 3rd & Fairfax, Los Angeles, CA
Logged By: Jorge Pelayo

Depth to Water> 22 Feet
Initial: 22 Feet
At Completion: 22 Feet

Depth (ft)	Description	Dry Density (pcf)	Moisture (%)	Type	Blows/ft.
0	Ground Surface				
0	ASPHALT PAVING = 4 inches AGGREGATE BASE = 6 inches				
1	***SANDY CLAY (CL)*** Stiff to very stiff, medium- to fine-grained; gray, moist				
5		113.6	14.8		22
10		119.6	4.5		29
14	***SILTY SAND (SM)*** Medium dense to dense, coarse- to fine-grained; gray, moist to wet				
15		117.4	13.9		30



Drill Method: Hollow Stem
Drill Date: 6-8-17

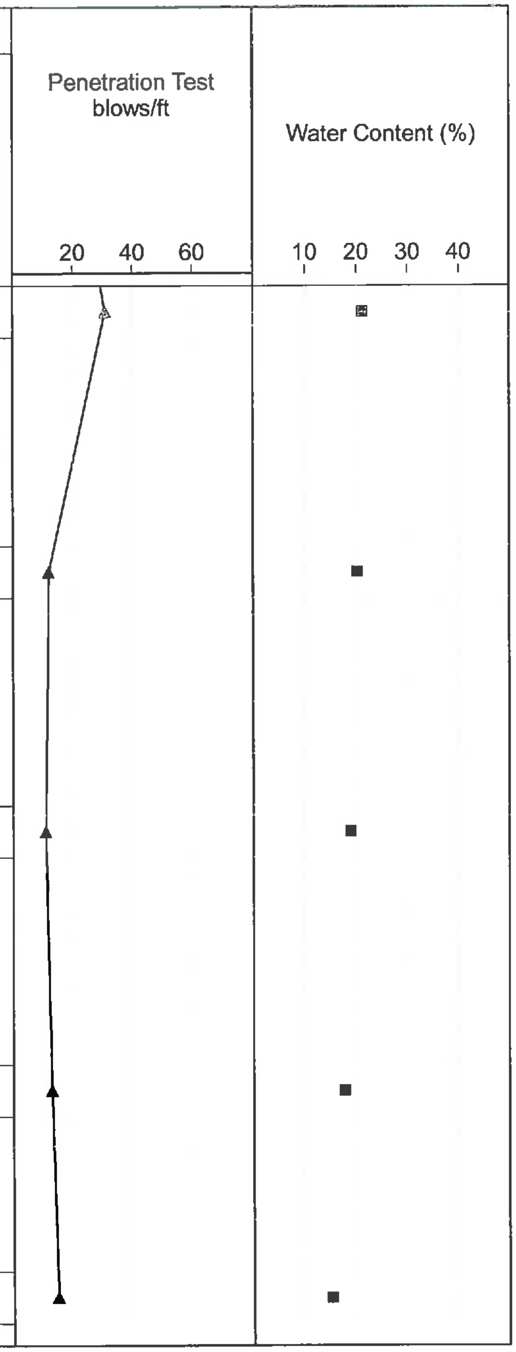
Drill Rig: CME 75
Krazan and Associates
Hole Size: 6½ Inches

Driller: Jorge Pelayo
Elevation: 40 Feet

Log of Boring B3

Project: Mixed Use Development
Project No: 112-17043
Client: Regency Centers, Inc.
Figure No.: A-3
Location: SEC 3rd & Fairfax, Los Angeles, CA
Logged By: Jorge Pelayo
Depth to Water> 22 Feet
Initial: 22 Feet
At Completion: 22 Feet

SUBSURFACE PROFILE			SAMPLE			
Depth (ft)	Symbol	Description	Dry Density (pcf)	Moisture (%)	Type	Blows/ft.
				21.3	■	17
22		▽				
24		***SANDY SILT (ML)*** Hard, fine-grained; gray, wet				
				20.3	■	42
26						
28						
		SANDY CLAY (CL) Very stiff to hard, fine-grained; dark gray, wet				
30				19.0	■	27
32						
34						
				17.8	■	22
36						
38		Water encountered at 22 feet Boring backfilled with soil cuttings				
				15.3	■	75
40						



Drill Method: Hollow Stem
Drill Date: 6-8-17
Drill Rig: CME 75
Krazan and Associates
Hole Size: 6½ Inches
Driller: Jorge Pelayo
Elevation: 40 Feet
Sheet: 2 of 2

Log of Boring B4

Project: Mixed Use Development

Project No: 112-17043

Client: Regency Centers, Inc.

Figure No.: A-4

Location: SEC 3rd & Fairfax, Los Angeles, CA

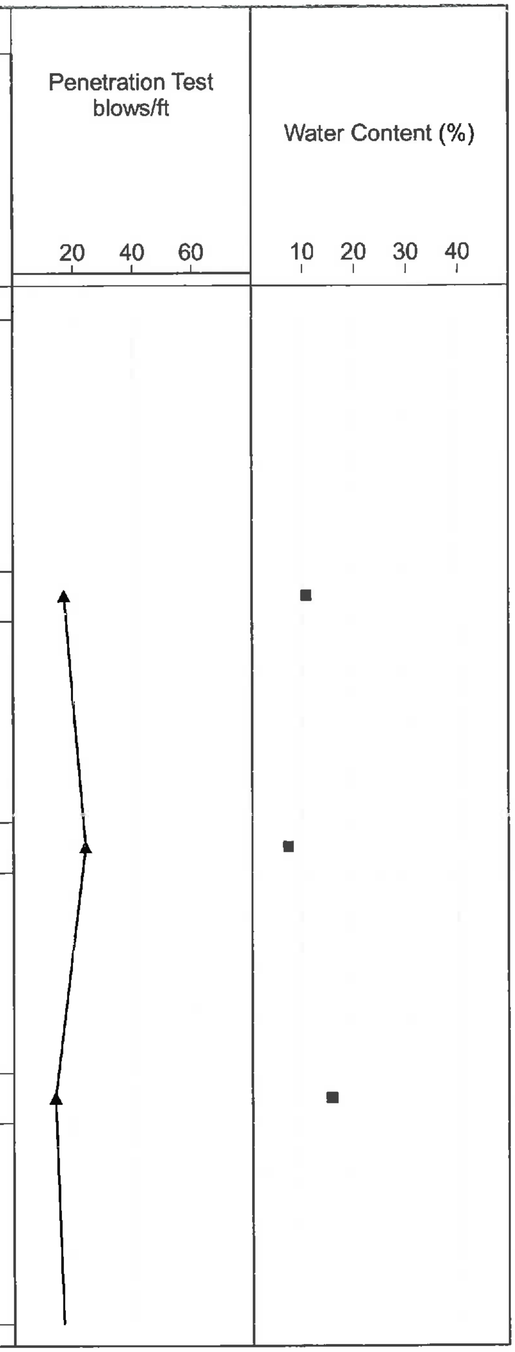
Logged By: Jorge Pelayo

Depth to Water> 20 Feet

Initial: 20 Feet

At Completion: 20 Feet

Depth (ft)	Description	Dry Density (pcf)	Moisture (%)	Type	Blows/ft.
0	Ground Surface				
0	ASPHALT PAVING = 4 inches AGGREGATE BASE = 6 inches				
1–19	***SANDY CLAY (CL)*** Stiff to very stiff, fine-grained; dark brown, moist				
~5.5		124.3	10.8		17
~10.5		107.6	7.2		24
~15.5			15.8		14
20	∇				



Drill Method: Hollow Stem

Drill Date: 6-8-17

Drill Rig: CME 75

Krazan and Associates

Hole Size: 6½ Inches

Driller: Jorge Pelayo

Elevation: 80 Feet

Sheet: 1 of 4

Log of Boring B4

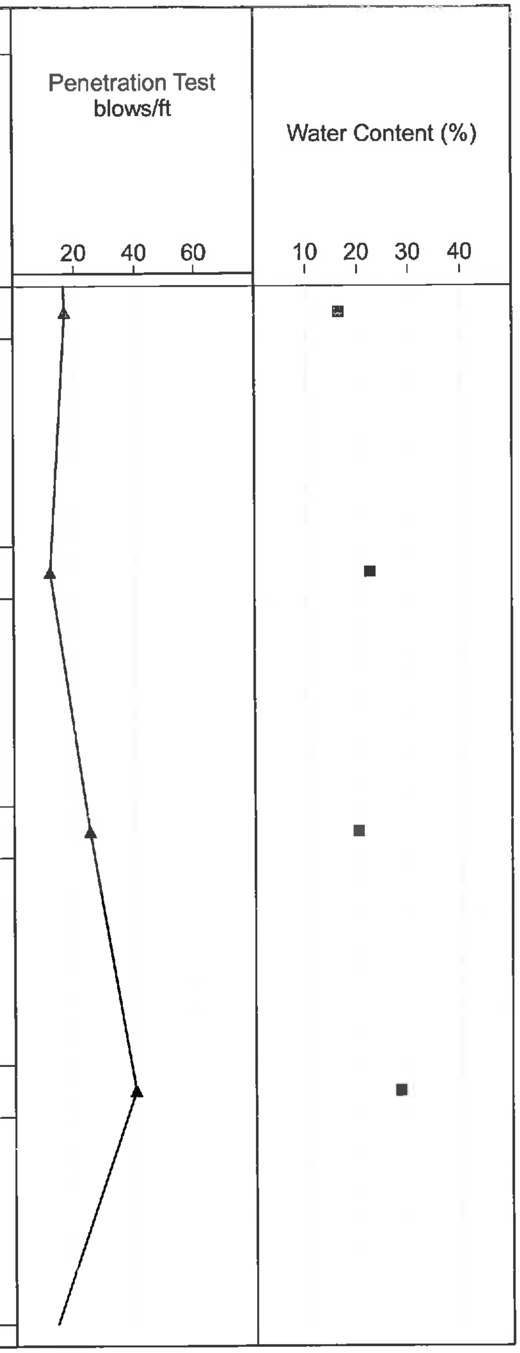
Project: Mixed Use Development
Project No: 112-17043

Client: Regency Centers, Inc.
Figure No.: A-4

Location: SEC 3rd & Fairfax, Los Angeles, CA
Logged By: Jorge Pelayo

Depth to Water> 20 Feet
Initial: 20 Feet
At Completion: 20 Feet

SUBSURFACE PROFILE			SAMPLE			
Depth (ft)	Symbol	Description	Dry Density (pcf)	Moisture (%)	Type	Blows/ft.
		SILTY SAND (SM) Medium dense, medium- to fine-grained with CLAY; gray, wet		16.5		17
22						
24						
				22.6		12
26						
28						
		SANDY CLAY (CL) Stiff to hard, medium- to fine-grained; gray, wet				
30			106.6	20.4		25
32						
34						
			102.4	28.5		40
36						
38						
40						



Drill Method: Hollow Stem
Drill Date: 6-8-17

Drill Rig: CME 75
Krazan and Associates
Hole Size: 6½ Inches

Driller: Jorge Pelayo
Elevation: 80 Feet

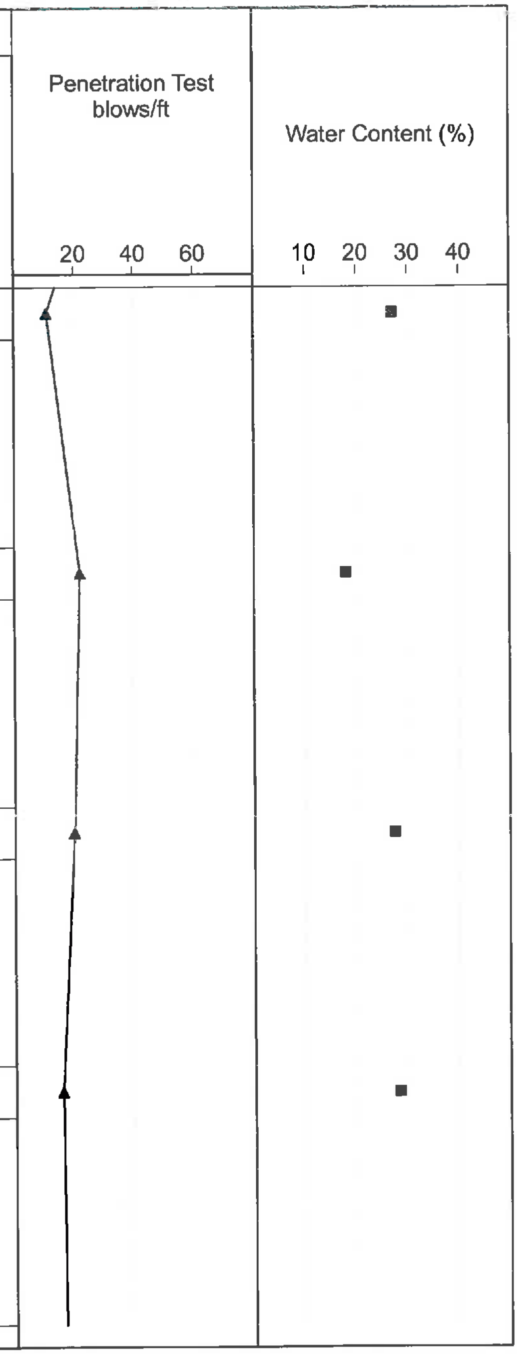
Log of Boring B4

Project: Mixed Use Development
Client: Regency Centers, Inc.
Location: SEC 3rd & Fairfax, Los Angeles, CA
Depth to Water> 20 Feet **Initial:** 20 Feet **At Completion:** 20 Feet

Project No: 112-17043
Figure No.: A-4
Logged By: Jorge Pelayo

SUBSURFACE PROFILE		SAMPLE			
Depth (ft)	Description	Dry Density (pcf)	Moisture (%)	Type	Blows/ft.
40–43			27.1		11
43–53	***CLAYEY SAND (SC)*** Meduim dense, medium- to fine-grained; dark gray, wet		18.1		22
			27.5		20
53–60	***SAND (SP)*** Medium dense, fine-grained, ASPHALT SAND, with TAR, poorly graded; black, wet		28.4		16

Depth scale: 42, 44, 46, 48, 50, 52, 54, 56, 58, 60



Drill Method: Hollow Stem
Drill Rig: CME 75
Driller: Jorge Pelayo

Krazan and Associates

Drill Date: 6-8-17
Hole Size: 6½ Inches
Elevation: 80 Feet
Sheet: 3 of 4

Log of Boring B4

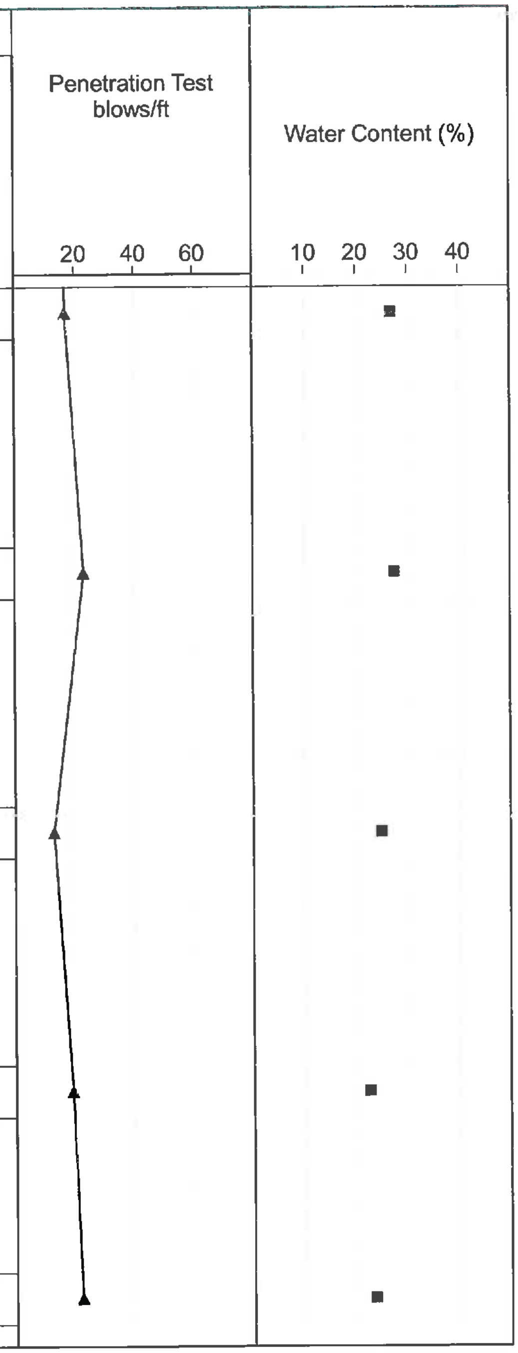
Project: Mixed Use Development
Project No: 112-17043

Client: Regency Centers, Inc.
Figure No.: A-4

Location: SEC 3rd & Fairfax, Los Angeles, CA
Logged By: Jorge Pelayo

Depth to Water> 20 Feet
Initial: 20 Feet
At Completion: 20 Feet

SUBSURFACE PROFILE			SAMPLE			
Depth (ft)	Symbol	Description	Dry Density (pcf)	Moisture (%)	Type	Blows/ft.
				26.9		17
62						
64						
				27.6		23
66						
68						
70						
				25.1		13
72						
74						
				22.6		19
76						
78						
		Water encountered at 20 feet Boring backfilled with soil cuttings		23.6		22
80						



Drill Method: Hollow Stem
Drill Date: 6-8-17

Drill Rig: CME 75
Krazan and Associates
Hole Size: 6½ Inches

Driller: Jorge Pelayo
Elevation: 80 Feet

Log of Boring B5

Project: Mixed Use Development
Project No: 112-17043
Client: Regency Centers, Inc.
Figure No.: A-5
Location: SEC 3rd & Fairfax, Los Angeles, CA
Logged By: Jorge Pelayo
Depth to Water> Not Encountered **Initial:** N/A **At Completion:** N/A

SUBSURFACE PROFILE			SAMPLE				Penetration Test blows/ft (20 40 60)	Water Content (%) (10 20 30 40)
Depth (ft)	Symbol	Description	Dry Density (pcf)	Moisture (%)	Type	Blows/ft.		
		Ground Surface						
0		ASPHALT PAVING = 4 inches AGGREGATE BASE = 5 inches						
2		***SANDY CLAY (CL)*** Stiff, medium- to fine-grained; dark brown, moist		11.5				
				11.6				
4		End of Borehole						
6								
8								
10								
12								
14								
16								
18		No water encountered Boring backfilled with soil cuttings						
20								

Drill Method: Hollow Stem
Drill Date: 6-8-17
Drill Rig: CME 75
Krazan and Associates
Hole Size: 6½ Inches
Driller: Jorge Pelayo
Elevation: 4 Feet
Sheet: 1 of 1

Log of Boring B6

Project: Mixed Use Development

Project No: 112-17043

Client: Regency Centers, Inc.

Figure No.: A-6

Location: SEC 3rd & Fairfax, Los Angeles, CA

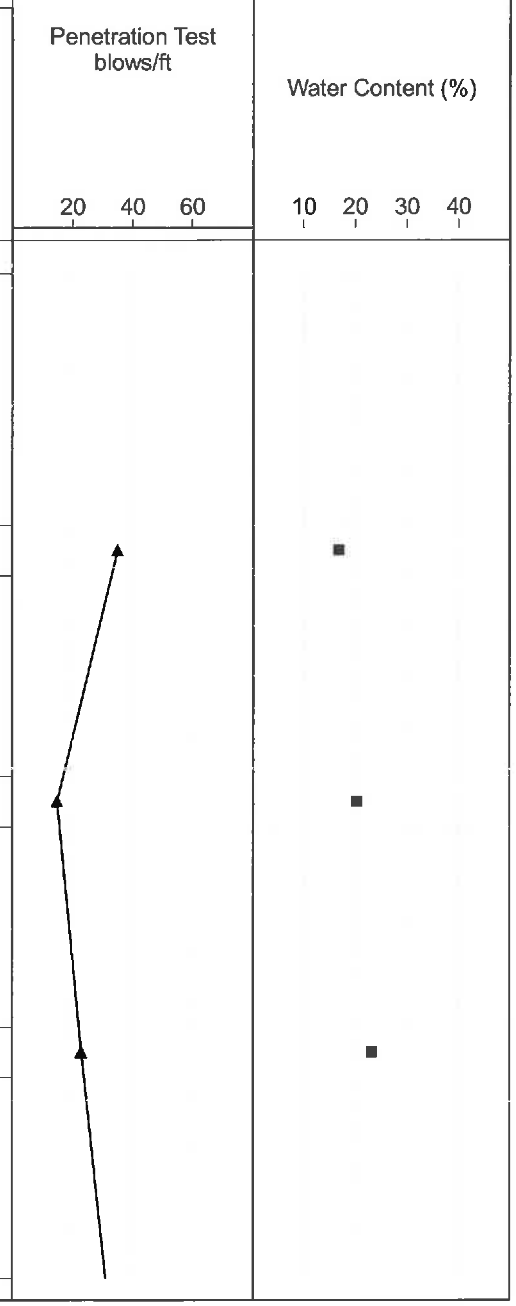
Logged By: Jorge Pelayo

Depth to Water> 21 Feet

Initial: 21 Feet

At Completion: 21 Feet

SUBSURFACE PROFILE			SAMPLE			
Depth (ft)	Symbol	Description	Dry Density (pcf)	Moisture (%)	Type	Blows/ft.
		Ground Surface				
0		ASPHALT PAVING = 4 inches AGGREGATE BASE = 6 inches				
1–14		***SANDY CLAY (CL)*** Stiff to very stiff, medium- to fine-grained; gray, moist				
5				17.0		35
10				20.4		15
14–19		***SILTY SAND (SM)*** Medium dense, fine-grained; gray, moist				
15				23.3		23



Drill Method: Hollow Stem

Drill Date: 6-8-17

Drill Rig: CME 75

Krazan and Associates

Hole Size: 6½ Inches

Driller: Jorge Pelayo

Elevation: 40 Feet

Sheet: 1 of 2

Log of Boring B6

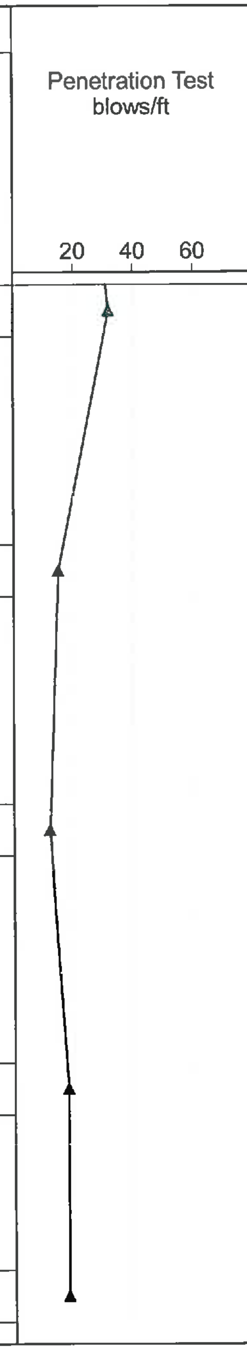
Project: Mixed Use Development
Project No: 112-17043

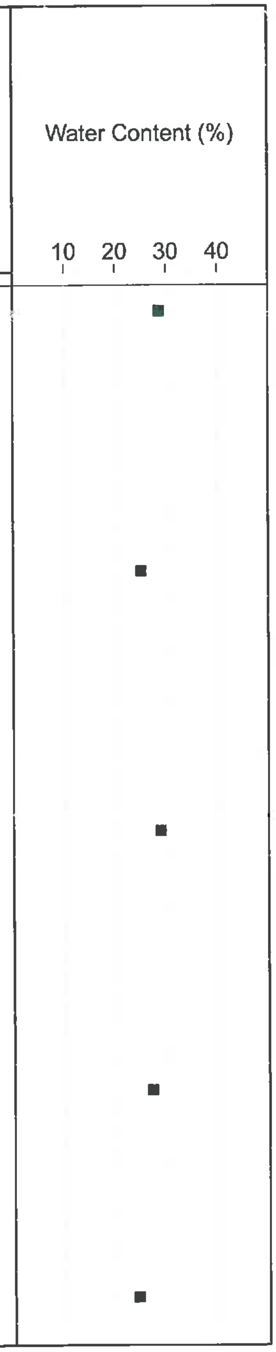
Client: Regency Centers, Inc.
Figure No.: A-6

Location: SEC 3rd & Fairfax, Los Angeles, CA
Logged By: Jorge Pelayo

Depth to Water> 21 Feet
Initial: 21 Feet
At Completion: 21 Feet

SUBSURFACE PROFILE			SAMPLE			
Depth (ft)	Symbol	Description	Dry Density (pcf)	Moisture (%)	Type	Blows/ft.
	▨	***SAND (SP)*** Dense, coarse- to fine-grained, poorly graded; gray, wet		28.7	■	32
22						
24	▥	***SANDY SILT (ML)*** Stiff, medium- to fine-grained; gray, wet				
				25.1	■	15
26						
28						
30				28.9	■	12
32	▨	***SANDY CLAY (CL)*** Very stiff, fine-grained; dark gray, wet				
34						
				27.2	■	18
36						
38		Water encountered at 21 feet Boring backfilled with soil cuttings				
				24.4	■	18
40						





Drill Method: Hollow Stem
Drill Date: 6-8-17

Drill Rig: CME 75
Krazan and Associates
Hole Size: 6½ Inches

Driller: Jorge Pelayo
Elevation: 40 Feet

Sheet: 2 of 2

Log of Boring B7

Project: Mixed Use Development
Project No: 112-17043

Client: Regency Centers, Inc.
Figure No.: A-6

Location: SEC 3rd & Fairfax, Los Angeles, CA
Logged By: Jorge Pelayo

Depth to Water> Not Encountered
Initial: N/A
At Completion: N/A

SUBSURFACE PROFILE			SAMPLE				Penetration Test blows/ft	Water Content (%)
Depth (ft)	Symbol	Description	Dry Density (pcf)	Moisture (%)	Type	Blows/ft.	20 40 60	10 20 30 40
		Ground Surface						
0		ASPHALT PAVING = 4 inches AGGREGATE BASE = 6 inches						
2		***SANDY CLAY (CL)*** Stiff to very stiff, medium- to fine-grained; dark brown, moist		7.8				
				12.1				
4		End of Borehole						
6								
8								
10								
12								
14								
16								
18		No water encountered Boring backfilled with soil cuttings						
20								

Drill Method: Hollow Stem
Drill Date: 6-8-17

Drill Rig: CME 75
Krazan and Associates
Hole Size: 6½ Inches

Driller: Jorge Pelayo
Elevation: 4 Feet

Sheet: 1 of 1

Log of Boring B8

Project: Mixed Use Development
Project No: 112-17043

Client: Regency Centers, Inc.
Figure No.: A-8

Location: SEC 3rd & Fairfax, Los Angeles, CA
Logged By: Jorge Pelayo

Depth to Water> Not Encountered
Initial: N/A
At Completion: N/A

SUBSURFACE PROFILE			SAMPLE				Penetration Test blows/ft	Water Content (%)
Depth (ft)	Symbol	Description	Dry Density (pcf)	Moisture (%)	Type	Blows/ft.	20 40 60	10 20 30 40
		Ground Surface						
0		ASPHALT PAVING = 3½ inches AGGREGATE BASE = 5 inches						
		SANDY CLAY (CL) Stiff to very stiff, medium- to fine-grained; dark brown, moist						
2				7.8				
				15.3				
4		End of Borehole						
18		No water encountered Boring backfilled with soil cuttings						
20								

Drill Method: Hollow Stem
Drill Date: 6-8-17

Drill Rig: CME 75
Hole Size: 6½ Inches

Krazan and Associates

Driller: Jorge Pelayo
Elevation: 4 Feet

Sheet: 1 of 1

Log of Boring B9

Project: Mixed Use Development
Project No: 112-17043

Client: Regency Centers, Inc.
Figure No.: A-9

Location: SEC 3rd & Fairfax, Los Angeles, CA
Logged By: Jorge Pelayo

Depth to Water> Not Encountered
Initial: N/A
At Completion: N/A

SUBSURFACE PROFILE			SAMPLE				Penetration Test blows/ft	Water Content (%)
Depth (ft)	Symbol	Description	Dry Density (pcf)	Moisture (%)	Type	Blows/ft.	20 40 60	10 20 30 40
		Ground Surface						
0		ASPHALT PAVING = 4 inches AGGREGATE BASE = 5½ inches						
2		***SANDY CLAY (CL)*** Stiff to very stiff, medium- to fine-grained; dark brown, moist		9.4				
				14.9				
4		End of Borehole						
6								
8								
10								
12								
14								
16								
18		No water encountered Boring backfilled with soil cuttings						
20								

Drill Method: Hollow Stem
Drill Date: 6-8-17

Drill Rig: CME 75
Krazan and Associates
Hole Size: 6½ Inches

Driller: Jorge Pelayo
Elevation: 4 Feet

Log of Boring B10

Project: Mixed Use Development | **Project No:** 112-17043

Client: Regency Centers, Inc. | **Figure No.:** A-10

Location: SEC 3rd & Fairfax, Los Angeles, CA | **Logged By:** Jorge Pelayo

Depth to Water> Not Encountered | **Initial:** N/A | **At Completion:** N/A

Depth (ft)	Symbol	Description	Dry Density (pcf)	Moisture (%)	Type	Blows/ft.	Penetration Test blows/ft (20 40 60)	Water Content (%) (10 20 30 40)
		Ground Surface						
0		ASPHALT PAVING = 3½ inches AGGREGATE BASE = 6 inches						
1–4		***SANDY CLAY (CL)*** Stiff to very stiff, medium- to fine-grained; dark brown, moist		8.3				■
				14.5				■
4		End of Borehole						
18		No water encountered Boring backfilled with soil cuttings						
20								

Drill Method: Hollow Stem | **Drill Date:** 6-8-17

Drill Rig: CME 75 | **Krazan and Associates** | **Hole Size:** 6½ Inches

Driller: Jorge Pelayo | **Elevation:** 4 Feet

Sheet: 1 of 1

Log of Boring B11

Project: Mixed Use Development
Project No: 112-17043

Client: Regency Centers, Inc.
Figure No.: A-11

Location: SEC 3rd & Fairfax, Los Angeles, CA
Logged By: Jorge Pelayo

Depth to Water> Not Encountered
Initial: N/A
At Completion: N/A

SUBSURFACE PROFILE			SAMPLE				Penetration Test blows/ft	Water Content (%)
Depth (ft)	Symbol	Description	Dry Density (pcf)	Moisture (%)	Type	Blows/ft.	20 40 60	10 20 30 40
0		Ground Surface						
0–1		ASPHALT PAVING = 4 inches AGGREGATE BASE = 6 inches						
1–4		***SANDY CLAY (CL)*** Stiff to very stiff, medium- to fine-grained; dark brown, moist						
				11.0				
				15.9				
4		End of Borehole						
18		No water encountered Boring backfilled with soil cuttings						
20								

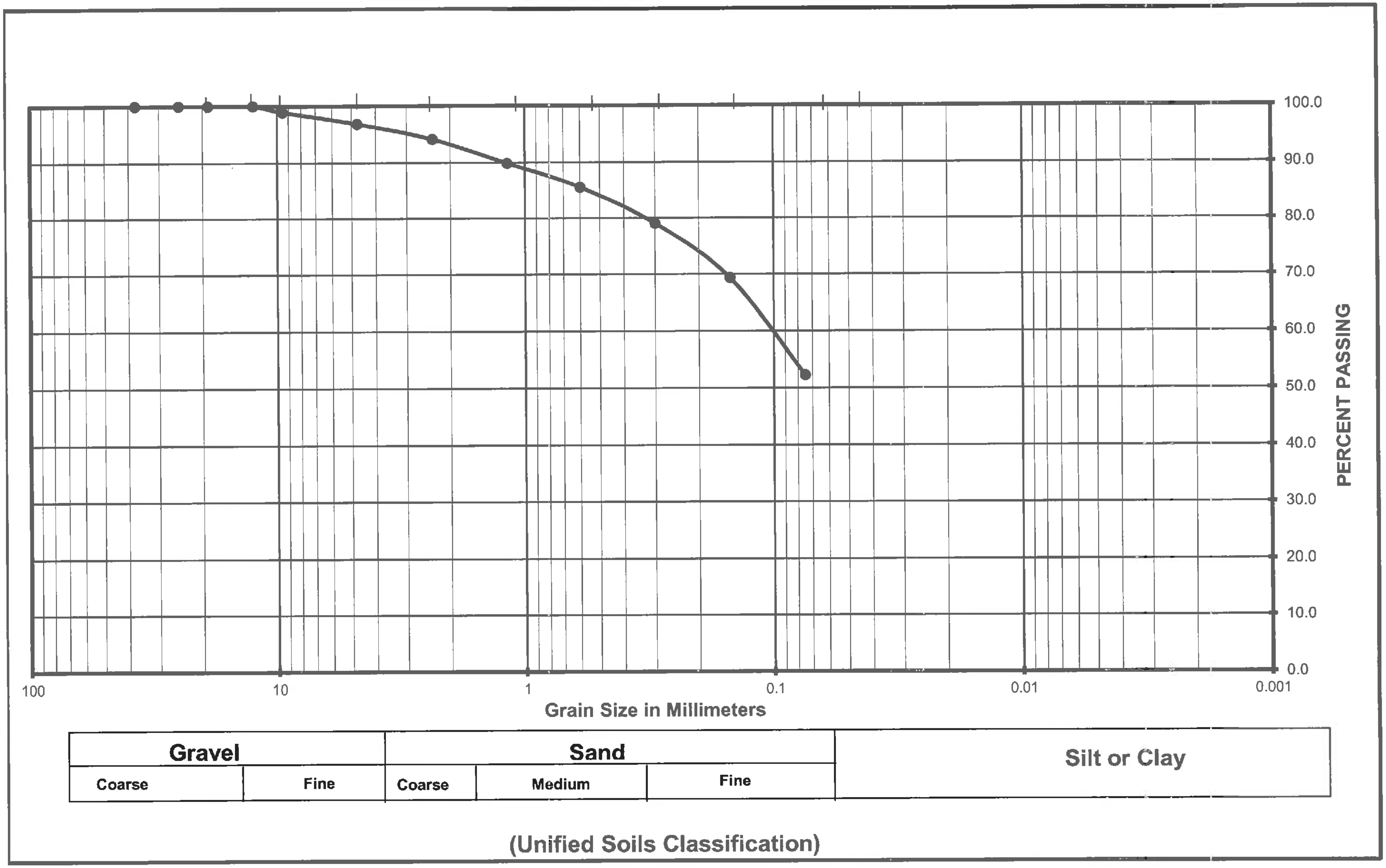
Drill Method: Hollow Stem
Drill Date: 6-8-17

Drill Rig: CME 75
Krazan and Associates
Hole Size: 6½ Inches

Driller: Jorge Pelayo
Elevation: 4 Feet

Sheet: 1 of 1

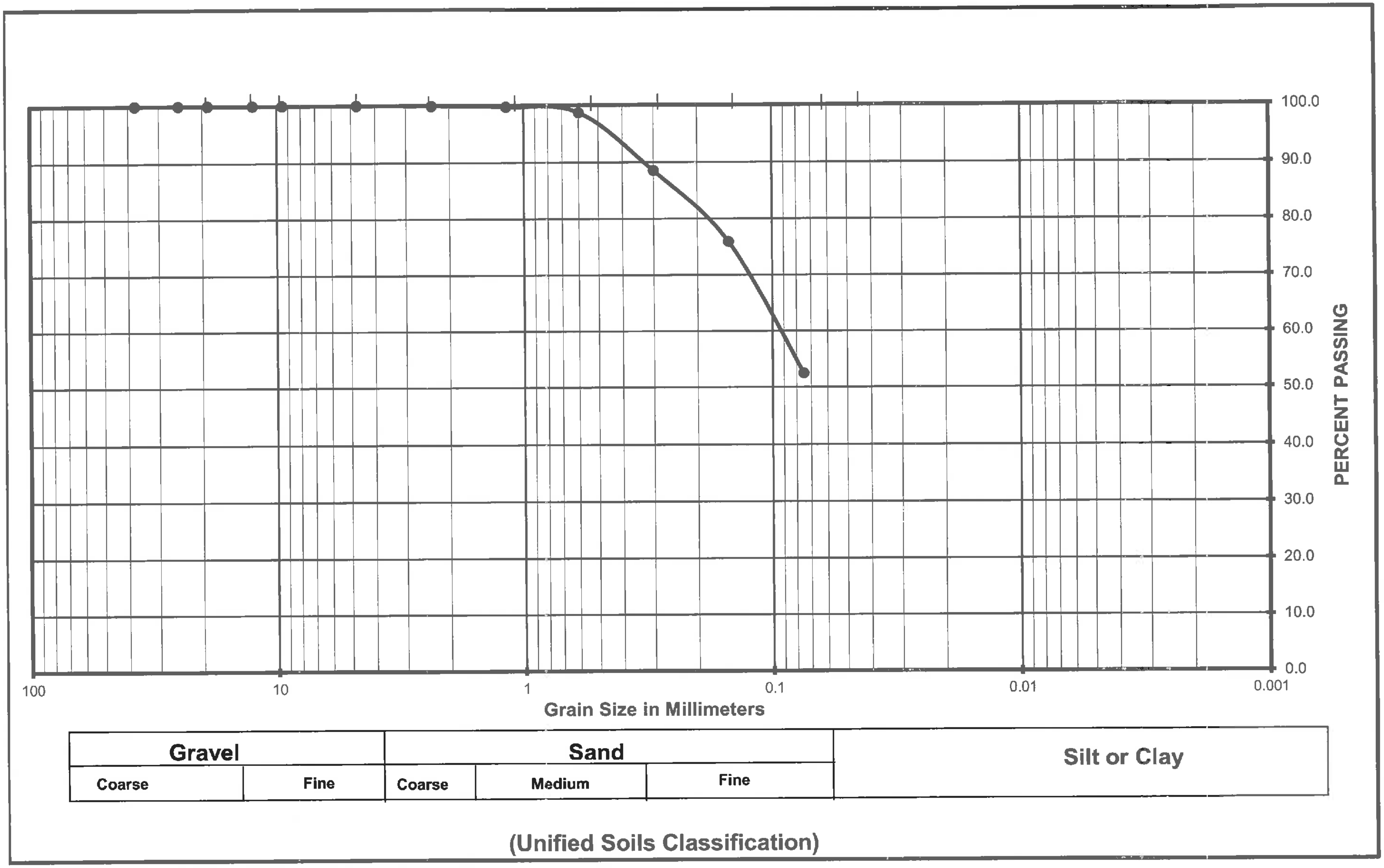
Grain Size Analysis



Project Name	Mixed Use Development LA
Project Number	11217043
Soil Classification	CL
Sample Number	B-1 @ 5'

Krazan Testing Laboratory

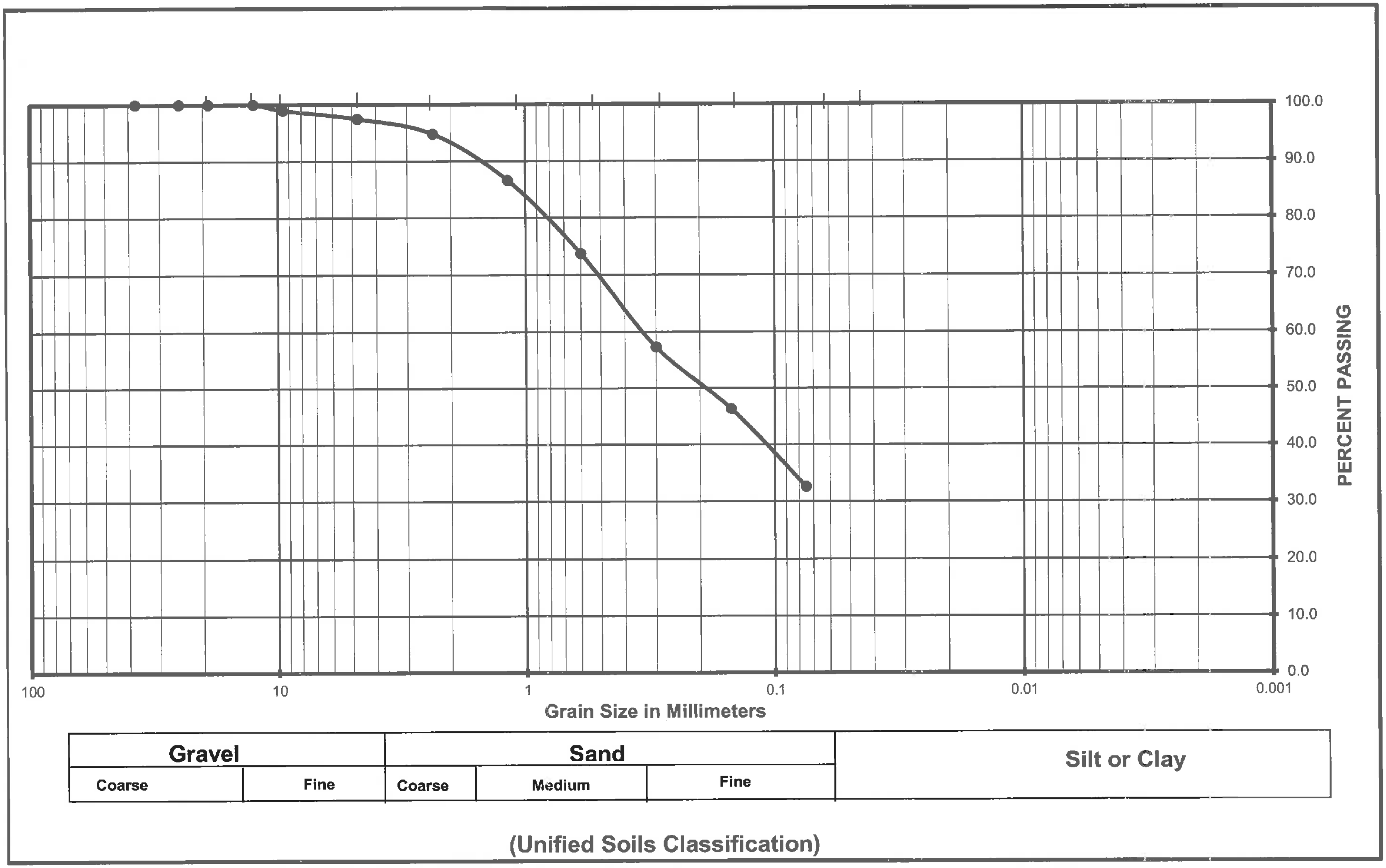
Grain Size Analysis



Project Name	Mixed Use Development LA
Project Number	11217043
Soil Classification	CL
Sample Number	B-1 @ 10'

Krazan Testing Laboratory

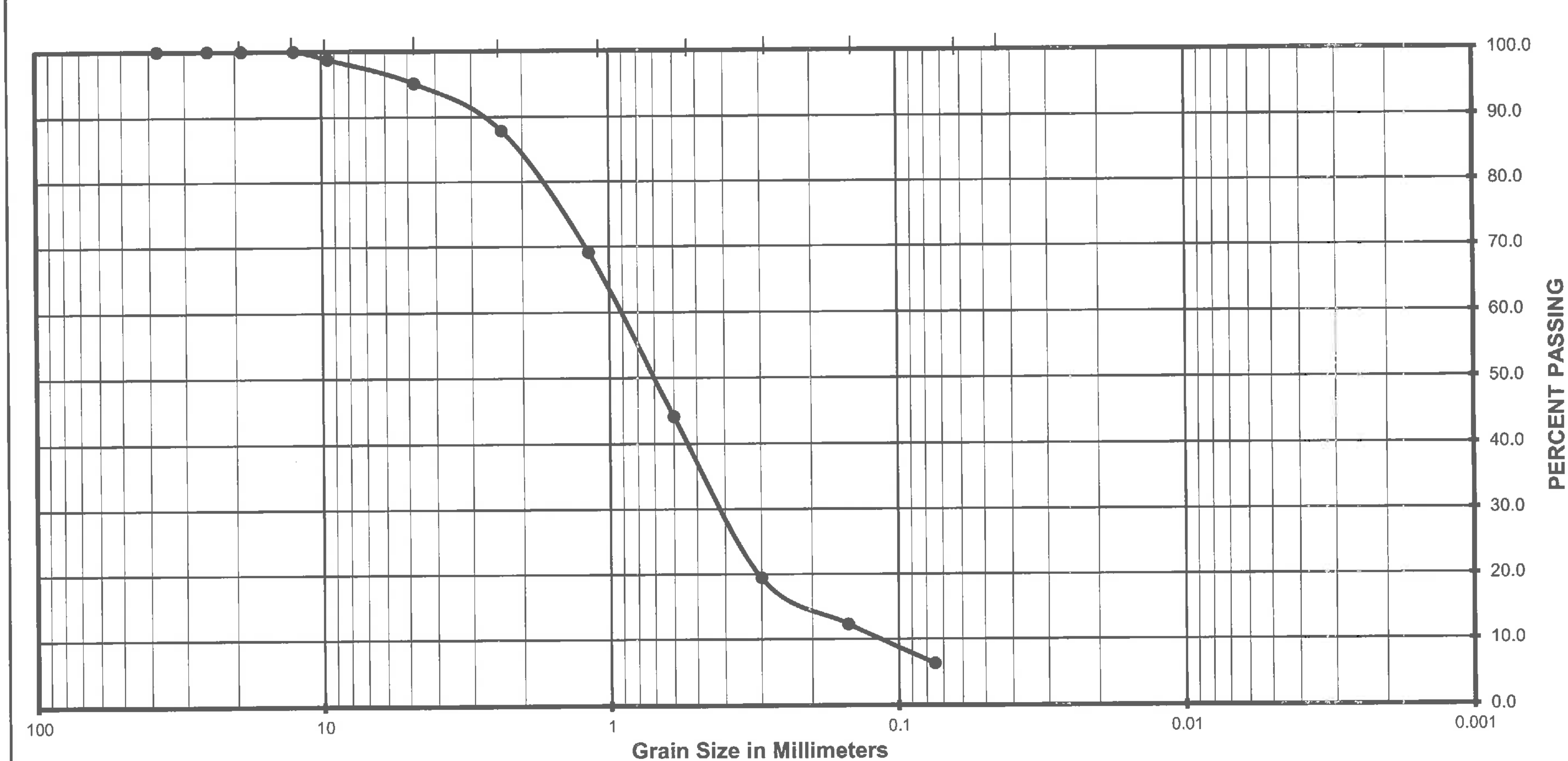
Grain Size Analysis



Project Name	Mixed Use Development LA
Project Number	11217043
Soil Classification	SM
Sample Number	B-1 @ 15'

Krazan Testing Laboratory

Grain Size Analysis



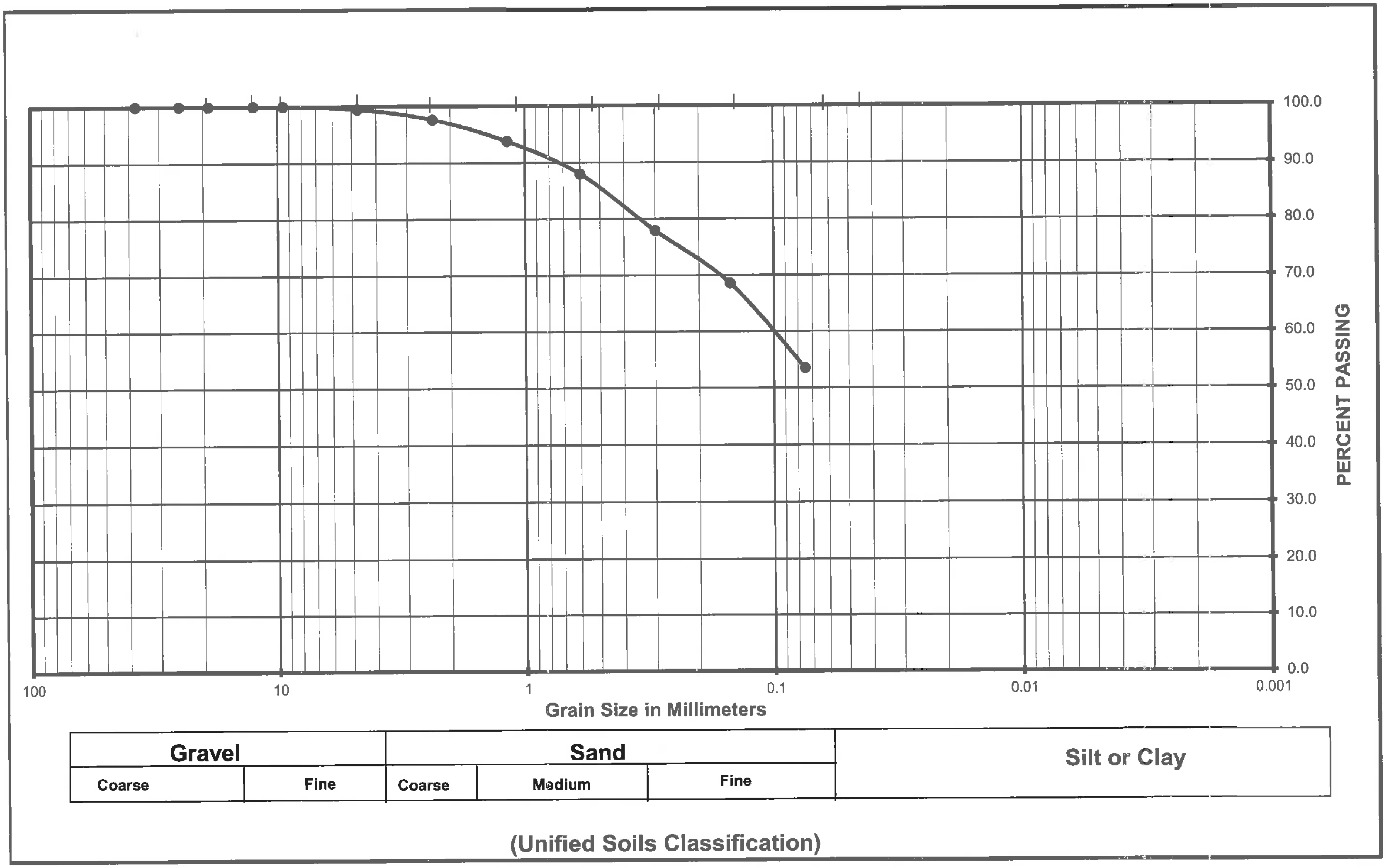
Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

(Unified Soils Classification)

Project Name	Mixed Use Development LA
Project Number	11217043
Soil Classification	SP
Sample Number	B-1 @ 20'

Krazan Testing Laboratory

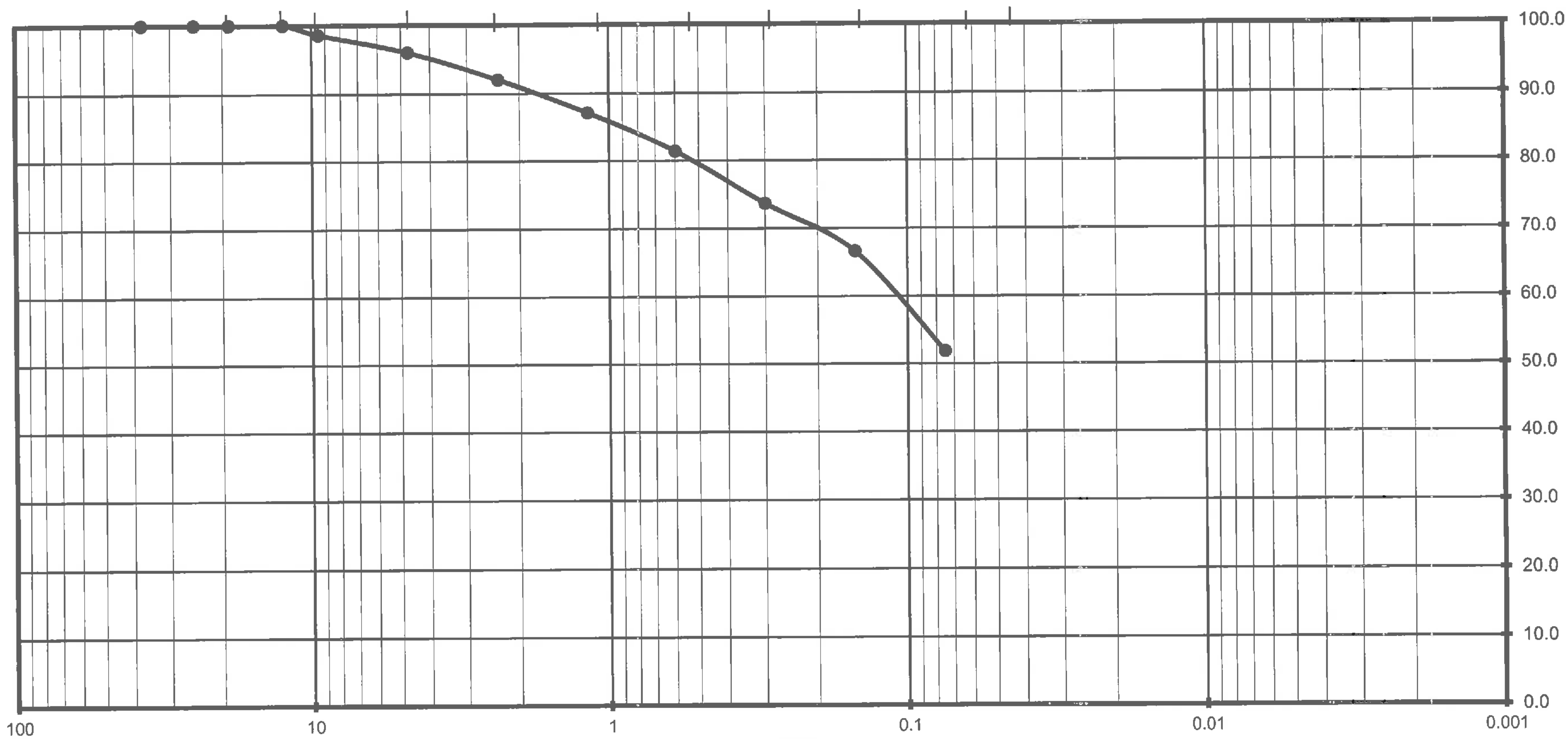
Grain Size Analysis



Project Name	Mixed Use Development LA
Project Number	11217043
Soil Classification	ML
Sample Number	B-1 @ 25'

Krazan Testing Laboratory

Grain Size Analysis



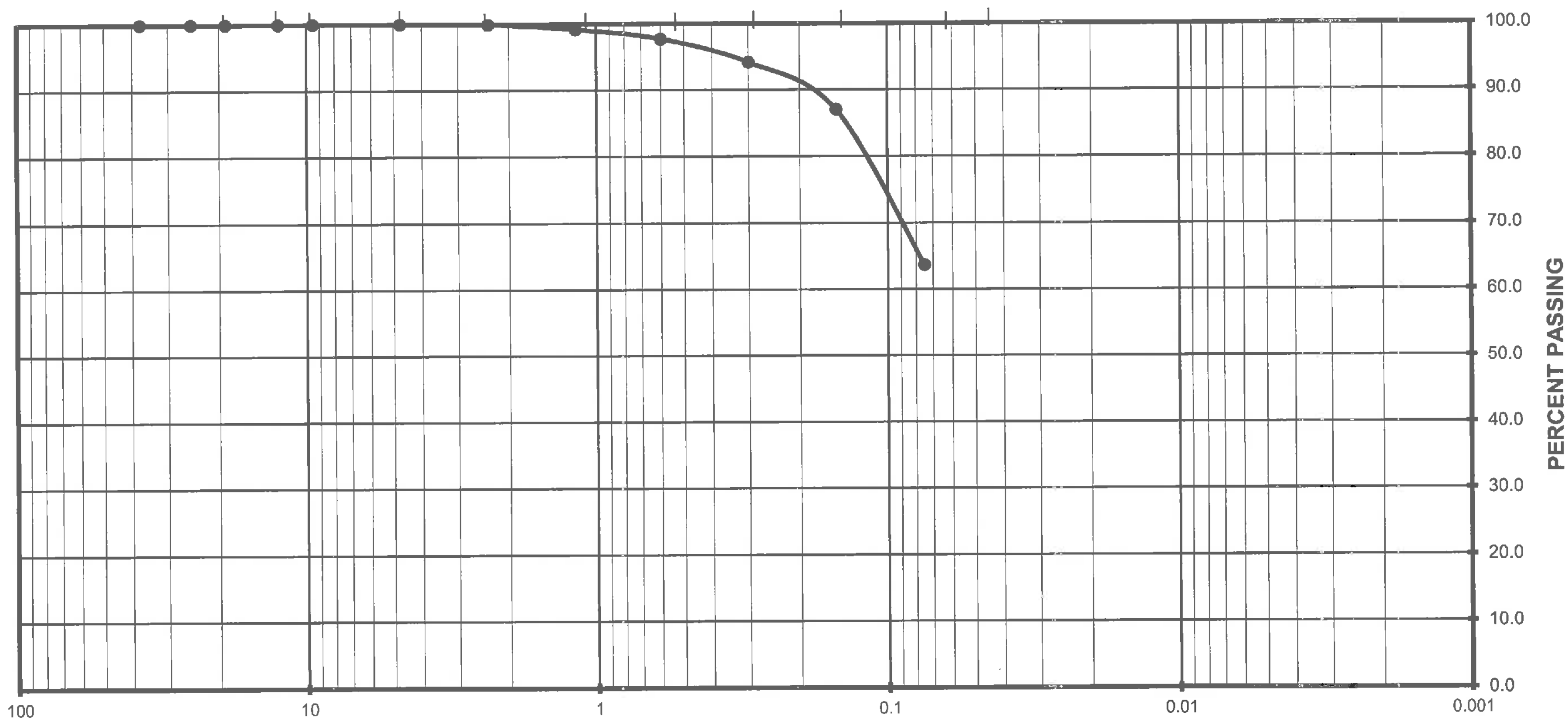
Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

(Unified Soils Classification)

Project Name	Mixed Use Development LA
Project Number	11217043
Soil Classification	CL
Sample Number	B-1 @ 30'

Krazan Testing Laboratory

Grain Size Analysis



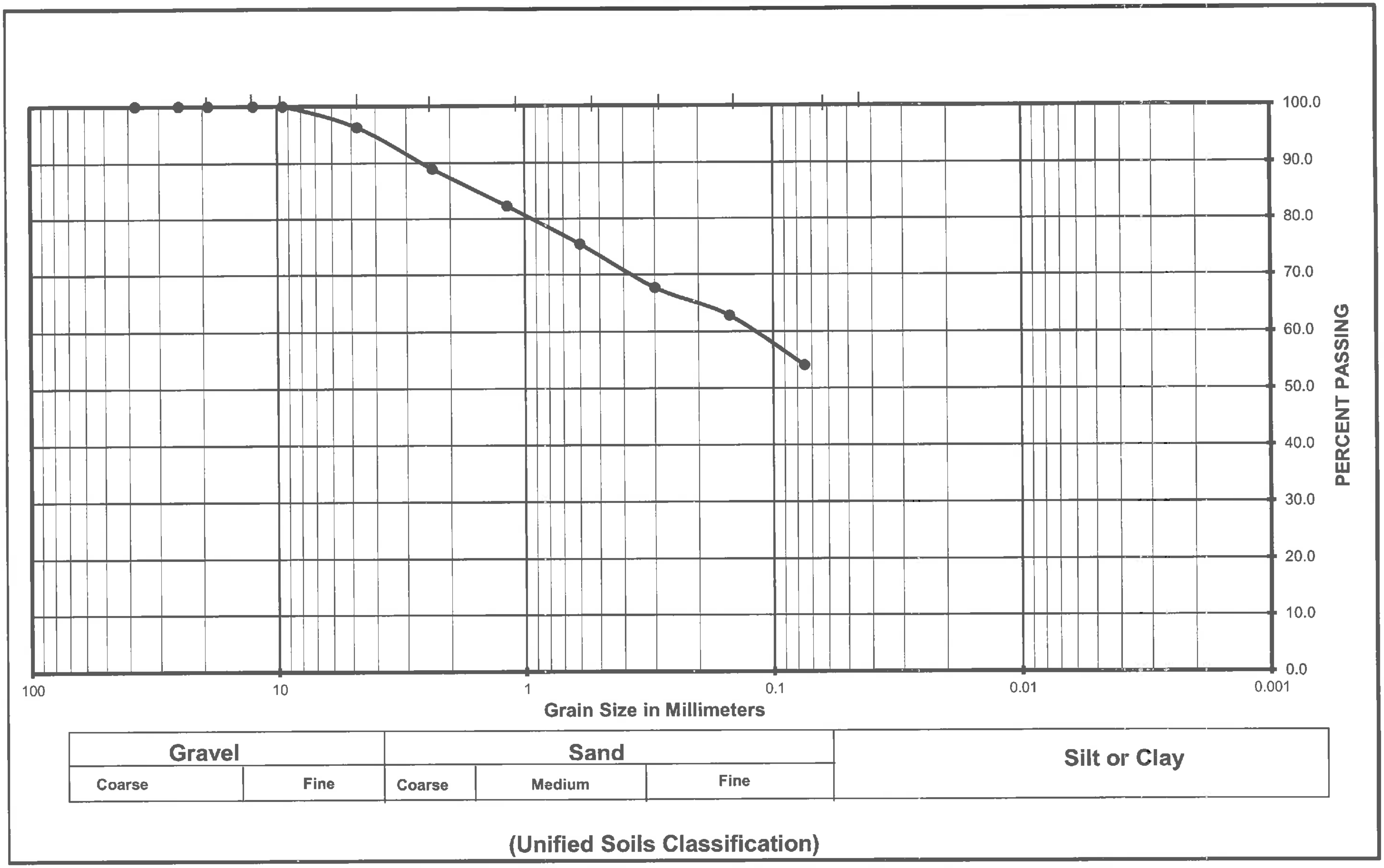
Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

(Unified Soils Classification)

Project Name	Mixed Use Development LA
Project Number	11217043
Soil Classification	CL
Sample Number	B-1 @ 35'

Krazan Testing Laboratory

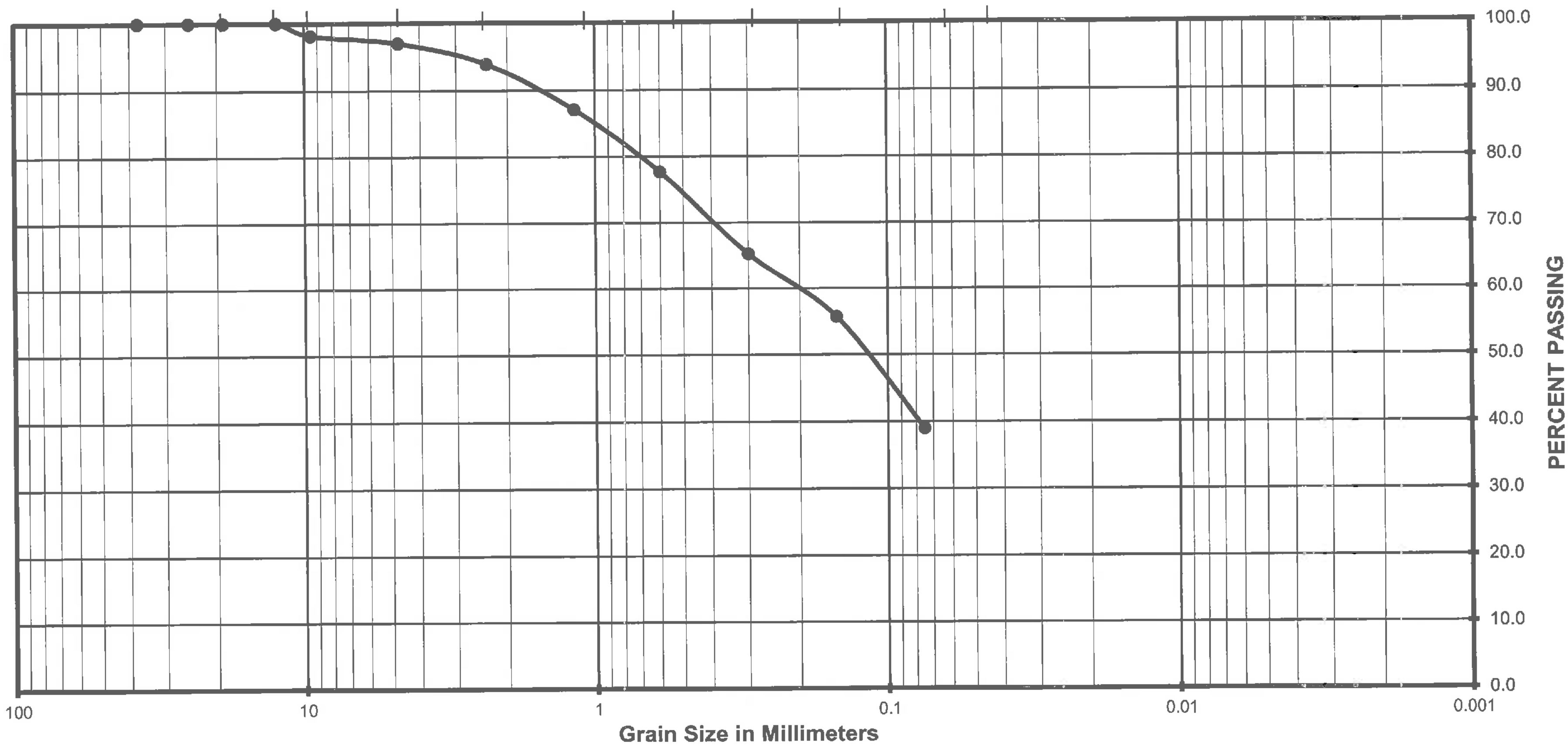
Grain Size Analysis



Project Name	Mixed Use Development LA
Project Number	11217043
Soil Classification	CL
Sample Number	B-1 @ 40'

Krazan Testing Laboratory

Grain Size Analysis



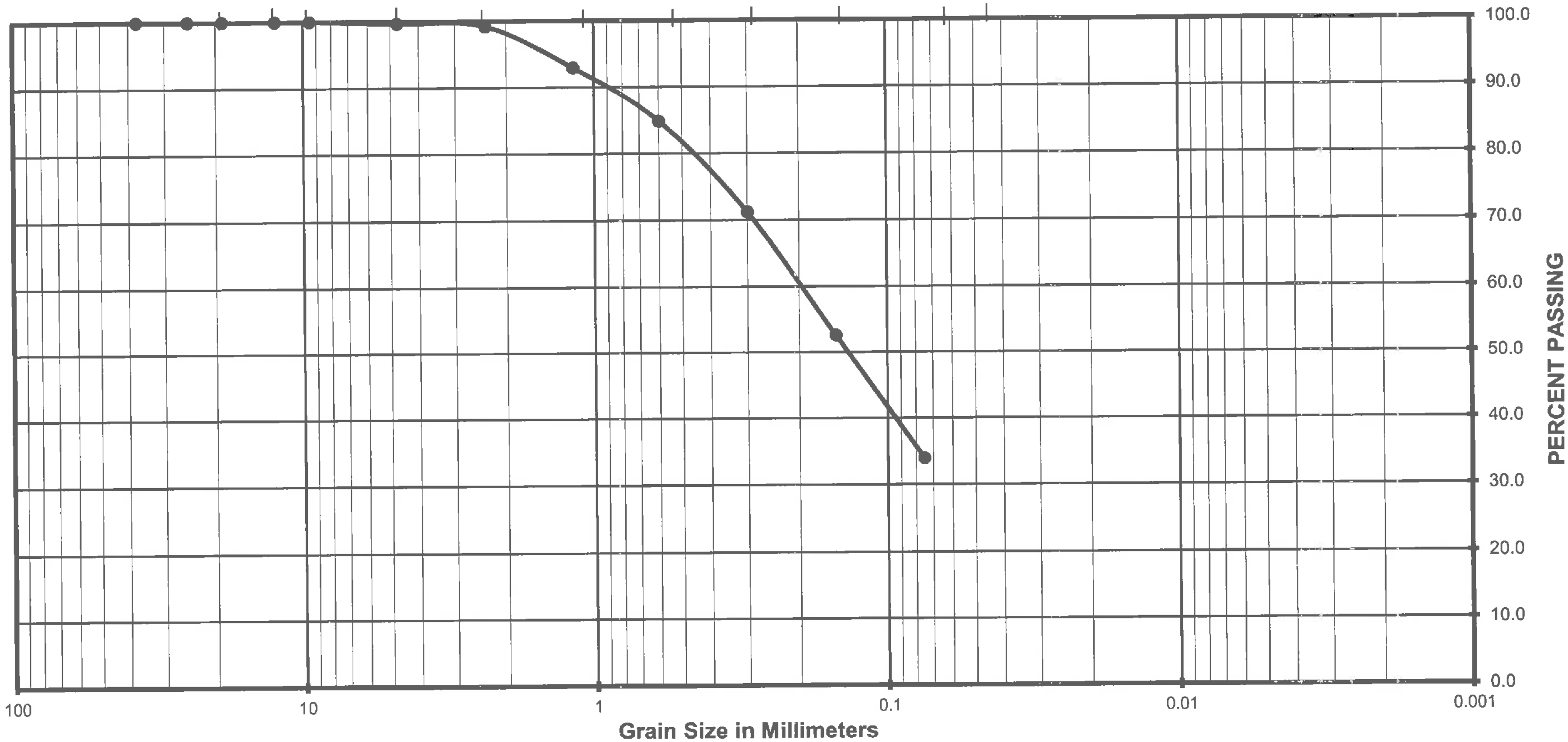
Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

(Unified Soils Classification)

Project Name	Mixed Use Development LA
Project Number	11217043
Soil Classification	SC
Sample Number	B-1 @ 45'

Krazan Testing Laboratory

Grain Size Analysis



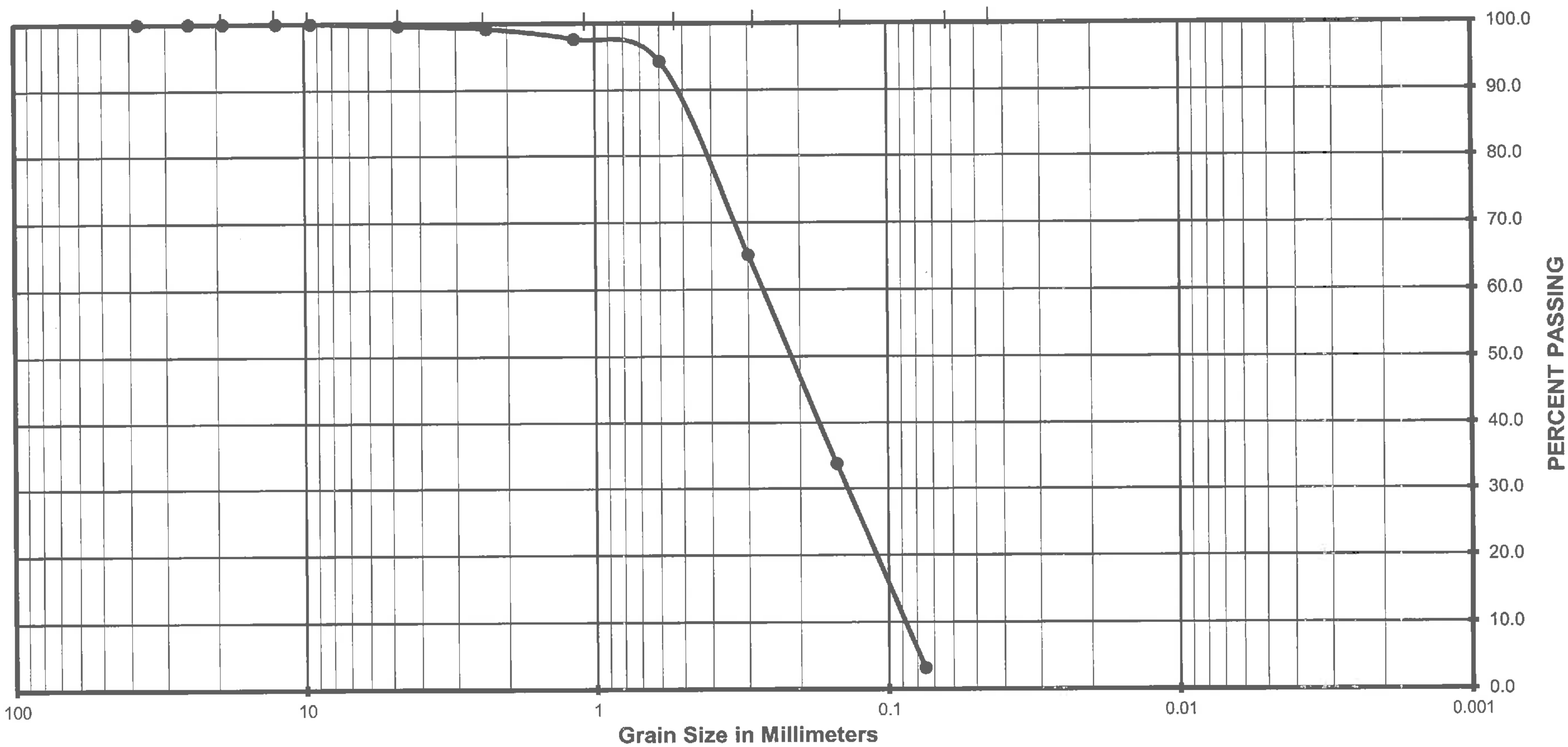
Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

(Unified Soils Classification)

Project Name	Mixed Use Development LA
Project Number	11217043
Soil Classification	SC
Sample Number	B-1 @ 50'

Krazan Testing Laboratory

Grain Size Analysis



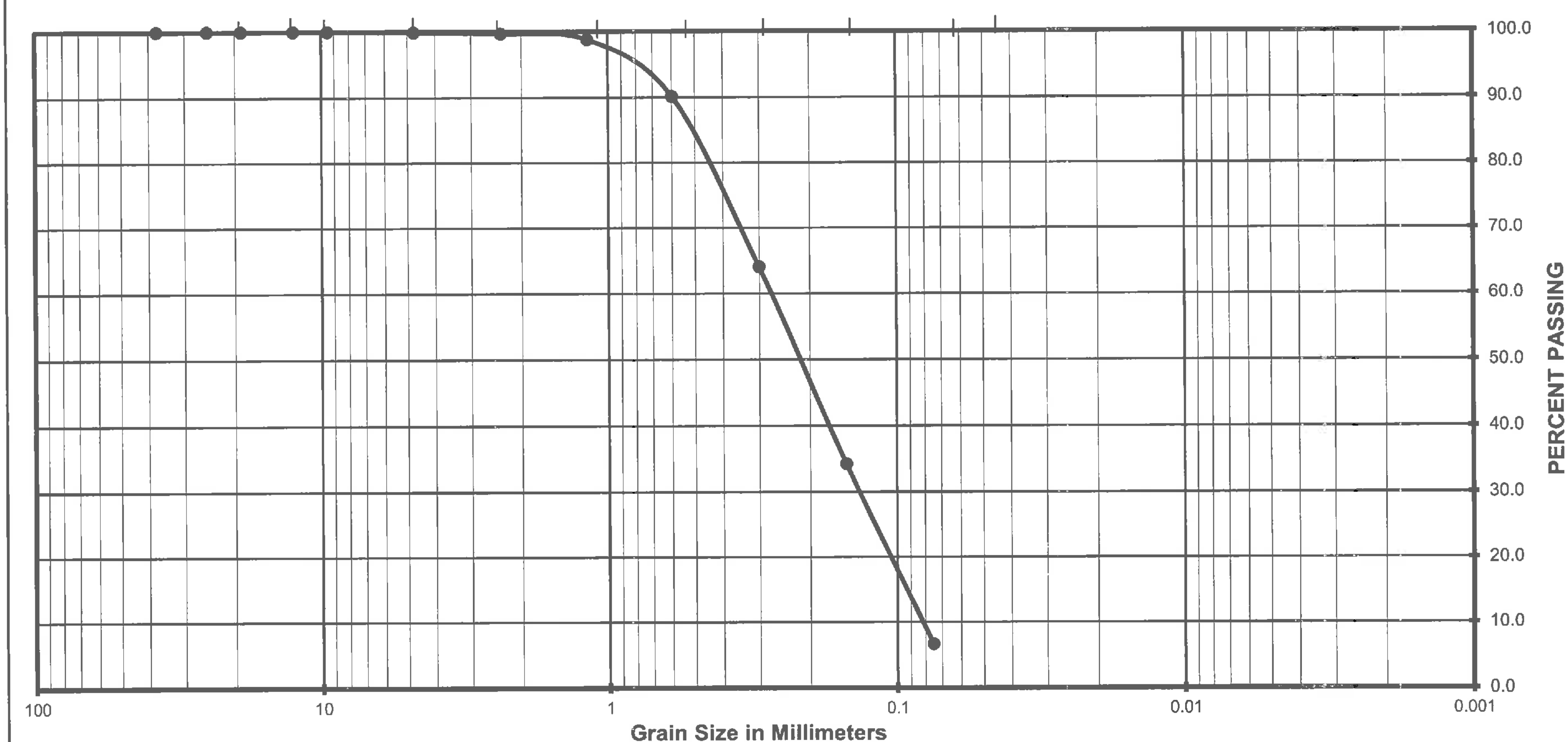
Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

(Unified Soils Classification)

Project Name	Mixed Use Development LA
Project Number	11217043
Soil Classification	SP
Sample Number	B-1 @ 55'

Krazan Testing Laboratory

Grain Size Analysis



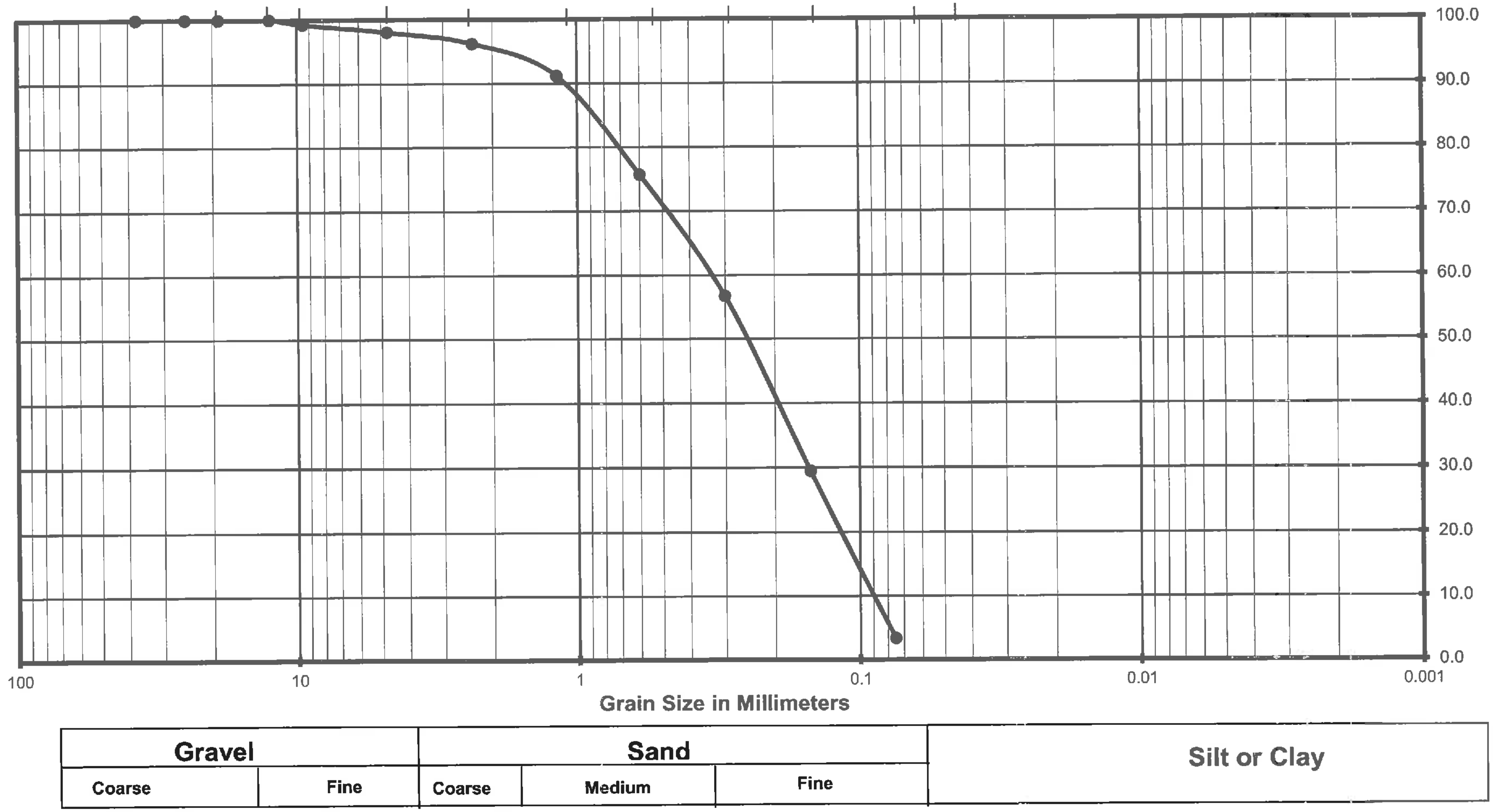
Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

(Unified Soils Classification)

Project Name	Mixed Use Development LA
Project Number	11217043
Soil Classification	SP
Sample Number	B-1 @ 60'

Krazan Testing Laboratory

Grain Size Analysis

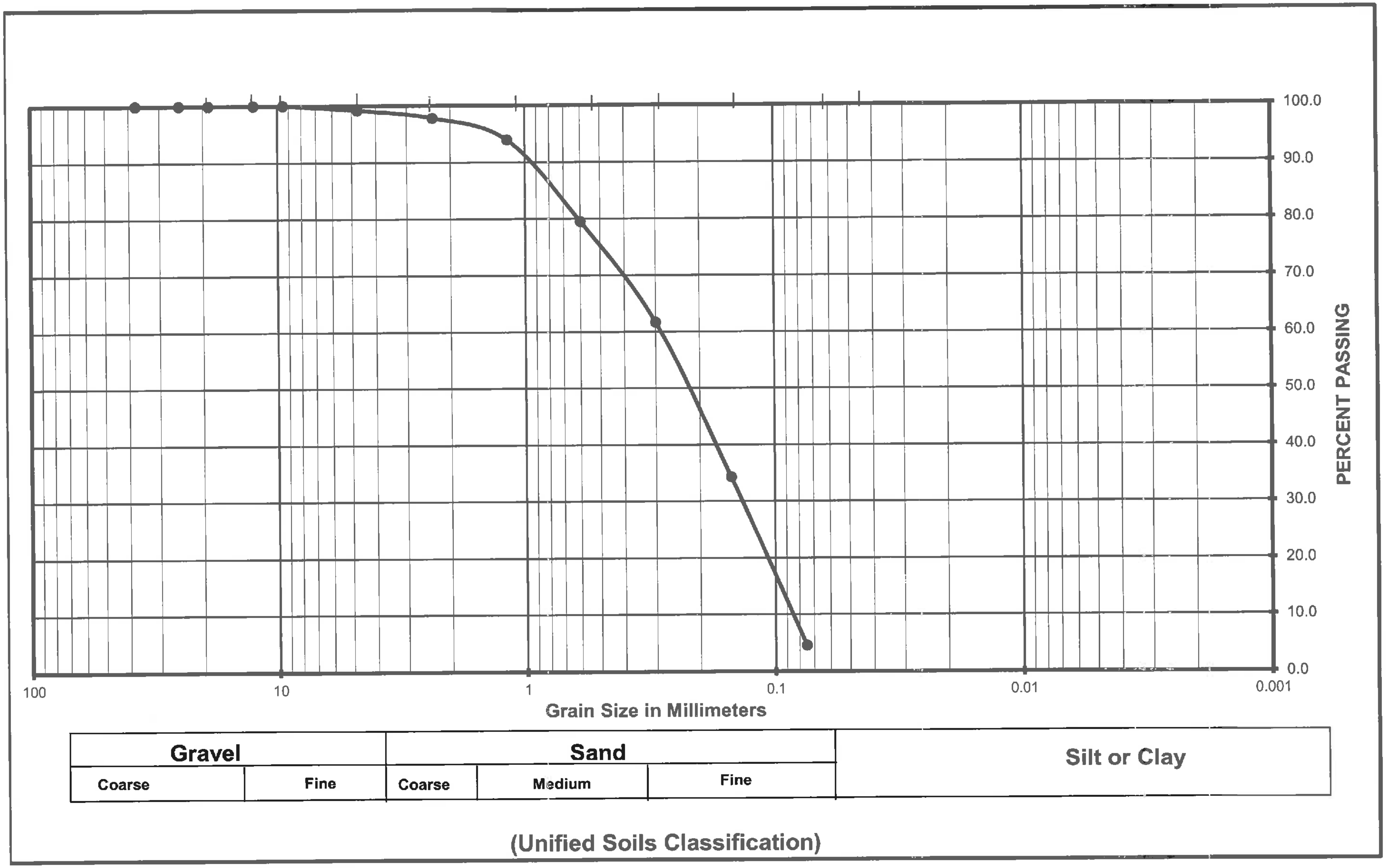


(Unified Soils Classification)

Project Name	Mixed Use Development LA
Project Number	11217043
Soil Classification	SP
Sample Number	B-1 @ 65'

Krazan Testing Laboratory

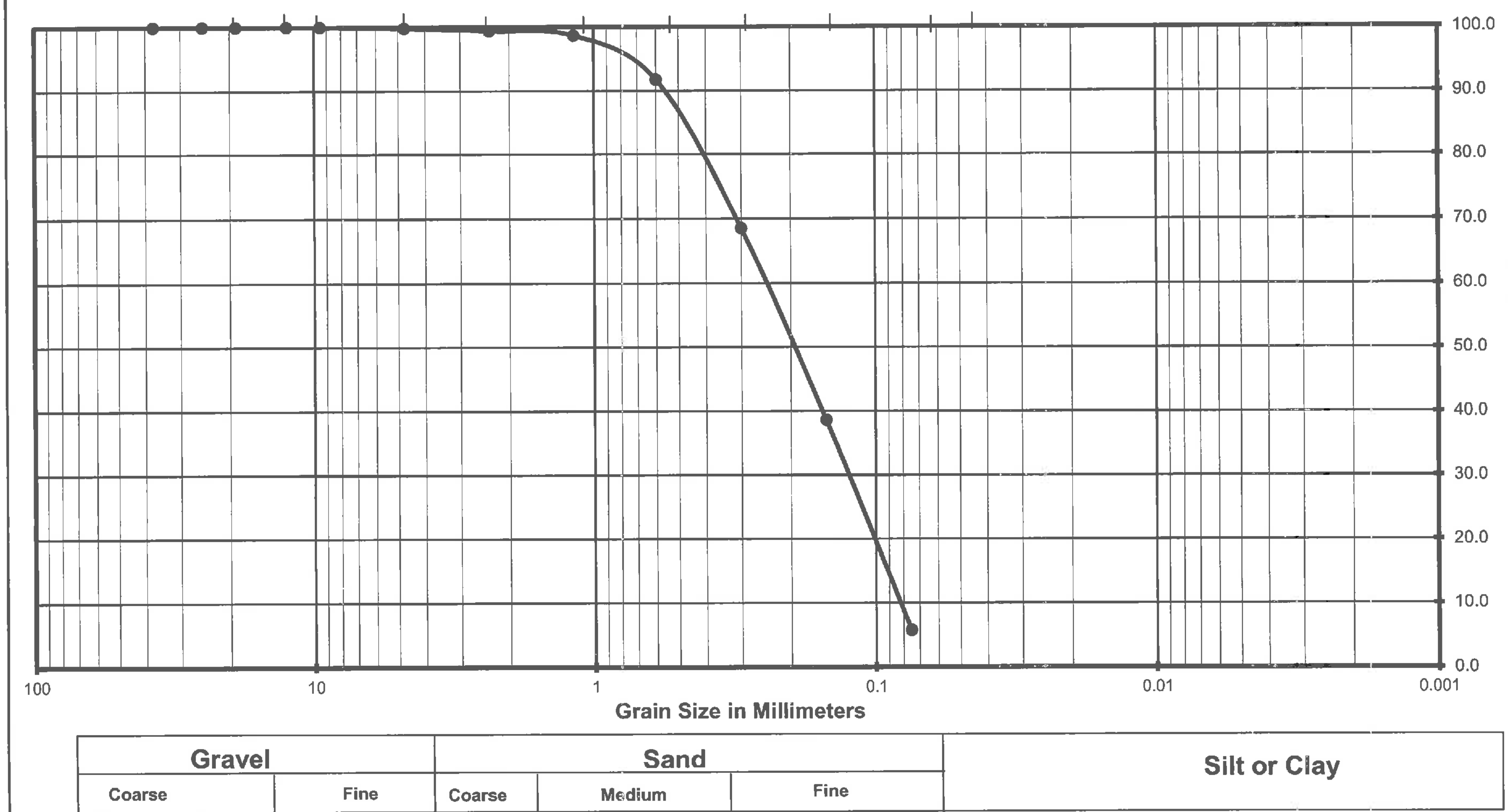
Grain Size Analysis



Project Name	Mixed Use Development LA
Project Number	11217043
Soil Classification	SP
Sample Number	B-1 @ 70'

Krazan Testing Laboratory

Grain Size Analysis



(Unified Soils Classification)

Project Name	Mixed Use Development LA
Project Number	11217043
Soil Classification	SP
Sample Number	B-1 @ 75'

Krazan Testing Laboratory

Grain Size Analysis

Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

(Unified Soils Classification)

Project Name	Mixed Use Development LA
Project Number	11217043
Soil Classification	SP
Sample Number	B-1 @ 80'

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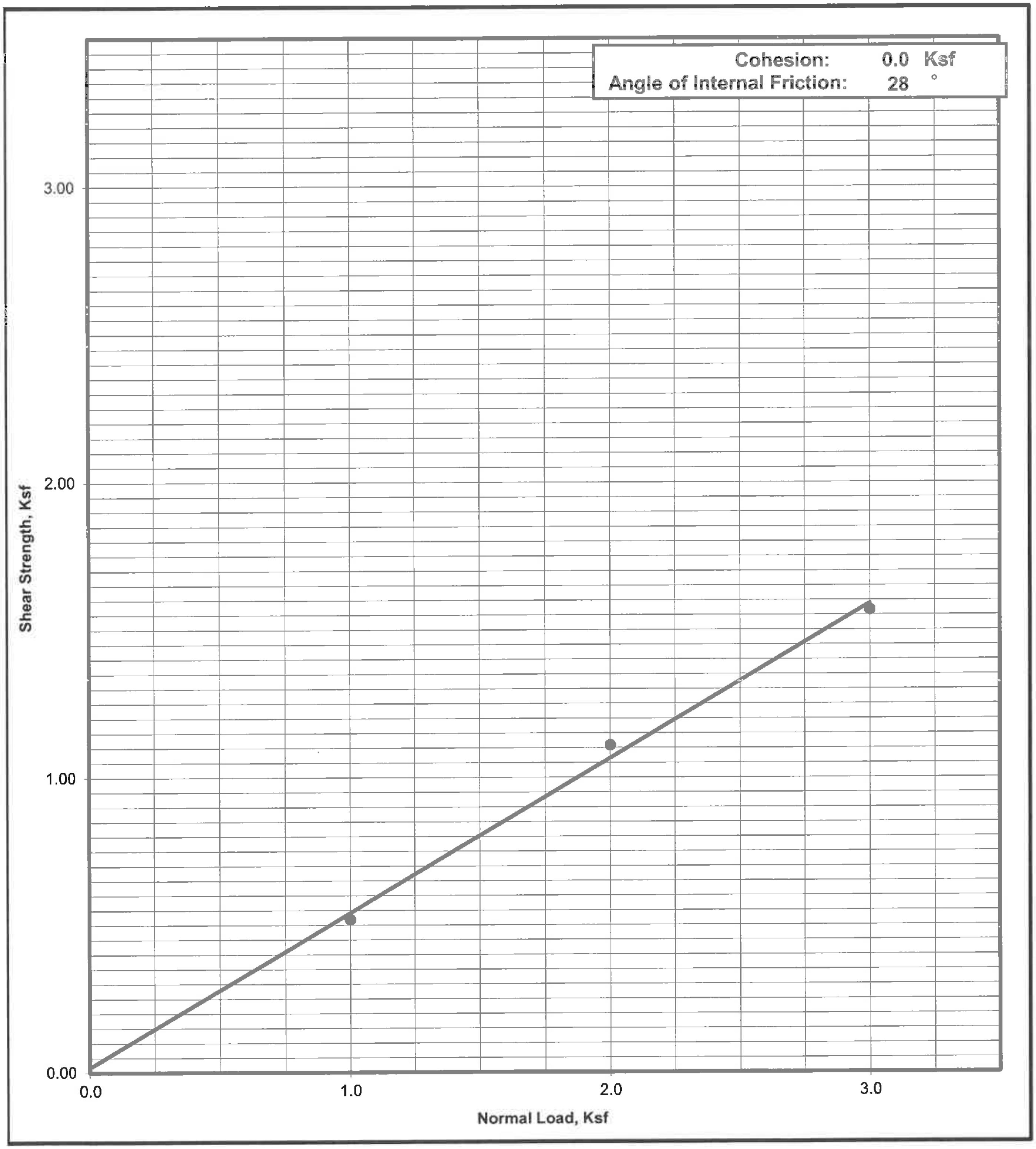
ASTM D - 3080 / AASHTO T - 236

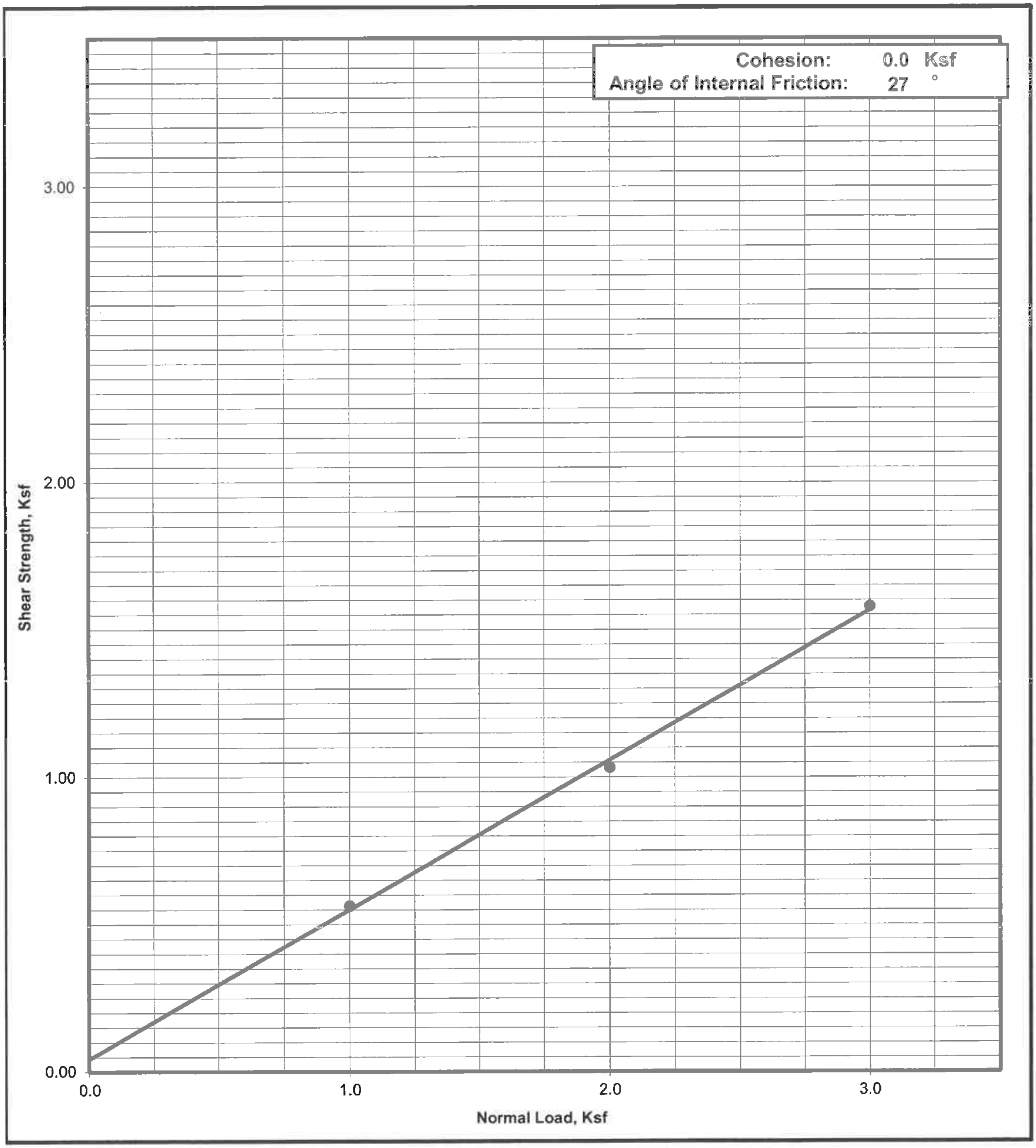
Project Number	Boring No. & Depth	Soil Type	Date
11217043	B-2 @ 5'	CL	6/26/2017



Krazan Testing Laboratory

ASTM D - 3080 / AASHTO T - 236

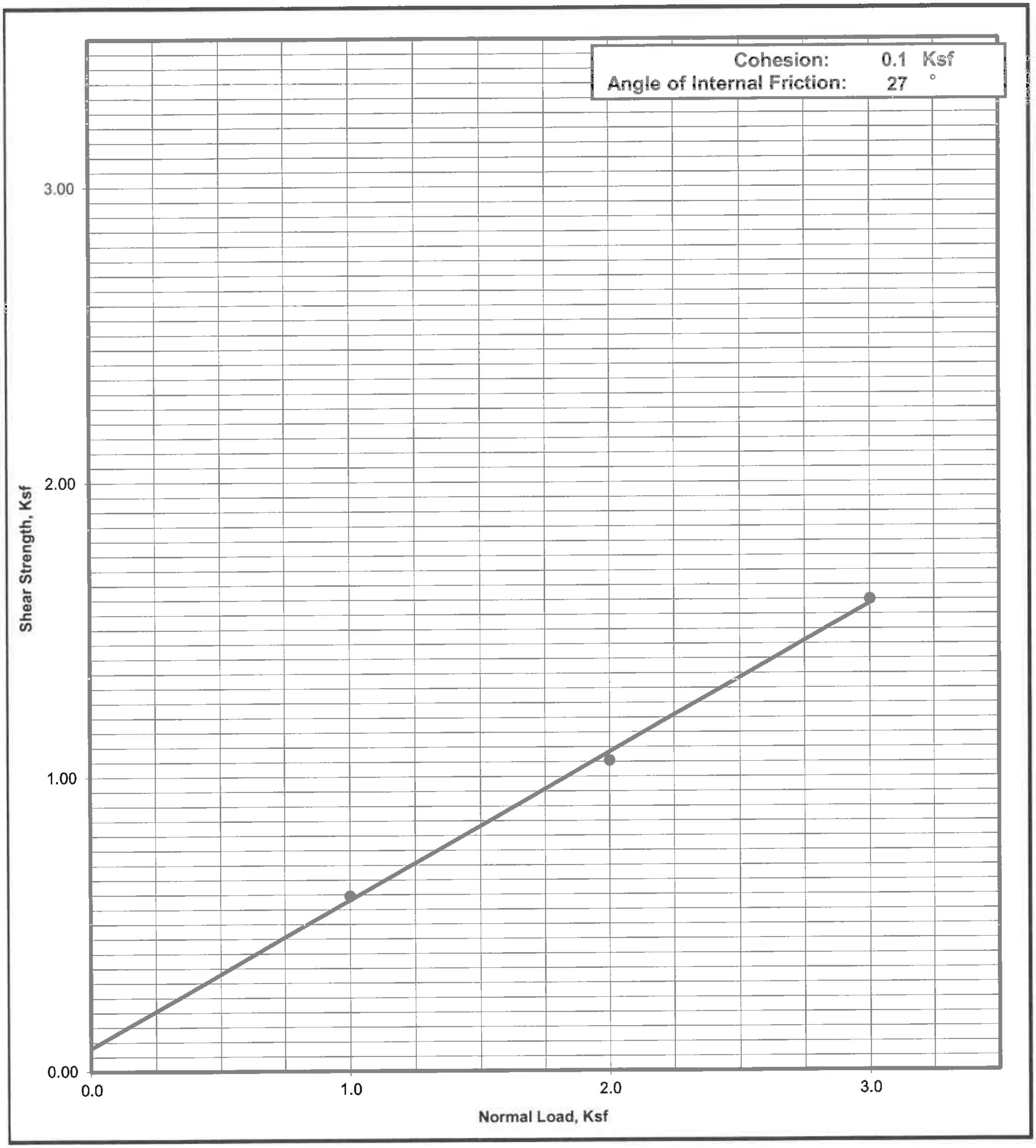
Project Number	Boring No. & Depth	Soil Type	Date
11217043	B-4 @ 5'	CL	6/26/2017



Krazan Testing Laboratory

ASTM D - 3080 / AASHTO T - 236

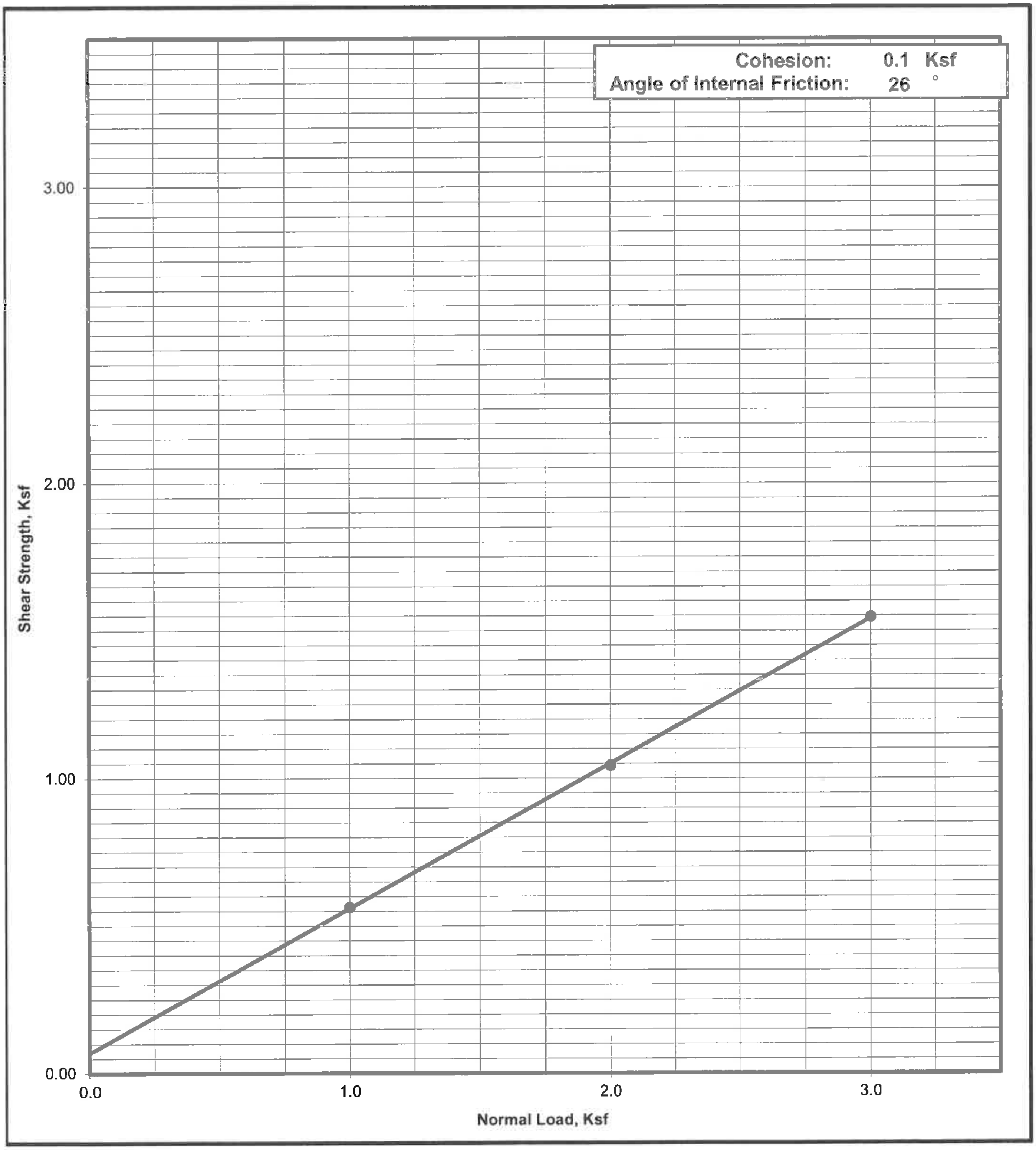
Project Number	Boring No. & Depth	Soil Type	Date
11217043	B-1 @ 30'	CL	6/27/2017



Krazan Testing Laboratory

ASTM D - 3080 / AASHTO T - 236

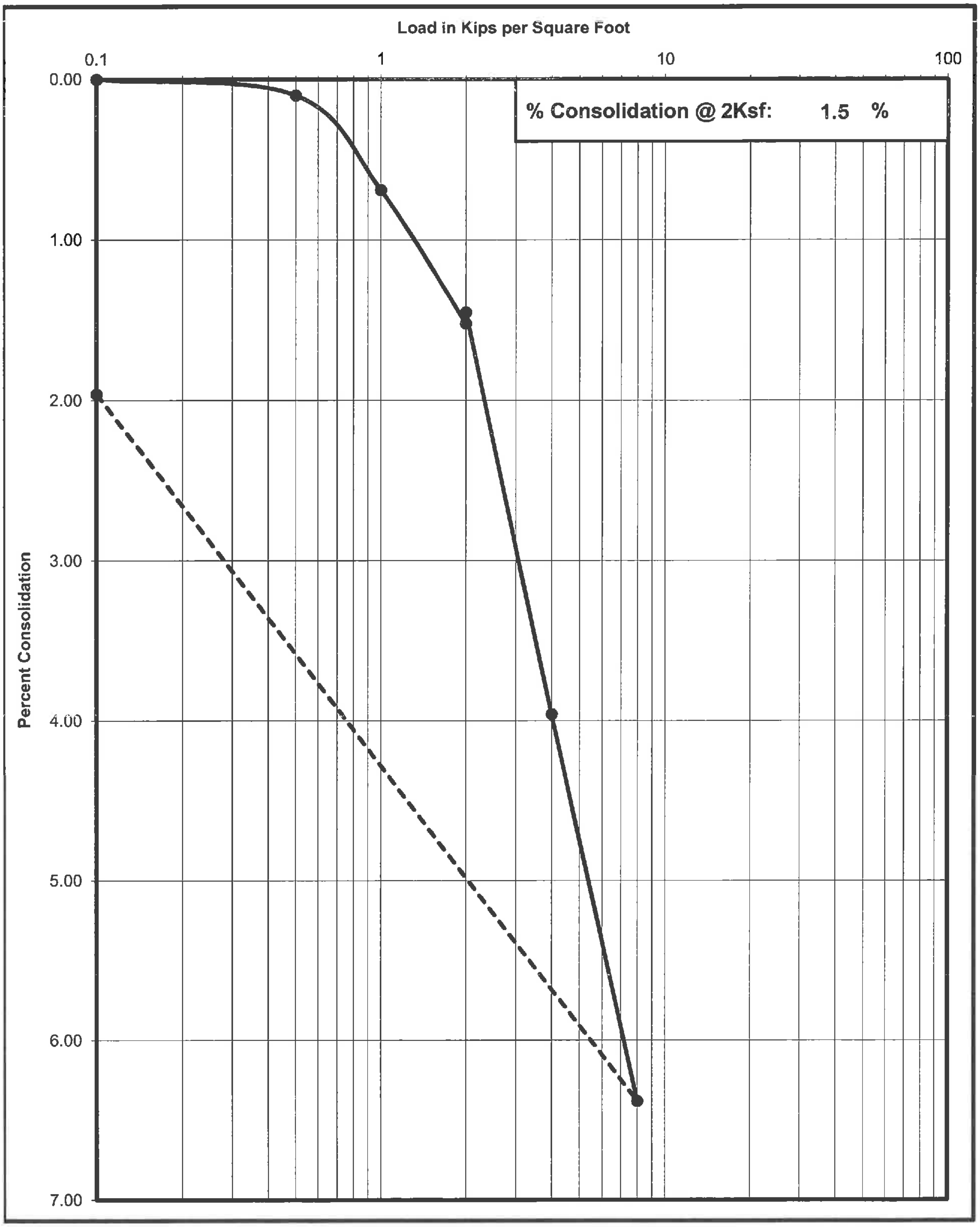
Project Number	Boring No. & Depth	Soil Type	Date
11217043	B-4 @ 30'	CL	6/27/2017



Krazan Testing Laboratory

Consolidation Test

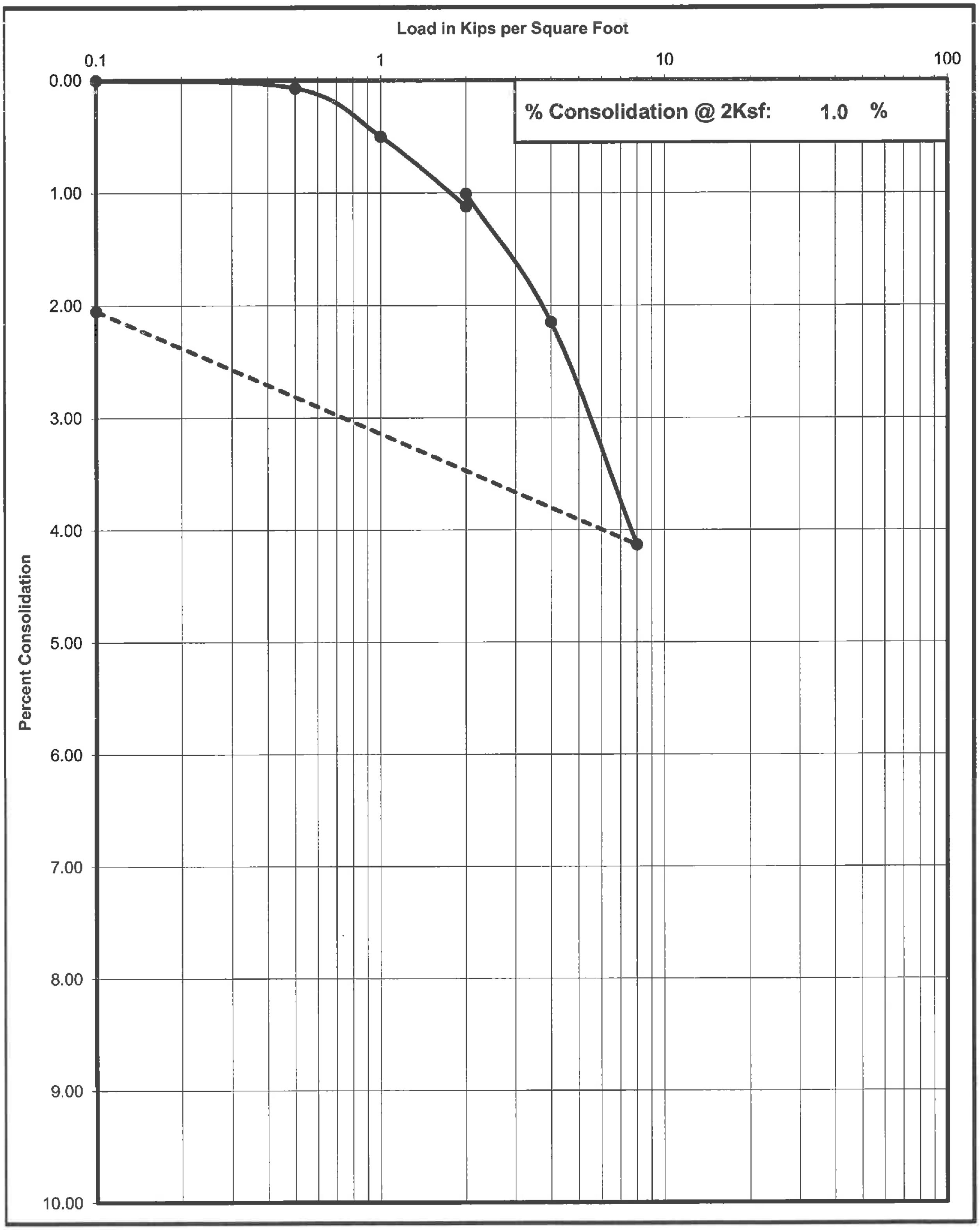
Project No	Boring No. & Depth	Date	Soil Classification
11217043	B-2 @ 5'	6/27/2017	CL



Krazan Testing Laboratory

Consolidation Test

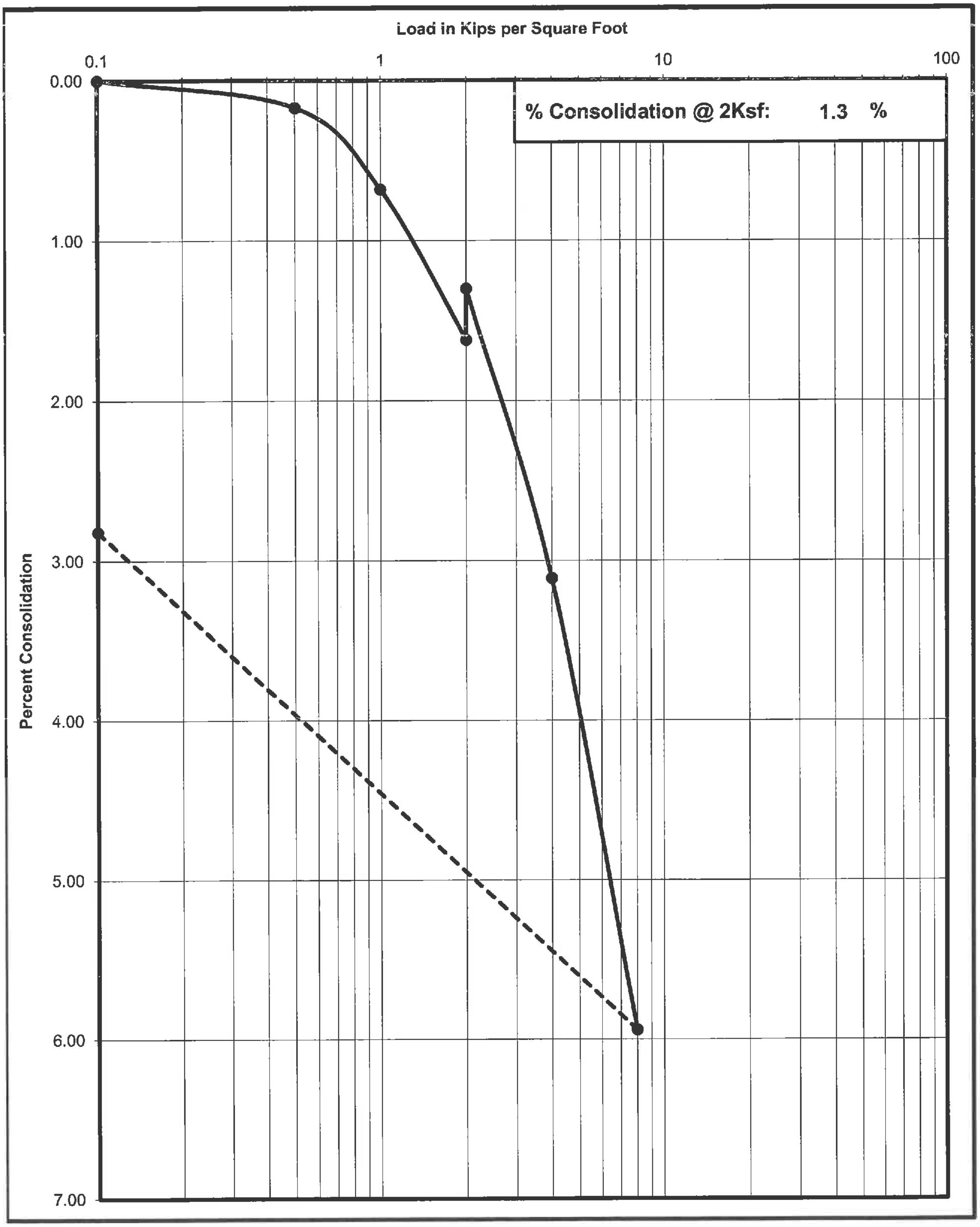
Project No	Boring No. & Depth	Date	Soil Classification
11217043	B-2 @ 10'	6/27/2017	CL



Krazan Testing Laboratory

Consolidation Test

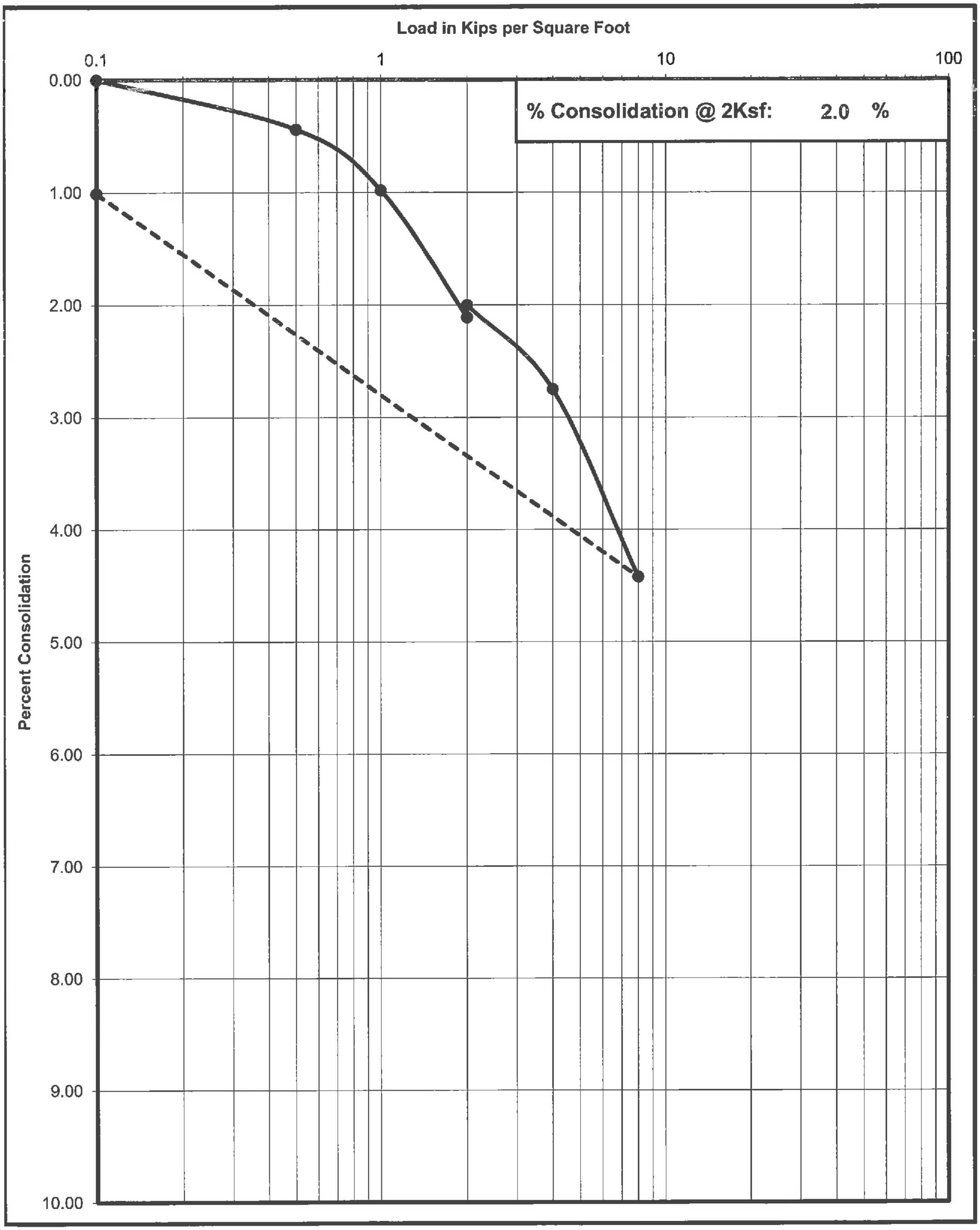
Project No	Boring No. & Depth	Date	Soil Classification
11217043	B-4 @ 5'	6/27/2017	CL



Krazan Testing Laboratory

Consolidation Test

Project No	Boring No. & Depth	Date	Soil Classification
11217043	B-4 @ 10'	6/27/2017	CL



Krazan Testing Laboratory

Expansion Index Test

ASTM D - 4829/ UBC Std. 18-2

Project Number	: 11217043
Project Name	: Mixed Use Development LA
Date	: 6/30/2017
Sample location/ Depth	: B-4 @ 0'-5'
Sample Number	: --
Soil Classification	: CL

Trial #	1	2	3
Weight of Soil & Mold, gms	545.8		
Weight of Mold, gms	172.4		
Weight of Soil, gms	373.4		
Wet Density, Lbs/cu.ft.	112.6		
Weight of Moisture Sample (Wet), gms	200.0		
Weight of Moisture Sample (Dry), gms	178.5		
Moisture Content, %	12.0		
Dry Density, Lbs/cu.ft.	100.5		
Specific Gravity of Soil	2.7		
Degree of Saturation, %	48.1		

Time	Inital	30 min	1 hr	6hrs	12 hrs	24 hrs
Dial Reading	0	--	--	--	--	0.059

Expansion Index $_{measured}$ = 59

Expansion Index $_{50}$ = 57.6

Expansion Index = 58

Expansion Potential Table	
Exp. Index	Potential Exp.
0 - 20	Very Low
21 - 50	Low
51 - 90	Medium
91 - 130	High
>130	Very High

Krazan Testing Laboratory

Expansion Index Test

ASTM D - 4829/ UBC Std. 18-2

Project Number	: 11217043
Project Name	: Mixed Use Development LA
Date	: 6/30/2017
Sample location/ Depth	: B-1 @ 0'-5'
Sample Number	: --
Soil Classification	: CL

Trial #	1	2	3
Weight of Soil & Mold, gms	554.0		
Weight of Mold, gms	171.0		
Weight of Soil, gms	383.0		
Wet Density, Lbs/cu.ft.	115.5		
Weight of Moisture Sample (Wet), gms	200.0		
Weight of Moisture Sample (Dry), gms	178.9		
Moisture Content, %	11.8		
Dry Density, Lbs/cu.ft.	103.3		
Specific Gravity of Soil	2.7		
Degree of Saturation, %	50.5		

Time	Inital	30 min	1 hr	6hrs	12 hrs	24 hrs
Dial Reading	0	--	--	--	--	0.055

Expansion Index $_{measured}$ = 55

Expansion Index $_{50}$ = 55.4

Expansion Index = 55

Expansion Potential Table	
Exp. Index	Potential Exp.
0 - 20	Very Low
21 - 50	Low
51 - 90	Medium
91 - 130	High
>130	Very High

Krazan Testing Laboratory

ANAHEIM TEST LAB, INC

3008 ORANGE AVENUE
SANTA ANA, CALIFORNIA 92707
PHONE (714) 549-7267

Krazan & Associates, Inc
1100 Olympic Drive, Ste. 103
Corona, CA 92881

DATE: 06/19/17

P.O. NO: Verbal

LAB NO: C-0676

SPECIFICATION: 417/422/643

MATERIAL: Soil

Project No: 11217043
Mixed Use Development
L.A.
B-1 @ 0-5'

ANALYTICAL REPORT

CORROSION SERIES
SUMMARY OF DATA

pH	SOLUBLE SULFATES per CA. 417 ppm	SOLUBLE CHLORIDES per CA. 422 ppm	MIN. RESISTIVITY per CA. 643 ohm-cm
6.9	160	139	3,100

RESPECTFULLY SUBMITTED

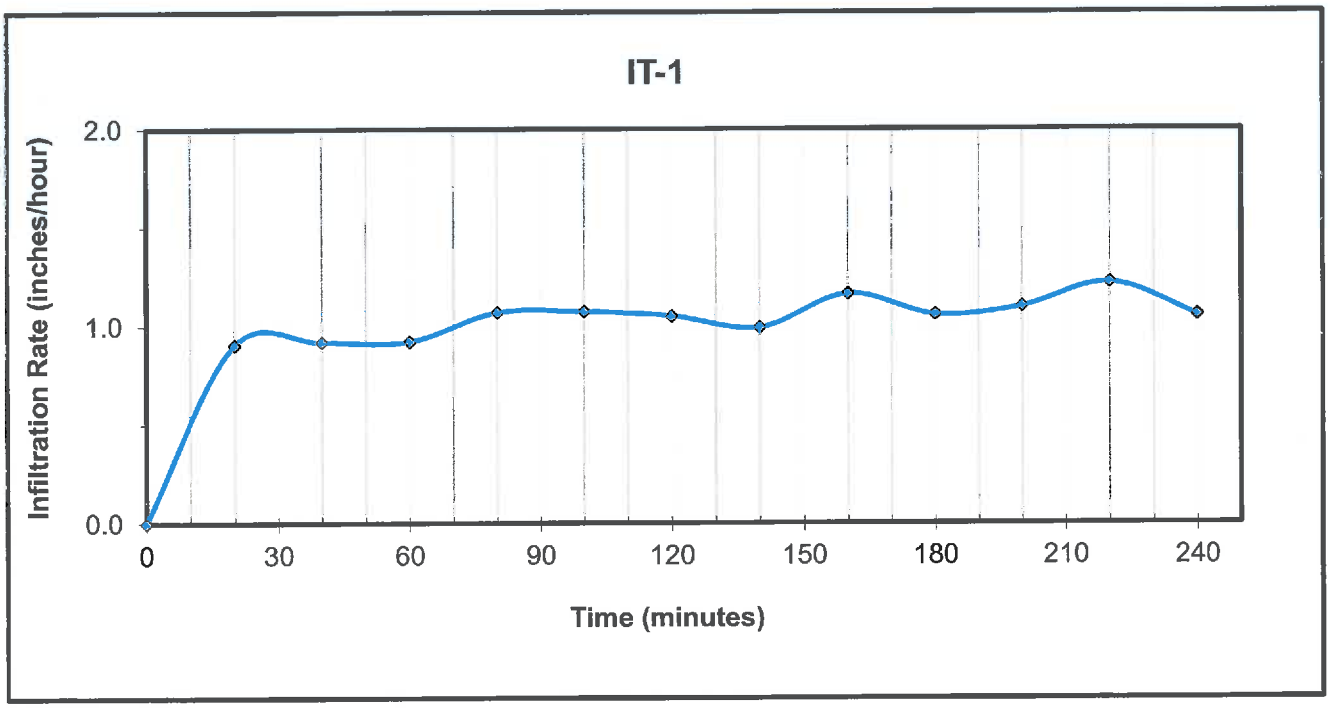
WES BRIDGER CHEMIST

RESULTS OF INFILTRATION TESTS - REVERSE BOREHOLE

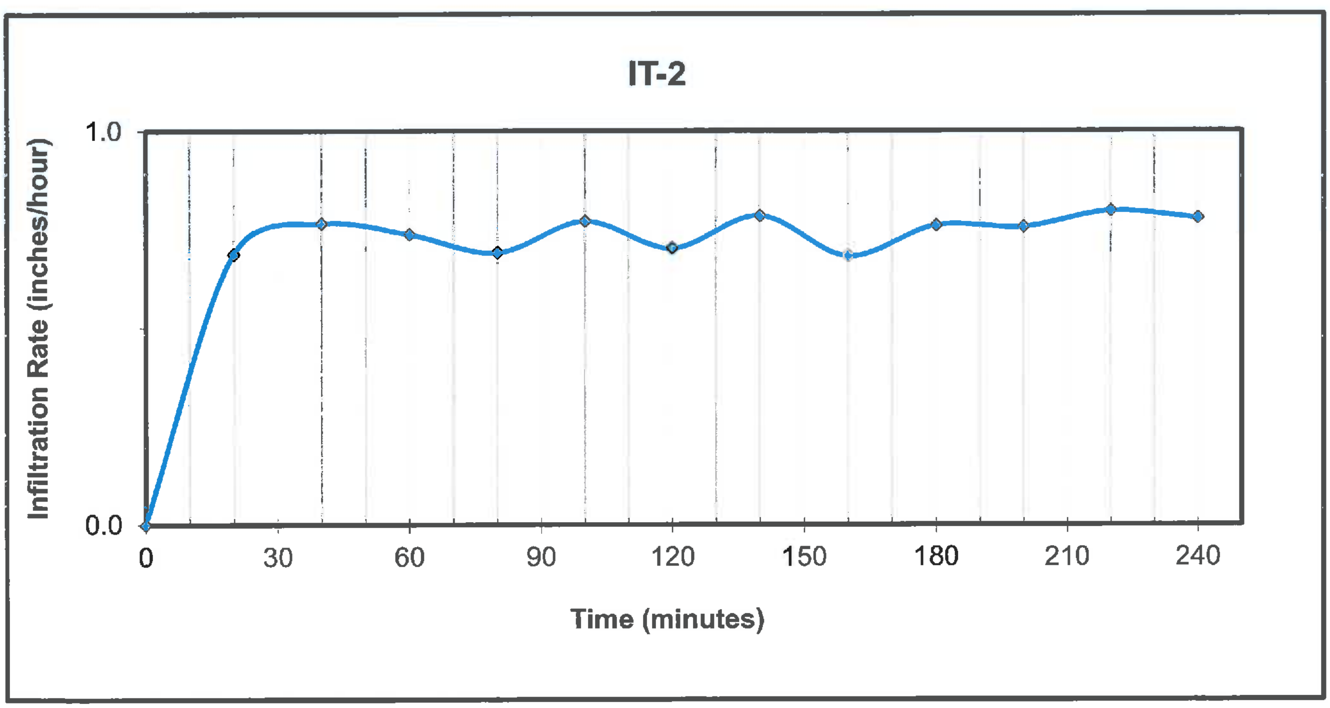
Project #	**11217043**	**Date**	**6/8/2017**
Project Name	Mixed Use Development Los Angeles		
Project Address	Los Angeles, California		

Test No:	IT-1	**Total Depth (in.)**	48	**Test Size (in)**	9
Depth To Water	20'	**Soil Classification**	CL		

Reading	Elasped Time(min.)	Incremental Time (min.)	Initial Depth To Water(in.)	Final Depth To Water(in.)	Incremental Fall of Water(in.)	Incremental Infiltration Rate (in/hr)
Start	0	0.00		5.5	--	--
1	20.00	20.00	5.5	10.5	5.00	0.91
2	40.00	20.00	10.5	15.0	4.50	0.92
3	60.00	20.00	15.0	19.0	4.00	0.92
4	80.00	20.00	19.0	23.0	4.00	1.07
5	100.00	20.00	23.0	26.5	3.50	1.07
6	120.00	20.00	26.5	29.5	3.00	1.05
7	140.00	20.00	29.5	32.0	2.50	0.99
8	160.00	20.00	32.0	34.5	2.50	1.16
9	180.00	20.00	34.5	36.5	2.00	1.06
10	200.00	20.00	36.5	38.3	1.80	1.10
11	220.00	20.00	38.3	40.0	1.70	1.22
12	240.00	20.00	40.0	41.3	1.30	1.06
	Infiltration Rate in Inches per Hour					**0.91**



RESULTS OF INFILTRATION TESTS - REVERSE BOREHOLE						
Project #	11217043				**Date**	6/8/2017
Project Name	Mixed Use Development Los Angeles					
Project Address	Los Angeles, California					
Test No:	IT-2	**Total Depth (in.)**		48	**Test Size (in)**	9
Depth To Water	20'	**Soil Classification**		CL		
Reading	**Elasped Time(min.)**	**Incremental Time (min.)**	**Initial Depth To Water(in.)**	**Final Depth To Water(in.)**	**Incremental Fall of Water(in.)**	**Incremental Infiltration Rate (in/hr)**
Start	0	0.00		5.0	--	--
1	20.00	20.00	5.0	9.0	4.00	0.69
2	40.00	20.00	9.0	13.0	4.00	0.77
3	60.00	20.00	13.0	16.5	3.50	0.74
4	80.00	20.00	16.5	19.5	3.00	0.69
5	100.00	20.00	19.5	22.5	3.00	0.77
6	120.00	20.00	22.5	25.0	2.50	0.70
7	140.00	20.00	25.0	27.5	2.50	0.78
8	160.00	20.00	27.5	29.5	2.00	0.68
9	180.00	20.00	29.5	31.5	2.00	0.76
10	200.00	20.00	31.5	33.3	1.80	0.76
11	220.00	20.00	33.3	35.0	1.70	0.80
12	240.00	20.00	35.0	36.5	1.50	0.78
	Infiltration Rate in Inches per Hour					**0.68**



General Earthwork Specifications

Appendix B

APPENDIX B

EARTHWORK SPECIFICATIONS

GENERAL

When the text of the report conflicts with the general specifications in this appendix, the recommendations in the report have precedence.

SCOPE OF WORK: These specifications and applicable plans pertain to and include all earthwork associated with the site rough grading, including but not limited to the furnishing of all labor, tools, and equipment necessary for site clearing and grubbing, stripping, preparation of foundation materials for receiving fill, excavation, processing, placement and compaction of fill and backfill materials to the lines and grades shown on the project grading plans, and disposal of excess materials.

PERFORMANCE: The Contractor shall be responsible for the satisfactory completion of all earthwork in accordance with the project plans and specifications. This work shall be inspected and tested by a representative of Krazan and Associates, Inc., hereinafter known as the Soils Engineer and/or Testing Agency. Attainment of design grades when achieved shall be certified by the project Civil Engineer. Both the Soils Engineer and the Civil Engineer are the Owner's representatives. If the Contractor should fail to meet the technical or design requirements embodied in this document and on the applicable plans, he shall make the necessary readjustments until all work is deemed satisfactory as determined by both the Soils Engineer and the Civil Engineer. No deviation from these specifications shall be made except upon written approval of the Soils Engineer, Civil Engineer or project Architect.

No earthwork shall be performed without the physical presence or approval of the Soils Engineer. The Contractor shall notify the Soils Engineer at least 2 working days prior to the commencement of any aspect of the site earthwork.

The Contractor agrees that he shall assume sole and complete responsibility for job site conditions during the course of construction of this project, including safety of all persons and property; that this requirement shall apply continuously and not be limited to normal working hours; and that the Contractor shall defend, indemnify and hold the Owner and the Engineers harmless from any and all liability, real or alleged, in connection with the performance of work on this project, except for liability arising from the sole negligence of the Owner or the Engineers.

TECHNICAL REQUIREMENTS: All compacted materials shall be densified to a density not less than 90 percent relative compaction based on ASTM Test Method D1557 or CAL-216, as specified in the technical portion of the Soil Engineer's report. The location and frequency of field density tests shall be as determined by the Soils Engineer. The results of these tests and compliance with these specifications shall be the basis upon which satisfactory completion of work will be judged by the Soils Engineer.

SOILS AND FOUNDATION CONDITIONS: The Contractor is presumed to have visited the site and to have familiarized himself with existing site conditions and the contents of the data presented in the soil report.

The Contractor shall make his own interpretation of the data contained in said report, and the Contractor shall not be relieved of liability under the Contract documents for any loss sustained as a result of any variance between conditions indicated by or deduced from said report and the actual conditions encountered during the progress of the work.

DUST CONTROL: The work includes dust control as required for the alleviation or prevention of any dust nuisance on or about the site or the borrow area, or off-site if caused by the Contractor's operation either during the performance of the earthwork or resulting from the conditions in which the Contractor leaves the site. The Contractor shall assume all liability, including court costs of codefendants, for all claims related to dust or windblown materials attributable to his work.

SITE PREPARATION

Site preparation shall consist of site clearing and grubbing and the preparations of foundation materials for receiving fill.

CLEARING AND GRUBBING: The Contractor shall accept the site in this present condition and shall demolish and/or remove from the area of designated project earthwork all structures, both surface and subsurface, trees, brush, roots, debris, organic matter, and all other matter determined by the Soils Engineer to be deleterious or otherwise unsuitable. Such materials shall become the property of the Contractor and shall be removed from the site.

Tree root systems in proposed building areas should be removed to a minimum depth of 3 feet and to such an extent which would permit removal of all roots larger than 1 inch. Tree roots removed in parking areas may be limited to the upper 1½ feet of the ground surface. Backfill of tree root excavations should not be permitted until all exposed surfaces have been inspected and the Soils Engineer is present for the proper control of backfill placement and compaction. Burning in areas which are to receive fill materials shall not be permitted.

SUBGRADE PREPARATION: Surfaces to receive Engineered Fill, building or slab loads shall be prepared as outlined above, excavated/scarified to a depth of 12 inches, moisture-conditioned as necessary, and compacted to 90 percent relative compaction.

Loose soil areas, areas of uncertified fill, and/or areas of disturbed soils shall be moisture-conditioned as necessary and recompacted to 90 percent relative compaction. All ruts, hummocks, or other uneven surface features shall be removed by surface grading prior to placement of any fill materials. All areas which are to receive fill materials shall be approved by the Soils Engineer prior to the placement of any of the fill material.

EXCAVATION: All excavation shall be accomplished to the tolerance normally defined by the Civil Engineer as shown on the project grading plans. All over-excavation below the grades specified shall be backfilled at the Contractor's expense and shall be compacted in accordance with the applicable technical requirements.

FILL AND BACKFILL MATERIAL: No material shall be moved or compacted without the presence of the Soils Engineer. Material from the required site excavation may be utilized for construction site fills provided prior approval is given by the Soils Engineer. All materials utilized for constructing site fills shall be free from vegetation or other deleterious matter as determined by the Soils Engineer.

PLACEMENT, SPREADING AND COMPACTION: The placement and spreading of approved fill materials and the processing and compaction of approved fill and native materials shall be the responsibility of the Contractor. However, compaction of fill materials by flooding, ponding, or jetting shall not be permitted unless specifically approved by local code, as well as the Soils Engineer.

Both cut and fill areas shall be surface-compacted to the satisfaction of the Soils Engineer prior to final acceptance.

SEASONAL LIMITS: No fill material shall be placed, spread, or rolled while it is frozen or thawing or during unfavorable wet weather conditions. When the work is interrupted by heavy rains, fill operations shall not be resumed until the Soils Engineer indicates that the moisture content and density of previously placed fill are as specified.

General Paving Specifications

Appendix C

APPENDIX C

PAVEMENT SPECIFICATIONS

1. **DEFINITIONS** - The term "pavement" shall include asphaltic concrete surfacing, untreated aggregate base, and aggregate subbase. The term "subgrade" is that portion of the area on which surfacing, base, or subbase is to be placed.

The term "Standard Specifications": hereinafter referred to is the May 2006 Standard Specifications of the State of California, Department of Transportation, and the "Materials Manual" is the Materials Manual of Testing and Control Procedures, State of California, Department of Public Works, Division of Highways. The term "relative compaction" refers to the field density expressed as a percentage of the maximum laboratory density as defined in the applicable tests outlined in the Materials Manual.

2. **SCOPE OF WORK** - This portion of the work shall include all labor, materials, tools, and equipment necessary for, and reasonably incidental to the completion of the pavement shown on the plans and as herein specified, except work specifically noted as "Work Not Included."

3. **PREPARATION OF THE SUBGRADE** - The Contractor shall prepare the surface of the various subgrades receiving subsequent pavement courses to the lines, grades, and dimensions given on the plans. The upper 12 inches of the soil subgrade beneath the pavement section shall be compacted to a minimum relative compaction of 90 percent. The finished subgrades shall be tested and approved by the Soils Engineer prior to the placement of additional pavement courses.

4. **UNTREATED AGGREGATE BASE** - The aggregate base material shall be spread and compacted on the prepared subgrade in conformity with the lines, grades, and dimensions shown on the plans. The aggregate base material shall conform to the requirements of Section 26 of the Standard Specifications for Class II material, 1½ inches maximum size. The aggregate base material shall be spread and compacted in accordance with Section 26 of the Standard Specifications. The aggregate base material shall be spread in layers not exceeding 6 inches and each layer of aggregate material course shall be tested and approved by the Soils Engineer prior to the placement of successive layers. The aggregate base material shall be compacted to a minimum relative compaction of 95 percent.

5. **AGGREGATE SUBBASE** - The aggregate subbase shall be spread and compacted on the prepared subgrade in conformity with the lines, grades, and dimensions shown on the plans. The aggregate subbase material shall conform to the requirements of Section 25 of the Standard Specifications for Class II material. The aggregate subbase material shall be compacted to a minimum relative compaction of 95 percent, and it shall be spread and compacted in accordance with Section 25 of the Standard Specifications. Each layer of aggregate subbase shall be tested and approved by the Soils Engineer prior to the placement of successive layers.

6. ASPHALTIC CONCRETE SURFACING - Asphaltic concrete surfacing shall consist of a mixture of mineral aggregate and paving grade asphalt, mixed at a central mixing plant and spread and compacted on a prepared base in conformity with the lines, grades and dimensions shown on the plans. The viscosity grade of the asphalt shall be PG 64-10. The mineral aggregate shall be Type B, ½ inch maximum size, medium grading and shall conform to the requirements set forth in Section 39 of the Standard Specifications. The drying, proportioning and mixing of the materials shall conform to Section 39.

The prime coat, spreading and compacting equipment and spreading and compacting mixture shall conform to the applicable chapters of Section 39, with the exception that no surface course shall be placed when the atmospheric temperature is below 50° F. The surfacing shall be rolled with a combination of steel wheel and pneumatic rollers, as described in Section 39-6. The surface course shall be placed with an approved self-propelled mechanical spreading and finishing machine.

7. FOG SEAL COAT - The fog seal (mixing type asphaltic emulsion) shall conform to and be applied in accordance with the requirements of Section 37.