APPENDIX D: GEOTECHNICAL INVESTIGATION

D.1: Geocon West, Inc.,
Geotechnical Investigation, Proposed Hotel Development,
2053 East 7th Street, Los Angeles, California,
August 1, 2018.

D.2: Department of Building and Safety,
Soils Report Approval Letter, 2053 E. 7th Street,
August 16, 2018.

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GEOTECHNICAL INVESTIGATION

PROPOSED HOTEL DEVELOPMENT
2053 EAST 7TH STREET
LOS ANGELES, CALIFORNIA
TRACT: WINGERTER, LOT: 213

GEOCON
WEST, INC.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

1711 LINCOLN LLC
LOS ANGELES, CALIFORNIA

PROJECT NO. A9815-06-01

AUGUST 1, 2018

Project No. A9815-06-01
August 1, 2018

Mr. Roberto Vasquez
1711 Lincoln LLC
1880 Century Park East, Suite 200
Los Angeles, California 90067

Subject: GEOTECHNICAL INVESTIGATION
PROPOSED HOTEL DEVELOPMENT
2053 EAST 7TH STREET, LOS ANGELES, CALIFORNIA
TRACT: WINGERTER, LOT: 213

Dear Mr. Vasquez:

In accordance with your authorization of our proposal dated May 23, 2018, we have performed a geotechnical investigation for the proposed hotel development located at 2053 East 7th Street 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.

PROFESSIONAL GEOLOGIST SUSAN FRANZEN KIRKGARD No. 1754 CERTIFIED ENGINEERING GEOLOGIST STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER JELISA THOMAS ADAMS GE3092 GEOTECHNICAL STATE OF CALIFORNIA

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(EMAIL) Addressee

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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a geotechnical investigation for the proposed hotel development located at 2053 East 7th Street 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 a site reconnaissance, field exploration, laboratory testing, engineering analysis, and the preparation of this report. The site was explored on June 22, 2018, by excavating two 8-inch diameter borings to depths of approximately 41 and 45½ 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 2053 East 7th Street in the City of Los Angeles, California. The property is an irregularly shaped parcel that is currently occupied by an asphalt paved parking area and a three-story masonry structure that is underlain by a basement level. The single level basement encompasses a partial area of the building footprint and is located within the north to northeastern portion of the structure. This property is bounded by a residential structure underlain by two subterranean levels (currently under construction) to the north and west, by East 7th Street to the south, and by South Santa Fe Avenue to the east. The site is roughly level with no significant highs or lows. Surface water drainage at the site appears to be by sheet flow along the existing ground contours towards the city streets. The site is fully developed and there is no vegetation onsite.

It is our understanding that the proposed development will consists of a hotel structure with up to 13-stories and underlain by one subterranean level. Preliminary plans depicting the proposed development, as well as the existing site conditions are depicted on the Site Plan (see Figure 2).

Based on the preliminary nature of the design at this time, wall and column loads were not available. It is anticipated that column loads for the proposed structure may be up to 1,500 kips, and wall loads may be up to 18 kips per linear foot.

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

3. GEOLOGIC SETTING

The site is located in the northwestern portion of the Los Angeles Basin, approximately 500 feet west of the Los Angeles River. The basin is 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 7.2 miles to the southwest.

4. SOIL AND GEOLOGIC CONDITIONS

Based on our field investigation and published geologic maps of the area, the site is underlain by artificial fill and Holocene age alluvium consisting of varying amounts of sand, silt, gravel, cobbles and boulders originating from the nearby Los Angeles River (California Geological Survey, 2012; Lamar, 1970). Detailed stratigraphic profiles are provided on the boring logs in Appendix A.

4.1 Artificial Fill

Artificial fill was encountered in our field explorations to a maximum depth of 4 feet below the existing ground surface. The artificial fill generally consists of dark brown silty sand with some fine gravel. The artificial fill is characterized as slightly moist and 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.

4.2 Alluvium

Holocene age younger alluvial fan deposits were encountered beneath the fill and consist primarily of poorly and well-graded sand and silty sand with varying amounts of fine to coarse gravel and some cobbles below a depth of 22 feet. The soil is primarily light brown to brown, slightly moist, and loose to very dense and generally becomes denser with increasing depth. The site is located within the ancestral flood plain of the Los Angeles River and, although gravel and cobbles were only locally encountered in our borings, zones of cobbles and boulders may be encountered during construction.

5. GROUNDWATER

Based on a review of the Seismic Hazard Zone Report for the Los Angeles Quadrangle (California Division of Mines and Geology [CDMG], 1998), the historically highest groundwater level in the area is greater than 150 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.

Groundwater was not encountered in our field explorations which were drilled to maximum depth of 45½ feet below the existing ground surface. Considering the historic high groundwater level, the lack of groundwater in our borings, and the depth of the proposed structure, it is unlikely that groundwater will be encountered during construction. However, 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. Recommendations for drainage are provided in the S*urface Drainage* section of this report (see Section 7.29).

6. GEOLOGIC HAZARDS

6.1 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, 2017) 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 3, Regional Fault Map.

The closest surface trace of an active fault to the site is the Raymond Fault located approximately 5.7 miles to the north (Ziony and Jones, 1989). Other nearby active faults include the Hollywood Fault, the Newport-Inglewood Fault Zone, and the Whittier Fault located approximately 5.9 miles north, 7.2 miles southwest, and 10.4 miles southeast of the site, respectively (Ziony and Jones, 1989). The active San Andreas Fault Zone is located approximately 35 miles northeast of the site.

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. The Los Angeles Segment of the Puente Hills Blind Thrust underlies the site at depth. However, this thrust fault and others in the Los Angeles area are not exposed at the surface and do not present a potential surface fault rupture hazard at the site. The deep thrust faults in the Los Angeles Basin, although not considered a potential surface fault rupture hazard, are considered active features capable of generating future earthquakes that could result in moderate to significant ground shaking at the site.

6.2 Seismicity

As with all of Southern California, the site has experienced historic earthquakes from various regional faults. The seismicity of the region surrounding the site was formulated based on research of an electronic database of earthquake data. The epicenters of recorded earthquakes with magnitudes equal to or greater than 5.0 in the site vicinity are depicted on Figure 4, Regional Seismicity Map. A partial list of moderate to major magnitude earthquakes that have occurred in the Southern California area within the last 100 years is included in the 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	73	ESE
Near Redlands	July 23, 1923	6.3	56	E
Long Beach	March 10, 1933	6.4	33	SE
Tehachapi	July 21, 1952	7.5	80	NW
San Fernando	February 9, 1971	6.6	28	NNW
Whittier Narrows	October 1, 1987	5.9	9	E
Sierra Madre	June 28, 1991	5.8	20	NE
Landers	June 28, 1992	7.3	103	E
Big Bear	June 28, 1992	6.4	81	E
Northridge	January 17, 1994	6.7	21	NW
Hector Mine	October 16, 1999	7.1	118	ENE

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

6.3 Seismic Design Criteria

The following table summarizes 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	2.324g	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}	2.324g	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}	1.550g	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	0.873g	Figure 22-7
Site Coefficient, F_{PGA}	1.0	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.873g	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 6.66 magnitude event occurring at a hypocentral distance of 7.18 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 6.65 magnitude occurring at a hypocentral distance of 11.8 kilometers from the site.

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

6.4 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 Los Angeles Quadrangle (CDMG, 1999; CGS, 2017) 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 Seismic Safety Element (Leighton, 1990) indicates that the site is not located within an area identified as having a potential for liquefaction. As previously discussed, the historic high groundwater level beneath the site is at a depth greater than 150 feet beneath the existing ground surface. Based on these considerations, it is our opinion that the potential for liquefaction and associated ground deformations beneath the site is very low.

6.5 Slope Stability

The topography at the site and in the site vicinity is relatively level and slopes gently to the south-southwest. The site is not located within a City of Los Angeles Hillside Grading Area or Hillside Ordinance Area (City of Los Angeles, 2018). According to the County of Los Angeles Safety Element (Leighton, 1990), the site is not within an area identified as a "Hillside Area" or an area identified as 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, 2017). 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.

6.6 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 Hansen Dam and Sepulveda Dam inundation areas. However, these reservoirs, 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.

6.7 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. Flooding 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).

6.8 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 not located within the limits of an oilfield (DOGGR, 2018). There are four plugged oil and gas wells within ½-mile of the site. These include the Chalmers-Santa Fe LLC SFRR Unit Wells No. 1 and No. 1, the Phillips Petroleum Company Signal-Standard-Exley Well No.1 and the Atlantic Richfield Company L.A River Community Well No. 1-1. All of these wells are plugged oil and gas exploration wells, the closest of which is located approximately 750 feet northwest of the site (DOGGR, 2018). Due to the voluntary nature of record reporting by the oil well drilling companies, oil/gas wells may be improperly located or not shown on the location map and undocumented wells could be encountered during construction. Any wells encountered during construction will need to be properly abandoned in accordance with the current requirements of the DOGGR.

The site is not located within the boundaries of a city-designated Methane Zone or Methane Buffer Zone (City of Los Angeles, 2018). Since the site is not located within the boundaries of a known oil field, the potential for the presence of methane or other volatile gases to occur at the site is considered low. However, should it be determined that a methane study is required for the proposed development it is recommended that a qualified methane consultant be retained to perform the study and provide mitigation measures as necessary.

6.9 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 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. There appears to be little or no potential for ground subsidence due to withdrawal of fluids or gases at the site.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

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

7.1.2 Up to 4 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, such as below the basement level of the existing structure. Future demolition of the existing structures which occupy the site will likely disturb the upper few feet of existing site soils. It is our opinion that the existing artificial fill, in its present condition, is not considered suitable for direct support of proposed new foundations or slabs; however, the existing site soils are suitable for re-use as engineered fill provided the recommendations in the *Grading* section of this report are followed (see Section 7.4).

7.1.3 Excavations for the proposed subterranean level are anticipated to penetrate through the existing artificial fill and expose competent alluvial soils throughout the excavation bottom. Based on these considerations, it is recommended that the subterranean level be supported on conventional foundation system deriving support in the competent alluvial soils found at or below a depth of 12 feet from existing grade. Alternatively, the structure may also be supported on a reinforced concrete mat foundation system deriving support in the competent alluvial soils found at or below a depth of 12 feet from existing grade. Recommendations for the design of a conventional foundation system and mat foundation are provided in Sections 7.7 and 7.8, respectively.

7.1.4 The adjacent offsite structure (currently under construction) is underlain by two subterranean levels that are anticipated to extend to a depth of up to 24 feet below the existing ground surface. The proposed subterranean level will be located within the surcharge zone of the offsite subterranean levels to the north and west. The surcharge zone may be defined by a 1:1 projection up and away from the bottom of the adjacent foundations (see Section A-A', Figure 2). Based on the depth of the offsite structures, foundations within the surcharge zone should be deepened to penetrate below the surcharge influence line. This can be accomplished with deepened foundations consisting of drilled friction piles. Recommendations for the design of a deepened foundation system is provided in Sections 7.9 and 7.10.

7.1.5 It is the intent of the Geotechnical Engineer to allow both conventional and deep foundation systems for this project, provided anticipated differential settlements are within the allowable structural tolerance. Geocon will consult with the project structural engineer during foundation design to ensure that the mixed foundation system is properly designed. All foundations must derive support in the competent undisturbed alluvial soils found at or below a depth of 12 feet below the existing ground surface.

7.1.6 Excavation for the subterranean parking level is anticipated to extend to depths of up to 23 feet below ground surface, including foundation construction. Due to the depth of the excavation and the proximity to the property lines, city streets and adjacent offsite structures, excavation of the proposed subterranean level will likely require sloping and shoring measures in order to provide a stable excavation. Where shoring is required it is recommended that a soldier pile shoring system be utilized. In addition, where the proposed excavation will be deeper than and adjacent to an offsite structure, the proposed shoring should be designed to resist the surcharge imposed by the adjacent offsite structure. Recommendations for *Shoring* are provided in Section 7.22 of this report.

7.1.7 Due to the subterranean nature of the proposed design, 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.

7.1.8 Foundations for small outlying structures, such as block walls up to 6 feet in height, planter walls or trash enclosures, which will not be tied to the proposed structure, may be supported on conventional foundations deriving support on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and compaction cannot be performed or is undesirable, foundations may derive support directly in the competent undisturbed alluvial soils and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials. If the soils exposed in the excavation bottom are soft or loose, compaction of the soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved by a Geocon representative. The design team and contractor should be aware that the depth to undisturbed alluvial soils may be on the order of 4 feet; recommendations for the design and construction of miscellaneous foundations should be reevaluated once formal plans are available.

7.1.9 Where new paving is to be placed, it is recommended that all existing fill and soft alluvial soils be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all existing fill and soft alluvial soils in the area of new paving is not required; however, paving constructed over existing uncertified fill or unsuitable alluvial soil may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper 12 inches of subgrade soil should be scarified and properly compacted for paving support. Paving recommendations are provided in *Preliminary Pavement Recommendations* section of this report (see Section 7.15).

7.1.10 Based on the results of percolation testing performed at the site, a stormwater infiltration system is considered feasible for this project. Recommendations for infiltration are provided in the *Stormwater Infiltration* section of this report (see Section 7.28).

7.1.11 Once the design and foundation loading configuration proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, if necessary. If the proposed building loads will exceed those presented herein, the potential for settlement should be reevaluated by this office.

7.1.12 Any changes in the design, location or elevation of improvements, 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.

7.2 Soil and Excavation Characteristics

7.2.1 The in-situ soils can be excavated with moderate effort using conventional excavation equipment. Due to the granular nature of the soils, moderate to excessive caving should be anticipated in vertical excavations. In addition, the contractor should also be aware that formwork may be required to prevent caving during foundation excavations. Furthermore, the site is located within the ancestral flood plain of the Los Angeles River and zones of cobbles and boulders could be encountered during construction.

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

7.2.3 All onsite excavations must be conducted in such a manner that potential surcharges from existing structures, construction equipment, and vehicle loads are resisted. The surcharge area may be defined by a 1:1 projection down and away from the bottom of an existing foundation or vehicle load. Penetrations below this 1:1 projection will require special excavation measures such as sloping or shoring. Excavation recommendations are provided in the *Temporary Excavations* section of this report (see Section 7.21).

7.2.4 Based on the predominantly granular nature of the soils encountered during site exploration, the soils at the proposed subterranean level are considered to be "non-expansive". The recommendations in this report assume that foundations and slabs will derive support in these materials.

7.3 Minimum Resistivity, pH, and Water-Soluble Sulfate

7.3.1 Potential of Hydrogen (pH) and resistivity testing as well as chloride content testing were performed on representative samples of soil to generally evaluate the corrosion potential to surface utilities. The tests were performed in accordance with California Test Method Nos. 643 and 422 and indicate that the soils are considered "moderately corrosive" with respect to corrosion of buried ferrous metals on site. The results are presented in Appendix B (Figure B6) and should be considered for design of underground structures.

7.3.2 Laboratory tests were performed on representative samples of the site materials to measure the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate tests are presented in Appendix B (Figure B6) 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.

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

7.4 Grading

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

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

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

7.4.4 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 structure 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 in writing by the Geotechnical Engineer. All existing underground improvement 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.

7.4.5 The proposed structure may be supported on a combination of conventional foundations and deepened foundations. All foundations must derive support in the competent undisturbed alluvial soils generally found at or below a depth of 12 feet below the existing ground surface. 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.).

7.4.6 The slab-on-grade may be placed directly on the competent undisturbed alluvium at the basement level. Any soft or disturbed alluvium should be properly compacted for slab support.

7.4.7 All excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc). If determined to be excessively soft, additional removals or stabilization of the excavation bottom may be required in order to provide a firm working surface. If required, recommendations for stabilization measures can be provided under separate cover.

7.4.8 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 a 95 percent relative compaction could be required; additional laboratory testing can be performed during grading to confirm the required degree of compaction. 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).

7.4.9 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).

7.4.10 Foundations for small outlying structures, such as block walls up to 6 feet high, planter walls or trash enclosures, which will not be tied to the proposed structure, may be supported on conventional foundations deriving support on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and compaction cannot be performed or is undesirable, such as adjacent to property lines, foundations may derive support in the undisturbed alluvial soils and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials. If the soils exposed in the excavation bottom are soft, compaction of the soft soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved by a Geocon representative. The design team and contractor should be aware that the depth to undisturbed alluvial soils may be on the order of 4 feet; recommendations for the design and construction of miscellaneous foundations should be reevaluated once formal plans are available.

7.4.11 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 B6).

7.4.12 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 observed 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 7.5). 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).

7.4.13 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.

7.5 Controlled Low Strength Material (CLSM)

7.5.1 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.

7.6 Foundation Design

7.6.1 The proposed structure may be supported on a conventional foundation system deriving support in the competent alluvial soils found at or below a depth of 12 feet from existing grade. Recommendations for the design of a conventional foundation system are provided in Section 7.7.

7.6.2 Alternatively, the proposed structure may also be supported on a reinforced concrete mat foundation system deriving support in the competent alluvial soils found at or below a depth of 12 feet from existing grade. Recommendations for the design of a mat foundation are provided in Section 7.8.

7.6.3 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.).

7.6.4 Based on the anticipated depths of the subterranean levels of the existing offsite structures, the proposed structure will be shallower than the existing offsite structures and could generate a surcharge of the existing structures. The surcharge zone may be defined by a 1:1 projection up and away from the bottom of the adjacent foundations (see Section A-A', Figure 2). It is recommended that within the surcharge zone the proposed structure may be supported on deepened foundations deriving support in the undisturbed alluvium found at and below a depth of 12 feet. Foundations should be deepened as necessary to extend below the surcharge zone.

7.6.5 It is the intent of the Geotechnical Engineer to allow both conventional and deep foundations systems for this project, provided anticipated differential settlements are within the allowable structural tolerance. Geocon will consult with the project structural engineer during foundation design to ensure that the mixed foundation system is properly designed.

7.6.6 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.

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

7.6.8 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.

7.7 Conventional Foundation Design

7.7.1 Continuous footings may be designed for an allowable bearing capacity of 2,500 pounds per square foot (psf), and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade, and 12 inches into the recommended bearing materials.

7.7.2 Isolated spread foundations may be designed for an allowable bearing capacity of 3,000 psf, and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade, and 12 inches into the recommended bearing materials.

7.7.3 The soil bearing pressure above may be increased by 400 psf and 800 psf for each additional foot of foundation width and depth, respectively, up to a maximum recommended bearing pressure of 4,000 psf. Higher bearing capacities are feasible; however, the use of higher bearing capacities will require coordination with the structural engineer on anticipated total and differentials settlements

7.7.4 The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

7.7.5 Continuous footings should be reinforced with a minimum of four No. 4 steel reinforcing bars, two placed near the top of the footing and two near the bottom. Reinforcement for spread footings should be designed by the project structural engineer.

7.7.6 If depth increases are utilized for the exterior wall footings, 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.

7.7.7 The above foundation dimensions and minimum reinforcement recommendations are based on soil conditions and building code requirements only, and are not intended to be used in lieu of those required for structural purposes.

7.8 Mat Foundation Design

7.8.1 It is anticipated that the mat foundation will impart an average pressure of less than 2,000 psf, with locally higher pressures up to 4,000 psf. The recommended maximum allowable bearing value is 4,000 psf. The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

7.8.2 A vertical modulus of subgrade reaction of 150 pounds per cubic inch (pci) may be used in the design of mat foundations deriving support in competent alluvial soils. This value is a unit value for use with a 1-foot square footing. The modulus should be adjusted in accordance with the following equation when used with larger foundations:

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

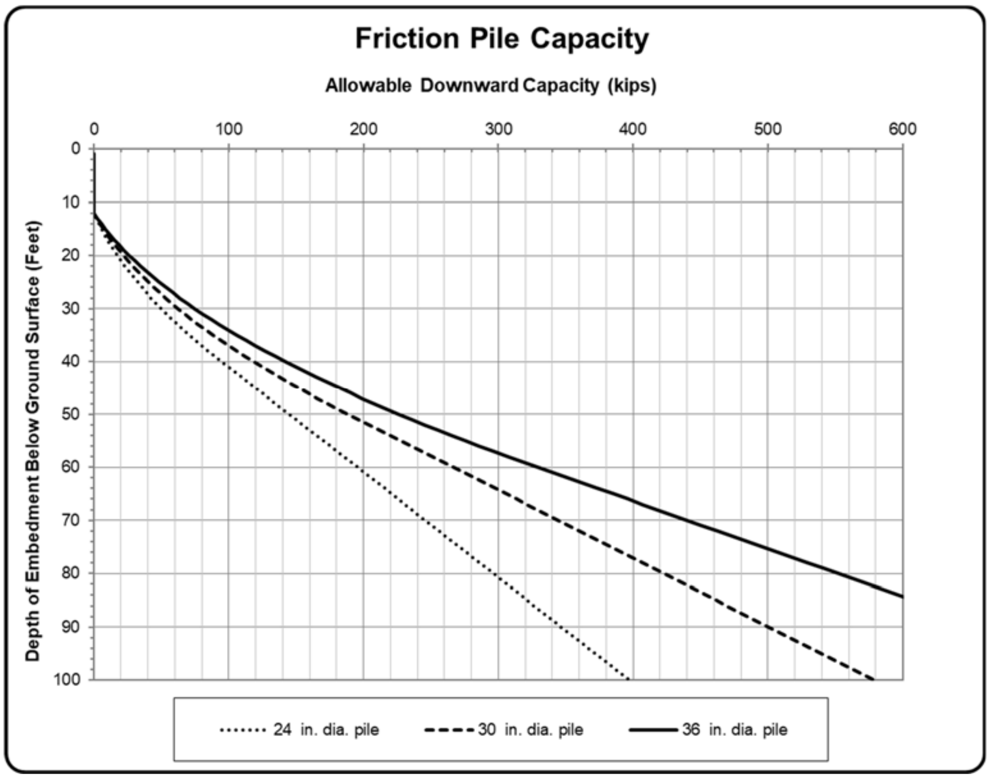
where: K_R = reduced subgrade modulus
K = unit subgrade modulus
B = foundation width (in feet)

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

7.8.4 For seismic design purposes, a coefficient of friction of 0.4 may be utilized between the concrete mat and alluvium or engineered fill without a moisture barrier, and 0.15 for slabs underlain by a moisture barrier or methane barrier.

7.9 Deepened Foundation Design

7.9.1 For preliminary design purposes 24-, 30-, and 36-inch diameter drilled cast-in-place friction piles have been evaluated. The allowable axial capacities for pile embedment into the competent alluvial soils below the surcharge line are provided in the chart on the following page. The axial capacities are based on skin friction; end-bearing capacity is not being considered.



7.9.2 Caissons within the surcharge influence zone should be deepened to extend below the surcharge influence line and should only develop capacity in the soils below the surcharge influence line (see Section A-A', Figure 2).

7.9.3 Single caisson uplift capacity can be taken as 60 percent of the allowable downward capacity. The allowable downward capacity and allowable uplift capacity may be increased by one-third when considering transient wind or seismic loads.

7.9.4 A continuous grade beam foundation and/or a structural slab may be placed across the top of the caisson foundations to tie the caissons in two directions, and the appropriate span between caissons should be determined by a qualified structural engineer.

8.9.5 All drilled pile excavations should be continuously observed by personnel of this firm to verify adequate penetration into the recommended bearing materials. The capacity presented is based on the strength of the soils. The compressive and tensile strength of the caisson sections should be checked to verify the structural capacity of the caissons.

7.9.5 Friction piles do not require the complete removal of all loose earth materials from the bottom of the excavation since the end-bearing capacity is not being considered for design. However, a cleanout of the excavation bottom will be required.

7.9.6 If pile spacing is at least three times the maximum dimension of the pile, no reduction in axial capacity is considered necessary for group effects. If pile spacing is closer than three pile diameters, an evaluation for group effects including appropriate reductions should be performed by Geocon based on pile dimension and spacing.

7.10 Deepened Foundation Installation

7.10.1 Casing may be required, especially where granular soils or loose fills are encountered. Furthermore, the site is located within the ancestral flood plain of the Los Angeles River and, although not encountered in our borings, zones of cobbles and boulders could be encountered during construction. The contractor should have casing available and should be prepared to use it. 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. Continuous observation of the drilling and pouring of the caissons by the Geotechnical Engineer (a representative of Geocon West, Inc.), is required.

7.10.2 Friction piles do not require the complete removal of all loose earth materials from the bottom of the excavation since the end-bearing capacity is not being considered for design. However, a cleanout of the excavation bottom will be required.

7.10.3 Groundwater was not encountered during site exploration, and the groundwater table is sufficiently deep that it is not expected to be encountered during caisson installation. However, local seepage may be encountered during excavations for the proposed soldier piles, especially if conducted during the rainy season. If more than 6 inches of water is present in the bottom of the excavation, a tremie is required 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.

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

7.10.5 Closely spaced piles should be drilled and filled alternately, with the concrete permitted to set at least eight hours before drilling an adjacent hole. Pile excavations should be filled with concrete as soon after drilling and inspection as possible; the holes should not be left open overnight.

7.11 Foundation Settlement

7.11.1 The maximum expected settlement for a structure supported on a conventional foundation system or reinforced mat foundation system deriving support in the recommended bearing materials and designed with a maximum bearing pressure of 4,000 psf is estimated to be less than 1 inch and occur below the heaviest loaded element. Settlement of the foundation system is expected to occur on initial application of loading. Differential settlement is expected to be less than ½ inch over a distance of 20 feet.

7.11.2 The maximum expected total settlement for proposed structure supported on a deepened foundation system is estimated to be less than ½ inch. Differential settlement between adjacent pile foundations is not expected to exceed ½ inch. Settlement of the foundation system is expected to occur on initial application of loading.

7.11.3 Differential settlement between conventional foundations and pile foundations is expected to be less than ½ inch.

7.11.4 Once the design and foundation loading configurations for the proposed structure proceeds to a more finalized plan, the estimated settlements presented in this report should be reviewed and revised, if necessary. If the final foundation loading configurations are greater than the assumed loading conditions, the potential for settlement should be reevaluated by this office.

7.12 Miscellaneous Foundations

7.12.1 Foundations for small outlying structures, such as block walls up to 6 feet in height, planter walls or trash enclosures, which will not be structurally supported by the proposed building, may be supported on conventional foundations deriving support on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and compaction cannot be performed or is undesirable, such as adjacent to property lines, foundations may derive support in the undisturbed alluvial soils, and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials. The design team and contractor should be aware that the depth to undisturbed alluvial soils may be on the order of 4 feet; recommendations for the design and construction of miscellaneous foundations should be reevaluated once formal plans are available.

7.12.2 If the soils exposed in the excavation bottom are soft, compaction of the soft soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved by a Geocon representative. Miscellaneous foundations may be designed for a bearing value of 1,500 psf, and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade and 12 inches into the recommended bearing material. The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

7.12.3 Foundation excavations should be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.), prior to the placement of reinforcing steel and concrete to verify that the excavations and exposed soil conditions are consistent with those anticipated.

7.13 Lateral Design

7.13.1 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 0.4 may be used with the dead load forces in newly placed engineered fill or competent alluvium.

7.13.2 Passive earth pressure for the sides of foundations and slabs poured against engineered fill or alluvium may be computed as an equivalent fluid having a density of 350 pounds per cubic foot (pcf) with a maximum earth pressure of 3,500 psf. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third. A one-third increase in the passive value may be used for wind or seismic loads.

7.13.3 Where existing offsite structures are deeper than the proposed structure, it is recommended that the lateral contribution of all foundation systems within 10 feet of an offsite structure be ignored in order to minimize or prevent a lateral surcharge on the offsite subterranean structures. The required lateral capacity can be accounted for by structural connections to other foundations that are outside of the defined surcharge area.

7.13.4 Once the project design proceeds to a more finalized state and the foundation system has been selected, analysis of lateral caisson capacity can be performed if necessary. If caissons are spaced at least at least 8 diameters on-center when loaded in-line and at least 3 diameters on-center when loaded in parallel, no reduction in lateral capacity is considered necessary for group effects. If caisson spacing is closer, an evaluation for group effects including appropriate reductions should be incorporated into the caisson design based on dimension, spacing, and the direction of loading.

7.14 Concrete Slabs-on-Grade

7.14.1 Concrete slabs-on-grade subject to vehicle loading should be designed in accordance with the recommendations in the *Preliminary Pavement Recommendations* section of this report (Section 7.15).

7.14.2 Subsequent to the recommended grading, the concrete slab-on-grade for structures, not subject to vehicle loading, should be a minimum of 4 inches thick and minimum slab reinforcement should consist of No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions. Steel reinforcing should be positioned vertically near the slab midpoint.

7.14.3 Slabs-on-grade at the ground surface that may receive moisture-sensitive floor coverings or may be used to store moisture-sensitive materials should be underlain by a vapor retarder placed directly beneath the slab. The vapor retarder and acceptable permeance should be specified by the project architect or developer based on the type of floor covering that will be installed. The vapor retarder design should be consistent with the guidelines presented in Section 9.3 of the American Concrete Institute's (ACI) Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials (ACI 302.2R-06) and should be installed in general conformance with ASTM E 1643 (latest edition) and the manufacturer's recommendations. A minimum thickness of 15 mils extruded polyolefin plastic is recommended; vapor retarders which contain recycled content or woven materials are not recommended. The vapor retarder should have a permeance of less than 0.01 perms demonstrated by testing before and after mandatory conditioning. The vapor retarder should be installed in direct contact with the concrete slab with proper perimeter seal. If the California Green Building Code requirements apply to this project, the vapor retarder should be underlain by 4 inches of clean aggregate. It is important that the vapor retarder be puncture resistant since it will be in direct contact with angular gravel. As an alternative to the clean aggregate suggested in the Green Building Code, it is our opinion that the concrete slab-on-grade may be underlain by a vapor retarder over 4 inches of clean sand (sand equivalent greater than 30), since the sand will serve a capillary break and will minimize the potential for punctures and damage to the vapor barrier.

7.14.4 For seismic design purposes, a coefficient of friction of 0.4 may be utilized between concrete slabs and subgrade soils without a moisture barrier, and 0.15 for slabs underlain by a moisture barrier.

7.14.5 Exterior slabs for walkways or flatwork, 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 moisture conditioned 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 10 feet and should be constructed using saw-cuts or other methods as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. Construction joints should be designed by the project structural engineer.

7.14.6 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.

7.15 Preliminary Pavement Recommendations

7.15.1 Where new paving is to be placed, it is recommended that all existing fill and soft or unsuitable alluvial materials be excavated and properly recompacted for paving support. The client should be aware that excavation and compaction of all existing artificial fill and soft alluvium in the area of new paving is not required; however, paving constructed over existing unsuitable material may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper 12 inches of paving subgrade should be scarified, moisture conditioned to optimum moisture content, and properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition).

7.15.2 The following pavement sections are based on an assumed R-Value of 35. Once site grading activities are complete an R-Value should be obtained by laboratory testing to confirm the properties of the soils serving as paving subgrade, prior to placing pavement.

7.15.3 The Traffic Indices listed below are estimates. Geocon does not practice in the field of traffic engineering. The actual Traffic Index for each area should be determined by the project civil engineer. If pavement sections for Traffic Indices other than those listed below are required, Geocon should be contacted to provide additional recommendations. Pavement thicknesses were determined following procedures outlined in the *California Highway Design Manual* (Caltrans). It is anticipated that the majority of traffic will consist of automobile and large truck traffic.

PRELIMINARY PAVEMENT DESIGN SECTIONS

Location	Estimated Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
Automobile Parking And Driveways	4.0	3.0	4.0
Trash Truck & Fire Lanes	7.0	4.0	9.0

7.15.4 Asphalt concrete should conform to Section 203-6 of the "*Standard Specifications for Public Works Construction*" (Green Book). Class 2 aggregate base materials should conform to Section 26-1.02A of the "*Standard Specifications of the State of California, Department of Transportation*" (Caltrans). The use of Crushed Miscellaneous Base in lieu of Class 2 aggregate base is acceptable. Crushed Miscellaneous Base should conform to Section 200-2.4 of the "*Standard Specifications for Public Works Construction*" (Green Book).

7.15.5 Unless specifically designed and evaluated by the project structural engineer, where exterior concrete paving will be utilized for support of vehicles, it is recommended that the concrete be a minimum of 6 inches of concrete reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions. Concrete paving supporting vehicular traffic should be underlain by a minimum of 4 inches of aggregate base and a properly compacted subgrade. The subgrade and base material should be compacted to 95 percent relative compaction as determined by ASTM Test Method D 1557 (latest edition).

7.15.6 The performance of pavements is highly dependent upon providing positive surface drainage away from the edge of pavements. Ponding of water on or adjacent to the pavement will likely result in saturation of the subgrade materials and subsequent cracking, subsidence and pavement distress. If planters are planned adjacent to paving, it is recommended that the perimeter curb be extended at least 12 inches below the bottom of the aggregate base to minimize the introduction of water beneath the paving.

7.16 Retaining Wall Design

7.16.1 The recommendations presented below are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 18 feet. In the event that walls significantly higher than 18 feet are planned, Geocon should be contacted for additional recommendations.

7.16.2 Retaining wall foundations may be designed in accordance with the recommendations provided in the *Foundation Design* sections of this report (see Section 7.6 through 7.10).

7.16.3 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 retaining wall pressures is provided as Figures 5 and 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 12	35	55
Up to 18	42	62

7.16.4 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.

7.16.5 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.

7.16.6 Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures and should be designed for each condition as the project progresses. Surcharges may be evaluated using section 7.27 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.

7.16.7 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, the traffic surcharge may be neglected.

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

7.17 Dynamic (Seismic) Lateral Forces

7.17.1 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).

7.17.2 A seismic load of 10 pcf should be used for design of walls that support more than 6 feet of backfill in accordance with Section 1803.5.12 of the 2015 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 seismic earth pressure is based on half of two-thirds of PGA_M calculated from ASCE 7-10 Section 11.8.3. No factor of safety has been applied to this value.

7.18 Retaining Wall Drainage

7.18.1 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.

7.18.2 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.

7.18.3 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.

7.18.4 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.

7.19 Elevator Pit Design

7.19.1 The elevator pit slab and retaining wall should be designed by the project structural engineer. As a minimum the slab-on-grade for the elevator pit bottom 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. Elevator pit walls may be designed in accordance with the recommendations in the *Foundation Design* and *Retaining Wall Design* section of this report (see Sections 7.6 through 7.10 and 7.16).

7.19.2 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.

7.19.3 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 7.18). Subdrain pipes at the base of the retaining wall drainage system should outlet to a location acceptable to the building official.

7.19.4 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.

7.20 Elevator Piston

7.20.1 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.

7.20.2 Caving is expected and the contractor should be prepared to use casing and should have it readily available at the commencement of drilling activities. Continuous observation of the drilling and installation of the elevator piston by the Geotechnical Engineer (a representative of Geocon West, Inc.) is required.

7.20.3 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.

7.21 Temporary Excavations

7.21.1 Excavations on the order of 23 feet in height are anticipated for excavation and construction of the proposed subterranean levels and foundation system. The excavations are expected to expose artificial fill and alluvial soils, which are suitable for vertical excavations up to 5 feet in height where loose soils or caving sands are not present, and where not surcharged by adjacent traffic or structures.

7.21.2 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 maximum height of 10 feet. A uniform slope does not have a vertical portion. Where space is limited, shoring measures will be required. *Shoring* recommendations are provided in the following section.

7.21.3 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.

7.22 Shoring – Soldier Pile Design and Installation

7.22.1 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.

7.22.2 One method of shoring would consist of steel soldier piles, placed in drilled holes and backfilled with concrete. Due to the proximity of adjacent offsite structures, installation of piles utilizing high frequency vibration is not recommended. 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.

7.22.3 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 and/or adjacent drainage systems.

7.22.4 All piles utilized for shoring can also be incorporated into a permanent retaining wall system (shotcrete wall) and should be designed in accordance with the earth pressure provided in the *Retaining Wall Design* section of this report (see Section 7.16).

7.22.5 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 350 psf per foot. The allowable passive value may be doubled for isolated piles, spaced a minimum of three times the pile diameter. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed alluvium.

7.22.6 Groundwater was not encountered during site exploration, and the groundwater table is sufficiently deep that it is not expected to be encountered during caisson installation. However, local seepage may be encountered during excavations for the proposed soldier piles, especially if conducted during the rainy season. If more than 6 inches of water is present in the bottom of the excavation, a tremie is required 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.

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

7.22.8 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. The site is located within the ancestral flood plain of the Los Angeles River and, although not encountered in our borings, zones of cobbles and boulders could be encountered during construction.

7.22.9 The frictional resistance between the soldier piles and retained soil may be used to resist the vertical component of the anchor load. The coefficient of friction may be taken as 0.40 based on uniform contact between the steel beam and lean-mix concrete and retained earth. The portion of soldier piles below the plane of excavation may also be employed to resist the downward loads. The downward capacity may be determined using a frictional resistance of 300 psf per foot.

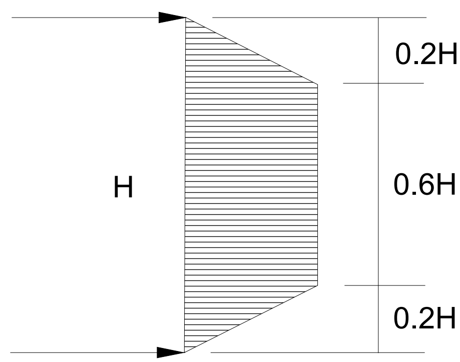
7.22.10 Due to the nature of the site soils, it is expected that continuous lagging between soldier piles will be required. However, it is recommended that the exposed soils be observed by the Geotechnical Engineer (a representative of Geocon West, Inc.), to verify the presence of any cohesive soils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

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

7.22.12 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 restrained by 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 Figures 9 and 10.

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 17	32	20H
Up to 23	36	23H

Trapezoidal Distribution of Pressure



7.22.13 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 52 pcf should be considered for the design of shoring up to 17 feet in height, and 57 pcf should be considered for the design of shoring up to 23 feet in height.

7.22.14 Where a combination of sloped embankment and shoring is utilized, the pressure will be greater and must be determined for each combination. Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic, or adjacent structures and must be determined for each combination. The surcharge pressure should be evaluated in accordance with the recommendations in Section 7.27 of this report.

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

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

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

7.22.18 Due to the depth of the excavation and proximity to adjacent structures, it is suggested that prior to excavation the existing improvements be inspected to document the present condition. For documentation purposes, photographs should be taken of preconstruction distress conditions and level surveys of adjacent grade and pavement should be considered. During excavation activities, the adjacent structures and pavement should be periodically inspected for signs of distress. In the 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.

7.23 Temporary Tie-Back Anchors

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

7.23.2 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. Based on the height of the proposed excavation, it is anticipated that one row of anchors may be required. For preliminary design purposes, it is estimated that drilled friction anchors constructed without utilizing post-grouting techniques will develop average skin frictions as follows:

- 7 feet below the top of the excavation – 730 pounds per square foot

7.23.3 Depending on the techniques utilized, and the experience of the contractor performing the installation, a maximum allowable friction capacity of 2.3 kips per linear foot for post-grouted anchors (for a 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.

7.24 Anchor Installation

7.24.1 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.

7.25 Anchor Testing

7.25.1 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.

7.25.2 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.

7.25.3 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.

7.25.4 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.

7.25.5 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.

7.26 Internal Bracing

7.26.1 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 1,800 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.

7.27 Surcharge from Adjacent Structures and Improvements

7.27.1 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.

7.27.2 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.

7.27.3 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.

7.28 Stormwater Infiltration

7.28.1 During the June 22, 2018, site exploration, boring B1 was utilized to perform percolation testing. The bottom 10 feet of the boring was backfilled with compacted soil spoils. Slotted casing was then placed in the boring, and the annular space between the casing and excavation was filled with gravel. The boring was then filled with water to pre-saturate the soils. On June 22, 2018, the casing was refilled with water and percolation test readings were performed after repeated flooding of the cased excavation. Based on the test results, the measured percolation rate and design infiltration rate, for the earth materials encountered, are provided in the following table. These values have been calculated in accordance with the Boring Percolation Test Procedure in the County of Los Angeles Department of Public Works GMED *Guidelines for Geotechnical Investigation and Reporting, Low Impact Development Stormwater Infiltration* (June 2017). Percolation test field data and calculation of the measured percolation rate and design infiltration rate are provided on Figure 11.

Boring	Soil Type	Infiltration Depth (ft)	Measured Percolation Rate (in / hour)	Design Infiltration Rate (in / hour)
B1	Poorly Graded Sand (SP)	20-35	5.29	2.65

7.28.2 Based on the test method utilized (Boring Percolation Test), the reduction factor RF_t may be taken as 2.0 in the infiltration system design. Based on the number of tests performed and consistency of the soils throughout the site, it is suggested that the reduction factor RF_v be taken as 1.0. In addition, provided proper maintenance is performed to minimize long-term siltation and plugging, the reduction factor RF_s may be taken as 1.0. Additional reduction factors may be required and should be applied by the engineer in responsible charge of the design of the stormwater infiltration system and based on applicable guidelines.

7.28.3 The results of the percolation testing indicate that the soils at depths in the above table are conductive to infiltration. It is our opinion that the soil zone encountered at the depth and location as listed in the table above are suitable for infiltration of stormwater.

7.28.4 It is our further opinion that infiltration of stormwater and will not induce excessive hydro-consolidation (see Figure B4), will not create a perched groundwater condition, will not affect soil structure interaction of existing or proposed foundations due to expansive soils, will not saturate soils supported by existing or proposed retaining walls, and will not increase the potential for liquefaction. Resulting settlements are anticipated to be less than ¼ inch, if any.

7.28.5 The infiltration system must be located such that the closest distance between an adjacent foundation is at least 10 feet in all directions from the zone of saturation. The zone of saturation may be assumed to project downward from the discharge of the infiltration facility at a gradient of 1:1. Additional property line or foundation setbacks may be required by the governing jurisdiction and should be incorporated into the stormwater infiltration system design as necessary.

7.28.6 Where the 10-foot horizontal setback cannot be maintained between the infiltration system and an adjacent footing, and the infiltration system penetrates below the foundation influence line, the proposed stormwater infiltration system must be designed to resist the surcharge from the adjacent foundation. The foundation surcharge line may be assumed to project down away from the bottom of the foundation at a 1:1 gradient. The stormwater infiltration system must still be sufficiently deep to maintain the 10-foot vertical offset between the bottom of the footing and the zone of saturation.

7.28.7 Subsequent to the placement of the infiltration system, it is acceptable to backfill the resulting void space between the excavation sidewalls and the infiltration system with minimum 2-sack slurry provided the slurry is not placed in the infiltration zone. It is recommended that pea gravel be utilized adjacent to the infiltration zone so communication of water to the soil is not hindered.

7.28.8 Due to the preliminary nature of the project at this time, the type of stormwater infiltration system and location of the stormwater infiltration systems has not yet been determined. The design drawings should be reviewed and approved by the Geotechnical Engineer. The installation of the stormwater infiltration system should be observed and approved by the Geotechnical Engineer (a representative of Geocon).

7.29 Surface Drainage

7.29.1 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.

7.29.2 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.

7.29.3 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.

7.29.4 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.

7.30 Plan Review

7.30.1 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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Ziony, J. I., and Jones, L. M., 1989, *Map Showing Late Quaternary Faults and 1978–1984 Seismicity of the Los Angeles Region, California*, U.S. Geological Survey Miscellaneous Field Studies Map MF-1964.

APPENDIX A

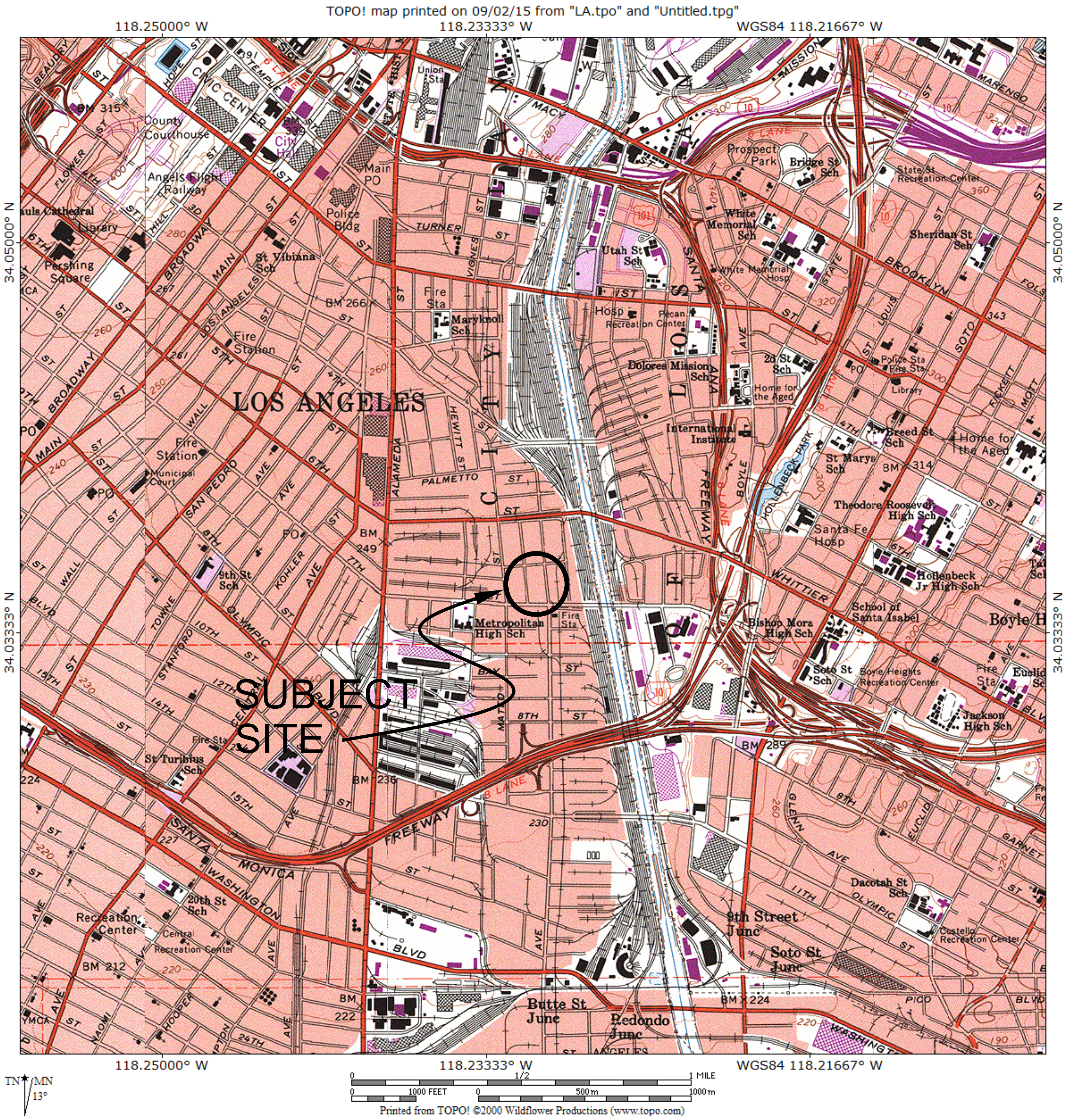
The site was explored on June 22, 2018, by excavating two 8-inch diameter borings to depths of approximately 45½ feet below the existing ground surface utilizing a truck-mounted hollow-stem auger drilling machine. Representative and relatively undisturbed samples were obtained by driving a 3 inch, O. D., California Modified Sampler into the "undisturbed" soil mass with blows from a 140-pound auto-hammer falling 30 inches. The California Modified Sampler was equipped with 1-inch high by 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 and A2. 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.

APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the International ASTM, or other suggested procedures. Selected samples were tested for direct shear strength, consolidation, corrosivity, compaction characteristics, and in-place dry density and moisture content. The results of the laboratory tests are summarized in Figures B1 through B6. The in-place dry density and moisture content of the samples tested are presented on the boring log, Appendix A.



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

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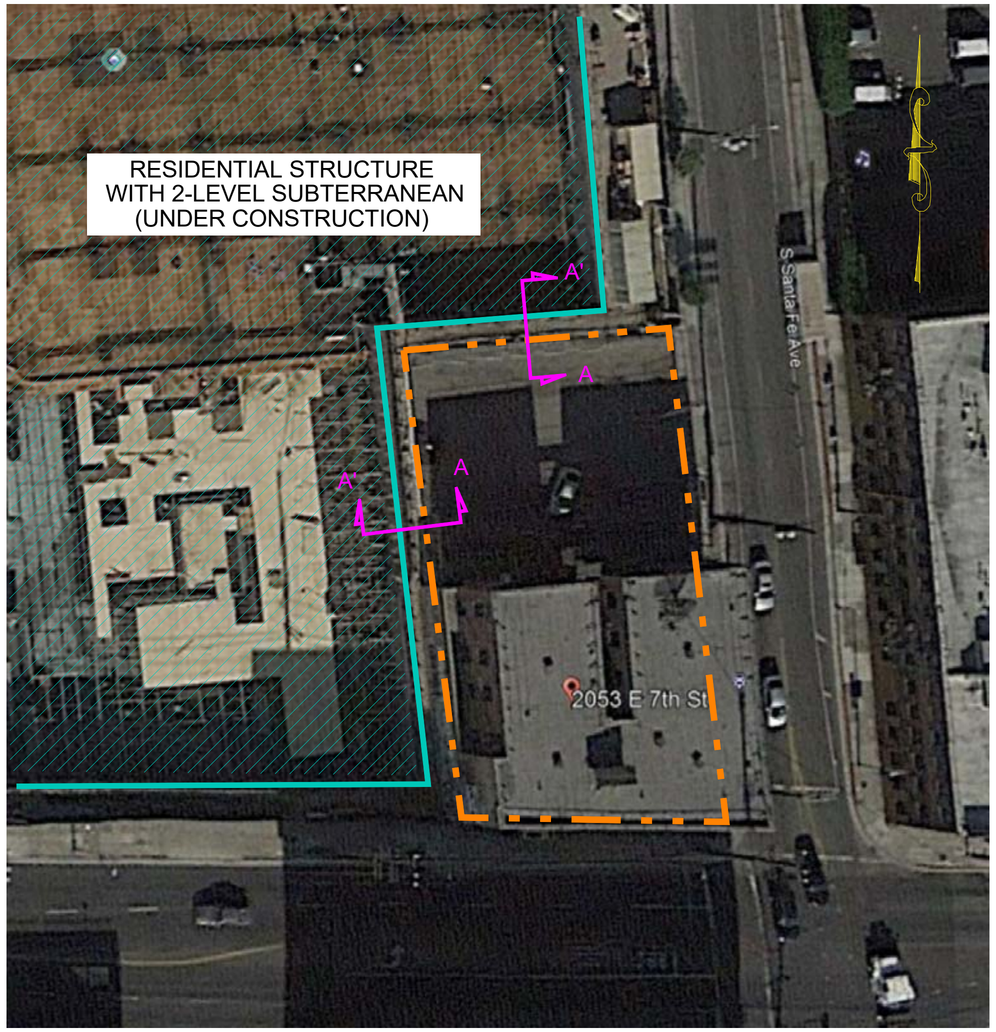
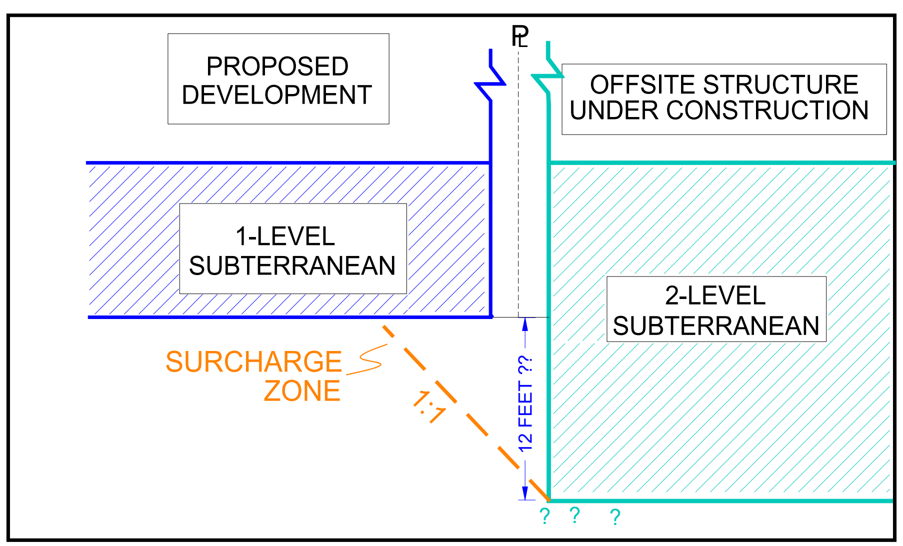
VICINITY MAP

2053 EAST 7TH STREET
LOS ANGELES, CALIFORNIA

DRAFTED BY: MDS	CHECKED BY: SFK	AUG 2018	PROJECT NO. A9815-06-01	FIG. 1



SECTION A-A'



FOR ILLUSTRATION PURPOSES ONLY
NO SCALE

LEGEND

B2 Location of Boring

Approximate Property Boundary

Approximate Location of Offsite Structure

Approximate Limits of Proposed Subterranean

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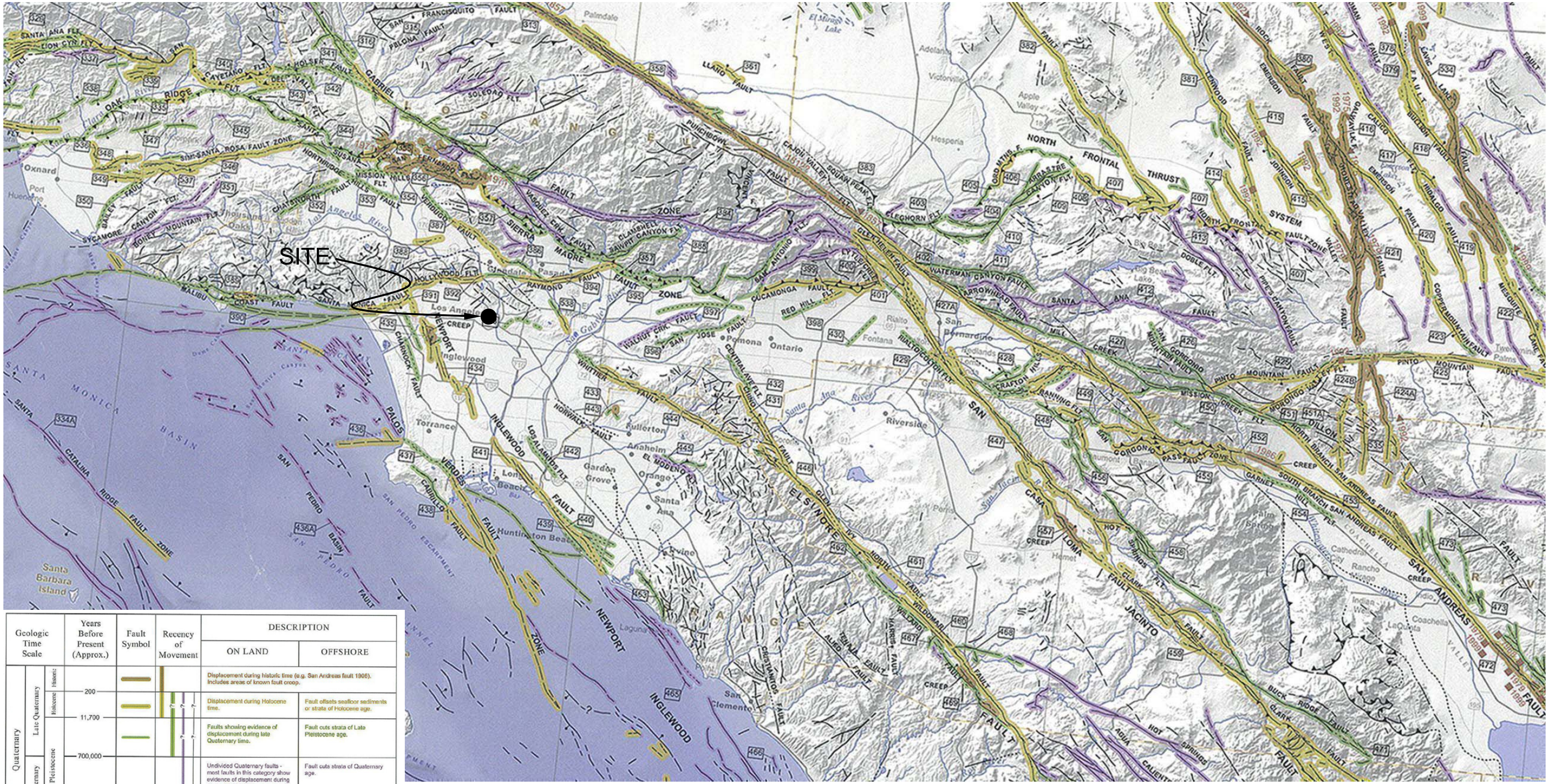
DRAFTED BY: RP	CHECKED BY: JTA/NDB

SITE PLAN

2053 EAST 7TH STREET
LOS ANGELES, CALIFORNIA

AUG 2018	PROJECT NO. A9815-06-01	FIG. 2

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



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

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

0 12 24 Miles

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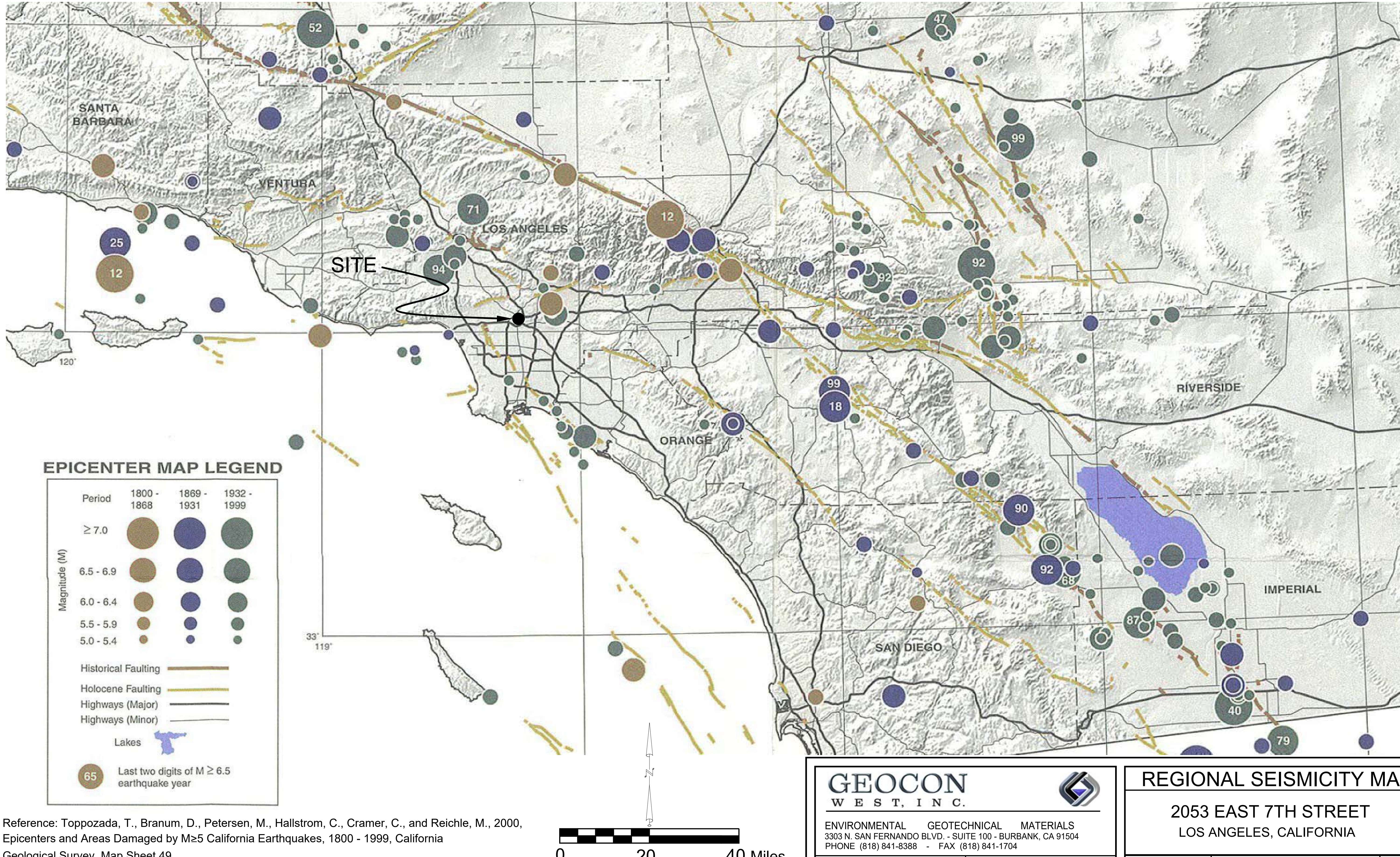
REGIONAL FAULT MAP

2053 EAST 7TH STREET
LOS ANGELES, CALIFORNIA

AUG 2018 | PROJECT NO. A9546-06-01 | FIG. 3



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

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REGIONAL SEISMICITY MAP

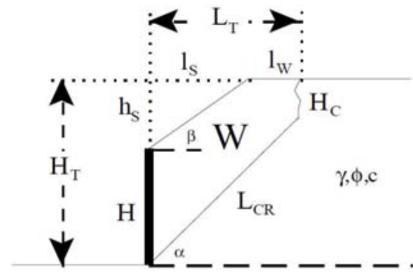
2053 EAST 7TH STREET
LOS ANGELES, CALIFORNIA

AUG 2018	PROJECT NO. A9815-06-01	FIG.4

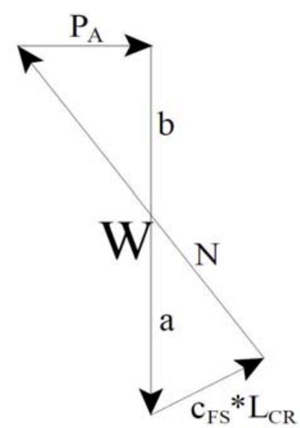
Retaining Wall Design with Transitioned Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	12.00 feet
Slope Angle of Backfill	(β)	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)	12.0 feet
Unit Weight of Retained Soils	(γ)	120.0 pcf
Friction Angle of Retained Soils	(ϕ)	30.0 degrees
Cohesion of Retained Soils	(c)	160.0 psf
Factor of Safety	(FS)	1.50
Factored Parameters:	(ϕ_{FS})	21.1 degrees
	(c_{FS})	106.7 psf



Failure Angle (α) 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	2.9	68	8153.2	12.9	3163.0	4990.2	2216.4
46	2.8	66	7893.0	12.8	3011.4	4881.6	2271.0
47	2.8	64	7638.0	12.6	2871.3	4766.7	2319.6
48	2.7	62	7388.2	12.5	2741.3	4646.8	2362.4
49	2.7	60	7143.5	12.3	2620.7	4522.8	2399.6
50	2.7	58	6904.0	12.2	2508.6	4395.4	2431.2
51	2.6	56	6669.4	12.1	2404.1	4265.3	2457.5
52	2.6	54	6439.8	11.9	2306.6	4133.2	2478.4
53	2.6	52	6214.7	11.8	2215.4	3999.3	2494.0
54	2.6	50	5994.3	11.6	2130.1	3864.2	2504.5
55	2.6	48	5778.1	11.5	2049.9	3728.1	2509.8
56	2.6	46	5566.0	11.4	1974.6	3591.4	2509.9
57	2.6	45	5357.9	11.2	1903.7	3454.2	2504.9
58	2.6	43	5153.6	11.1	1836.8	3316.8	2494.7
59	2.6	41	4952.8	11.0	1773.5	3179.3	2479.3
60	2.6	40	4755.3	10.8	1713.5	3041.8	2458.7
61	2.7	38	4561.0	10.7	1656.5	2904.5	2432.7
62	2.7	36	4369.7	10.5	1602.3	2767.5	2401.3
63	2.7	35	4181.2	10.4	1550.4	2630.8	2364.4
64	2.8	33	3995.3	10.3	1500.8	2494.5	2321.9
65	2.8	32	3811.8	10.1	1453.1	2358.7	2273.6
66	2.9	30	3630.5	10.0	1407.2	2223.4	2219.4
67	3.0	29	3451.3	9.8	1362.6	2088.7	2159.0
68	3.0	27	3274.0	9.7	1319.3	1954.6	2092.3
69	3.1	26	3098.3	9.5	1277.0	1821.3	2019.1
70	3.2	24	2924.0	9.4	1235.3	1688.7	1939.1



Design Equations (Vector Analysis):

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

$b = W-a$

$P_A = b*\tan(\alpha-\phi_{FS})$

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\ max}$ 2509.90 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 34.9 pcf 55.1 pcf

Design Wall for an Equivalent Fluid Pressure: **35 pcf** Active **55 pcf** At-Rest

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RETAINING WALL PRESSURE CALCULATION

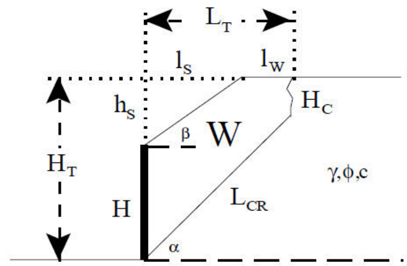
2053 EAST 7TH STREET
LOS ANGELES, CALIFORNIA

DRAFTED BY: RP	CHECKED BY: JTA/NDB	AUG 2018	PROJECT NO. A9815-06-01	FIG. 5

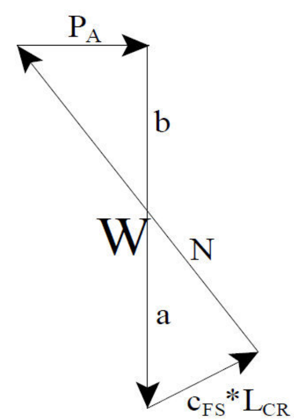
Retaining Wall Design with Transitioned Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	18.00 feet
Slope Angle of Backfill	(β)	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)	18.0 feet
Unit Weight of Retained Soils	(γ)	120.0 pcf
Friction Angle of Retained Soils	(ϕ)	30.0 degrees
Cohesion of Retained Soils	(c)	160.0 psf
Factor of Safety	(FS)	1.50
Factored Parameters:	(ϕ_{FS})	21.1 degrees
	(c_{FS})	106.7 psf



Failure Angle (α) 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	2.9	158	18960.4	21.4	5243.9	13716.5	6092.1
46	2.8	153	18329.4	21.1	4980.0	13349.4	6210.3
47	2.8	148	17715.9	20.8	4737.7	12978.2	6315.4
48	2.7	143	17119.0	20.6	4514.8	12604.2	6407.8
49	2.7	138	16538.1	20.3	4309.4	12228.7	6488.0
50	2.7	133	15972.3	20.0	4119.5	11852.8	6556.1
51	2.6	129	15420.9	19.8	3943.6	11477.3	6612.6
52	2.6	124	14883.3	19.5	3780.5	11102.8	6657.6
53	2.6	120	14358.6	19.3	3628.8	10729.8	6691.2
54	2.6	115	13846.1	19.1	3487.5	10358.7	6713.7
55	2.6	111	13345.4	18.8	3355.6	9989.7	6725.1
56	2.6	107	12855.6	18.6	3232.3	9623.3	6725.3
57	2.6	103	12376.2	18.4	3116.9	9259.4	6714.6
58	2.6	99	11906.7	18.2	3008.5	8898.2	6692.7
59	2.6	95	11446.4	18.0	2906.6	8539.8	6659.6
60	2.6	92	10994.9	17.7	2810.6	8184.2	6615.2
61	2.7	88	10551.6	17.5	2720.1	7831.5	6559.4
62	2.7	84	10116.0	17.3	2634.4	7481.6	6491.8
63	2.7	81	9687.7	17.1	2553.3	7134.5	6412.3
64	2.8	77	9266.3	16.9	2476.2	6790.1	6320.5
65	2.8	74	8851.3	16.8	2402.7	6448.5	6216.0
66	2.9	70	8442.2	16.6	2332.6	6109.6	6098.6
67	3.0	67	8038.7	16.4	2265.5	5773.3	5967.6
68	3.0	64	7640.4	16.2	2200.9	5439.5	5822.6
69	3.1	60	7246.8	16.0	2138.6	5108.2	5662.9
70	3.2	57	6857.5	15.7	2078.2	4779.4	5488.0



Design Equations (Vector Analysis):

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

$b = W-a$

$P_A = b*\tan(\alpha-\phi_{FS})$

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\ max}$ 6725.34 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 41.5 pcf 61.8 pcf

Design Wall for an Equivalent Fluid Pressure: **42 pcf** Active **62 pcf** At-Rest

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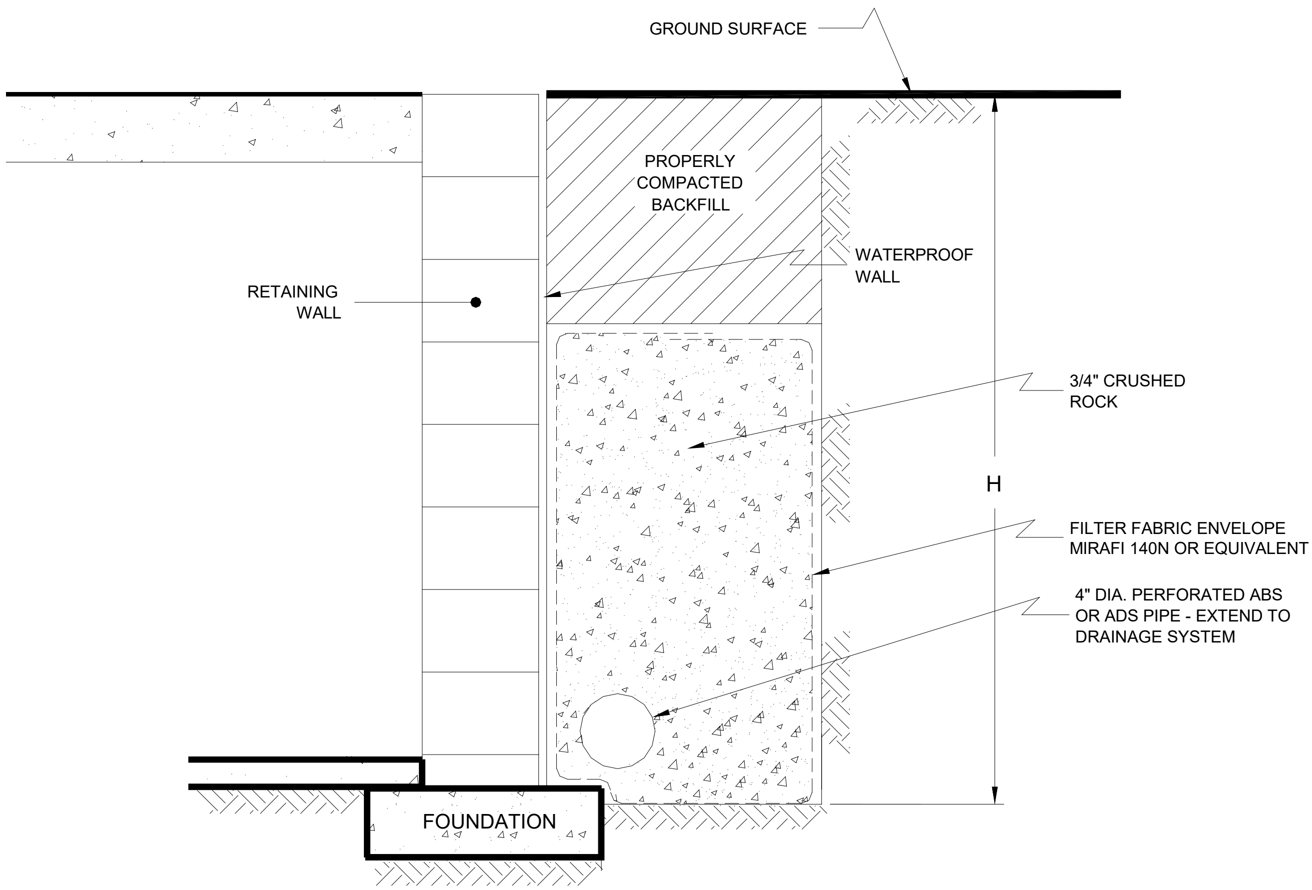
RETAINING WALL PRESSURE CALCULATION

2053 EAST 7TH STREET
LOS ANGELES, CALIFORNIA

DRAFTED BY: RP | CHECKED BY: JTA/NDB | AUG 2018 | PROJECT NO. A9815-06-01 | FIG. 6



NO SCALE

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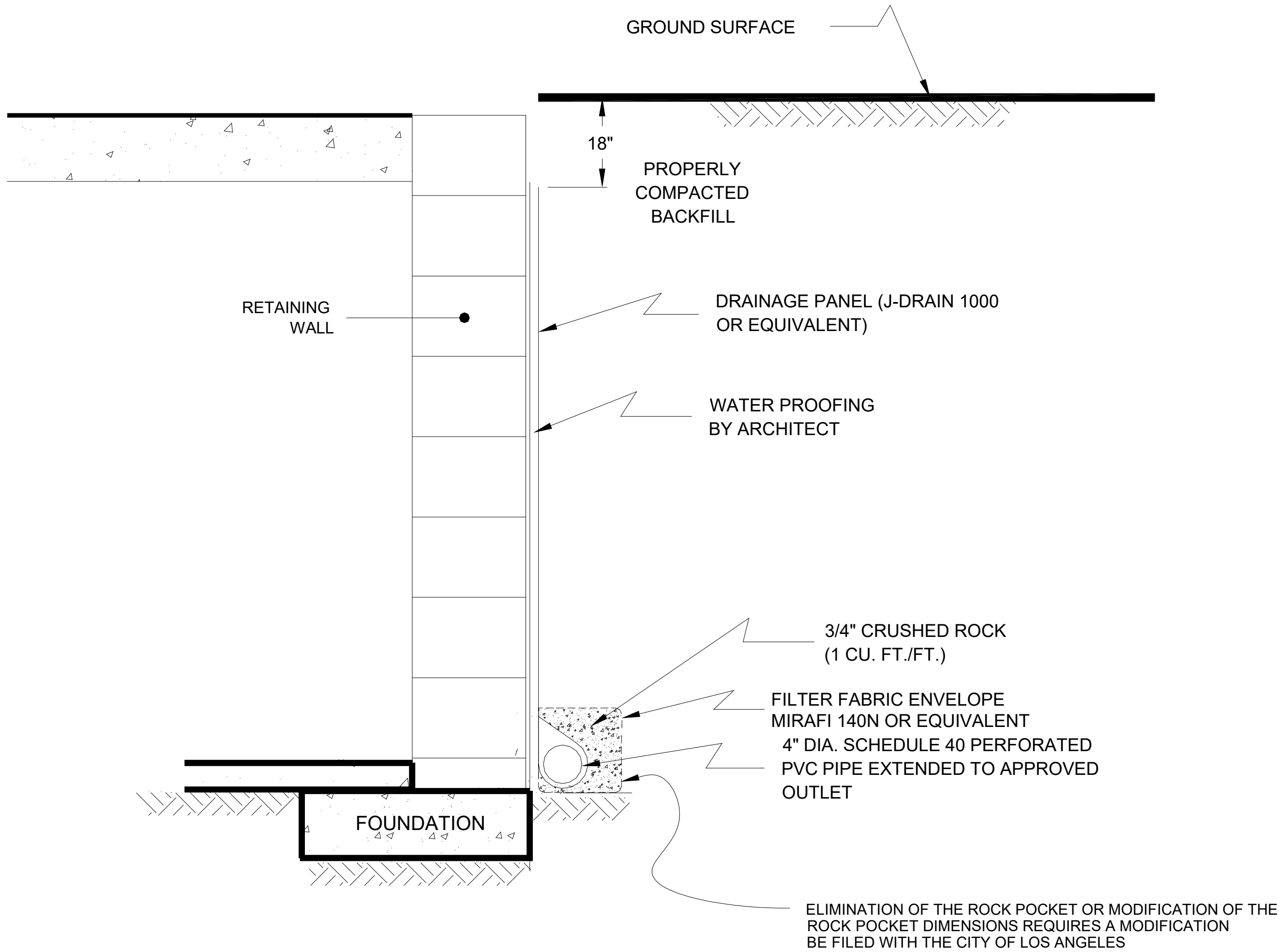
RETAINING WALL DRAIN DETAIL

2053 EAST 7TH STREET
LOS ANGELES, CALIFORNIA

AUG 2018 | PROJECT NO. A9815-06-01 | FIG. 7



NO SCALE

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RETAINING WALL DRAIN DETAIL

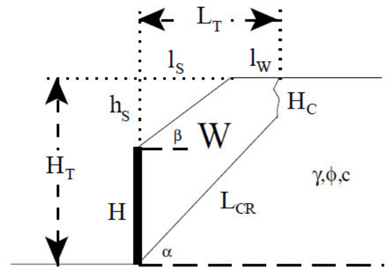
2053 EAST 7TH STREET
LOS ANGELES, CALIFORNIA

AUG 2018	PROJECT NO. A9815-06-01	FIG. 8

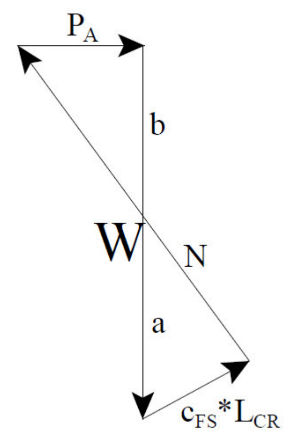
Shoring Design with Transitioned Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	17.00 feet
Slope Angle of Backfill	(β)	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)	17.0 feet
Unit Weight of Retained Soils	(γ)	120.0 pcf
Friction Angle of Retained Soils	(φ)	30.0 degrees
Cohesion of Retained Soils	(c)	160.0 psf
Factor of Safety	(FS)	1.25
Factored Parameters:	(ϕ_{FS})	24.8 degrees
	(c_{FS})	128.0 psf



Failure Angle (α) 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	4.0	137	16417.4	18.4	6206.2	10211.2	3758.8
46	3.9	133	15904.4	18.3	5874.9	10029.5	3891.9
47	3.8	128	15399.3	18.1	5571.2	9828.1	4012.5
48	3.7	124	14902.8	17.9	5292.2	9610.6	4120.8
49	3.6	120	14415.4	17.8	5035.4	9380.0	4217.2
50	3.5	116	13937.2	17.6	4798.5	9138.8	4302.1
51	3.5	112	13468.4	17.4	4579.4	8888.9	4375.6
52	3.4	108	13008.7	17.2	4376.5	8632.2	4438.0
53	3.4	105	12558.1	17.0	4188.2	8369.9	4489.5
54	3.4	101	12116.2	16.9	4013.0	8103.2	4530.3
55	3.4	97	11682.9	16.7	3849.8	7833.1	4560.6
56	3.3	94	11257.7	16.5	3697.3	7560.4	4580.3
57	3.3	90	10840.4	16.3	3554.7	7285.7	4589.6
58	3.3	87	10430.6	16.1	3420.9	7009.7	4588.5
59	3.3	84	10028.0	15.9	3295.3	6732.7	4577.1
60	3.4	80	9632.2	15.8	3176.9	6455.2	4555.1
61	3.4	77	9242.8	15.6	3065.3	6177.5	4522.7
62	3.4	74	8859.5	15.4	2959.6	5899.9	4479.7
63	3.4	71	8482.0	15.2	2859.5	5622.5	4425.9
64	3.5	68	8109.9	15.0	2764.2	5345.7	4361.2
65	3.5	65	7742.8	14.9	2673.4	5069.4	4285.3
66	3.6	62	7380.5	14.7	2586.5	4794.0	4198.1
67	3.7	59	7022.5	14.5	2503.1	4519.4	4099.2
68	3.8	56	6668.5	14.3	2422.6	4245.9	3988.4
69	3.9	53	6318.1	14.1	2344.7	3973.5	3865.2
70	4.0	50	5971.1	13.9	2268.8	3702.3	3729.3



Design Equations (Vector Analysis):

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

$b = W-a$

$P_A = b*\tan(\alpha-\phi_{FS})$

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\ max}$ 4589.62 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 31.8 pcf 52.3 pcf

Design Shoring for an Equivalent Fluid Pressure: **32 pcf** Active **52 pcf** At-Rest

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SHORING PRESSURE CALCULATION

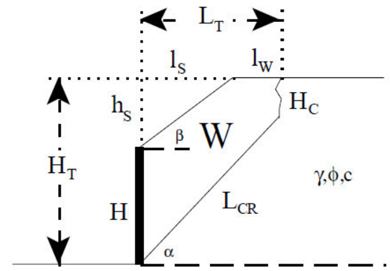
2053 EAST 7TH STREET
LOS ANGELES, CALIFORNIA

DRAFTED BY: RP	CHECKED BY: JTA/NDB	AUG 2018	PROJECT NO. A9815-06-01	FIG. 9

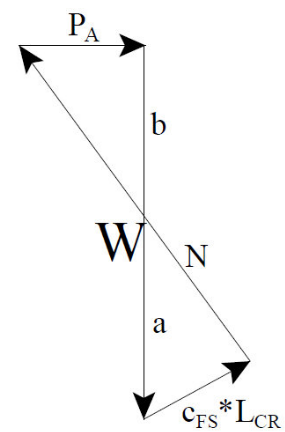
Shoring Design with Transitioned Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	23.00	feet
Slope Angle of Backfill	(β)	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)	23.0	feet
Unit Weight of Retained Soils	(γ)	120.0	pcf
Friction Angle of Retained Soils	(ϕ)	30.0	degrees
Cohesion of Retained Soils	(c)	160.0	psf
Factor of Safety	(FS)	1.25	
Factored Parameters:	(ϕ_{FS})	24.8	degrees
	(c_{FS})	128.0	psf



Failure Angle (α) 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	4.0	257	30824.6	26.9	9060.6	21764.0	8011.4
46	3.9	248	29817.3	26.6	8554.2	21263.1	8251.1
47	3.8	240	28834.2	26.3	8093.4	20740.8	8467.9
48	3.7	232	27875.1	26.0	7673.0	20202.1	8662.3
49	3.6	224	26939.4	25.7	7288.3	19651.1	8835.1
50	3.5	217	26026.3	25.4	6935.4	19090.9	8987.0
51	3.5	209	25135.1	25.1	6610.8	18524.2	9118.6
52	3.4	202	24264.8	24.8	6311.6	17953.2	9230.2
53	3.4	195	23414.7	24.5	6035.1	17379.6	9322.2
54	3.4	188	22583.7	24.3	5779.1	16804.6	9395.1
55	3.4	181	21770.9	24.0	5541.4	16229.5	9449.1
56	3.3	175	20975.5	23.7	5320.4	15655.1	9484.3
57	3.3	168	20196.5	23.5	5114.4	15082.2	9500.9
58	3.3	162	19433.2	23.2	4922.0	14511.2	9499.0
59	3.3	156	18684.7	22.9	4742.1	13942.6	9478.5
60	3.4	150	17950.2	22.7	4573.3	13376.9	9439.4
61	3.4	144	17228.8	22.4	4414.7	12814.1	9381.5
62	3.4	138	16520.0	22.2	4265.5	12254.5	9304.6
63	3.4	132	15822.8	22.0	4124.6	11698.2	9208.5
64	3.5	126	15136.7	21.7	3991.4	11145.4	9092.8
65	3.5	121	14461.0	21.5	3865.1	10596.0	8957.0
66	3.6	115	13795.0	21.2	3745.0	10050.0	8800.8
67	3.7	109	13138.0	21.0	3630.5	9507.5	8623.5
68	3.8	104	12489.4	20.7	3520.9	8968.4	8424.5
69	3.9	99	11848.5	20.5	3415.8	8432.8	8203.0
70	4.0	93	11214.9	20.2	3314.3	7900.6	7958.3



Design Equations (Vector Analysis):

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

$b = W-a$

$P_A = b*\tan(\alpha-\phi_{FS})$

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\ max}$ 9500.94 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

	Active	At-Rest
EFP	35.9 pcf	56.5 pcf
Design Shoring for an Equivalent Fluid Pressure:	**36 pcf**	**57 pcf**

GEOCON WEST, INC.

ENVIRONMENTAL GEOTECHNICAL MATERIALS

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DRAFTED BY: RP | CHECKED BY: JTA/NDB

SHORING PRESSURE CALCULATION

2053 EAST 7TH STREET

LOS ANGELES, CALIFORNIA

AUG 2018 | PROJECT NO. A9815-06-01 | FIG. 10

BORING PERCOLATION TEST FIELD LOG

Date:	Friday, June 22, 2018	**Boring/Test Number:**	Boring B1
Project Number:	A9815-06-01	**Diameter of Boring:**	8 inches
Project Location:	Rendon Hotel	**Diameter of Casing:**	2 inches
Earth Description:	Poorly Graded Sand (SP)	**Depth of Boring:**	35 feet
Tested By:	RP	**Depth to Invert of BMP:**	20 feet
Liquid Description:	Clear Clean Tap Water	**Depth to Water Table:**	n/a feet
Measurement Method:	Sounder	**Depth to Initial Water Depth (d_1):**	240 inches
Start Time for Pre-Soak:	8:40 AM	**Water Remaining in Boring (Y/N):**	None after 30 mins
Start Time for Standard:	9:40 AM	**Standard Time Interval Between Readings:**	10 min

Reading Number	Time Start (hh:mm)	Time End (hh:mm)	Elapsed Time Δtime (min)	Water Drop During Standard Time Interval, Δd (in)	Soil Description Notes Comments
1	10:00 AM	10:10 AM	10	82.8	
2	10:15 AM	10:25 AM	10	82.2	
3	10:30 AM	10:40 AM	10	82.0	Stabilized Readings
4	10:45 AM	10:55 AM	10	81.4	Achieved with Readings
5	11:00 AM	11:10 AM	10	81.0	2 through 8
6	11:15 AM	11:25 AM	10	80.6	
7	11:30 AM	11:40 AM	10	80.4	
8	11:45 AM	11:55 AM	10	79.8	

MEASURED PERCOLATION RATE & DESIGN INFILTRATION RATE CALCULATIONS*

* Calculations Below Based on Stabilized Readings Only

Boring Radius, r: 4 inches

Test Section Height, h: 180.0 inches

$$Test\ Section\ Surface\ Area, A = 2\pi rh + \pi r^2$$

A = **4574** in^2

$$Discharged\ Water\ Volume, V = \pi r^2 \Delta d$$

$$Percolation\ Rate = \left(\frac{V/A}{\Delta T}\right)$$

Reading 6	V = **4053** in^3	Percolation Rate =	**5.32**	inches/hour
Reading 7	V = **4041** in^3	Percolation Rate =	**5.30**	inches/hour
Reading 8	V = **4011** in^3	Percolation Rate =	**5.26**	inches/hour

Measured Percolation Rate = **5.29** inches/hour

Reduction Factors

Boring Percolation Test, RF_t = 2

Site Variability, RF_v = 1

Long Term Siltation, RF_s = 1

$$Total\ Reduction\ Factor, RF = RF_t \times RF_v \times RF_s$$

Total Reduction Factor = 2

Design Infiltration Rate

$$Design\ Infiltration\ Rate = Measured\ Percolation\ Rate / RF$$

Design Infiltration Rate = **2.65** inches/hour

FIGURE 11

APPENDIX A

APPENDIX A

The site was explored on June 22, 2018, by excavating two 8-inch diameter borings to depths of approximately 45½ feet below the existing ground surface utilizing a truck-mounted hollow-stem auger drilling machine. Representative and relatively undisturbed samples were obtained by driving a 3 inch, O. D., California Modified Sampler into the “undisturbed” soil mass with blows from a 140-pound auto-hammer falling 30 inches. The California Modified Sampler was equipped with 1-inch high by 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 and A2. 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.

PROJECT NO. A9815-06-01

BORING 1

ELEV. (MSL.) - - DATE COMPLETED 6/22/18

EQUIPMENT HOLLOW STEM AUGER BY: RP

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ASPHALT: 2"**			
					ARTIFICIAL FILL Silty Sand, very loose, slightly moist, dark brown, fine-grained.			
2					- bricks			
4					**ALLUVIUM** Silty Sand, very loose, slightly moist, brown, fine-grained, some medium grained.			
	B1@5'			SM		5	99.3	9.7
6								
8					Sand with Gravel, well graded, medium dense, slightly moist, light brown, fine- to coarse-grained, gravel (to 3/4").			
	B1@9'					24	116.0	2.7
10								
12	B1@12'			SW	- no recovery	50 (6")	---	---
14								
	B1@15'				- no recovery	50 (6")	---	---
16								
					Sand, poorly graded, medium dense, slightly moist, light brown, fine- to medium-grained, trace fine gravel.			
18	B1@18'					52	121.7	2.7
20								
	B1@21'				- very dense, some gravel (to 1.5")	50 (5")	128.7	1.9
22					- cobbles			
24				SP				
	B1@25'				- dense, little to no gravel	60	105.1	10.4
26								
28								

A9815-06-01 BORING LOGS.GPJ

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

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

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

GEOCON

PROJECT NO. A9815-06-01

BORING 1

ELEV. (MSL.) - - DATE COMPLETED 6/22/18

EQUIPMENT **HOLLOW STEM AUGER** BY: **RP**

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'				- no recovery	50 (6")	---	---
32				SP	- cobbles			
34					Sand with Gravel, well graded, very dense, slightly moist, light brown, fine- to coarse-grained, gravel (to 2").			
36	B1@35'					50 (5")	121.9	3.0
38				SW				
40	B1@40'				- increase in gravel	50 (5")	126.4	2.0
42					- very gravelly			
44								
	B1@45'					50 (3")	114.6	7.4

Total depth of boring: 45.5 feet
Fill to 4 feet.
No groundwater encountered.
Percolation testing perfromed on 6/22/18.
Backfilled with soil cuttings and tamped.
Surface patched with ASP.

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

A9815-06-01 BORING LOGS.GPJ

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

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

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

GEOCON

PROJECT NO. A9815-06-01

BORING 2

ELEV. (MSL.) - - DATE COMPLETED 6/22/18

EQUIPMENT HOLLOW STEM AUGER BY: RP

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ASPHALT: 2.5"** **ARTIFICIAL FILL** Silty Sand, loose, slightly moist, dark brown, fine-grained, some fine gravel. - light brown, fine-grained - olive brown			
4	B2@5'			SM	**ALLUVIUM** Silty Sand, very loose, slightly moist, brown to olive, fine-grained.	6	99.2	2.0
7	B2@8'			SP	Sand, poorly graded, loose, slightly moist, brown with gray mottles, fine-grained.	15	102.5	7.7
10	B2@10'				- medium dense	21	94.7	9.4
11	B2@12'			SW	Sand with Gravel, well graded, medium dense, slightly moist, light brown, gravel (to 1.5").	39	115.9	8.5
15	B2@15'				- loose, trace interbedded silt and clay	13	123.1	3.0
15.5				SP	Sand, poorly graded, loose, slightly moist, brown, fine-grained.			
17	B2@18'			SW	Sand with Gravel, well graded, medium dense, slightly moist, light brown, fine- to coarse-grained, some oxidation staining.	42	125.7	2.0
21	B2@21'					22	121.3	2.9
23					- gravelly			
25	B2@25'				- very dense, no recovery	50 (6")	---	---

A9815-06-01 BORING LOGS.GPJ

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

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. A9815-06-01

BORING 2

ELEV. (MSL.) - - DATE COMPLETED 6/22/18

EQUIPMENT HOLLOW STEM AUGER BY: RP

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B2@30'			SW	- gravel (to 2.5")	50 (5")	108.2	8.0
32								
34				SP	Sand, poorly graded, dense, slightly moist, light brown, fine- to medium-grained.			
36	B2@35'					83	116.0	2.1
38					- cobbles			
40	B2@40'				- no recovery, very dense - cobbles - refusal	50 (5")	---	---

Total depth of boring: 41 feet
Fill to 4 feet.
No groundwater encountered.

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

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

A9815-06-01 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

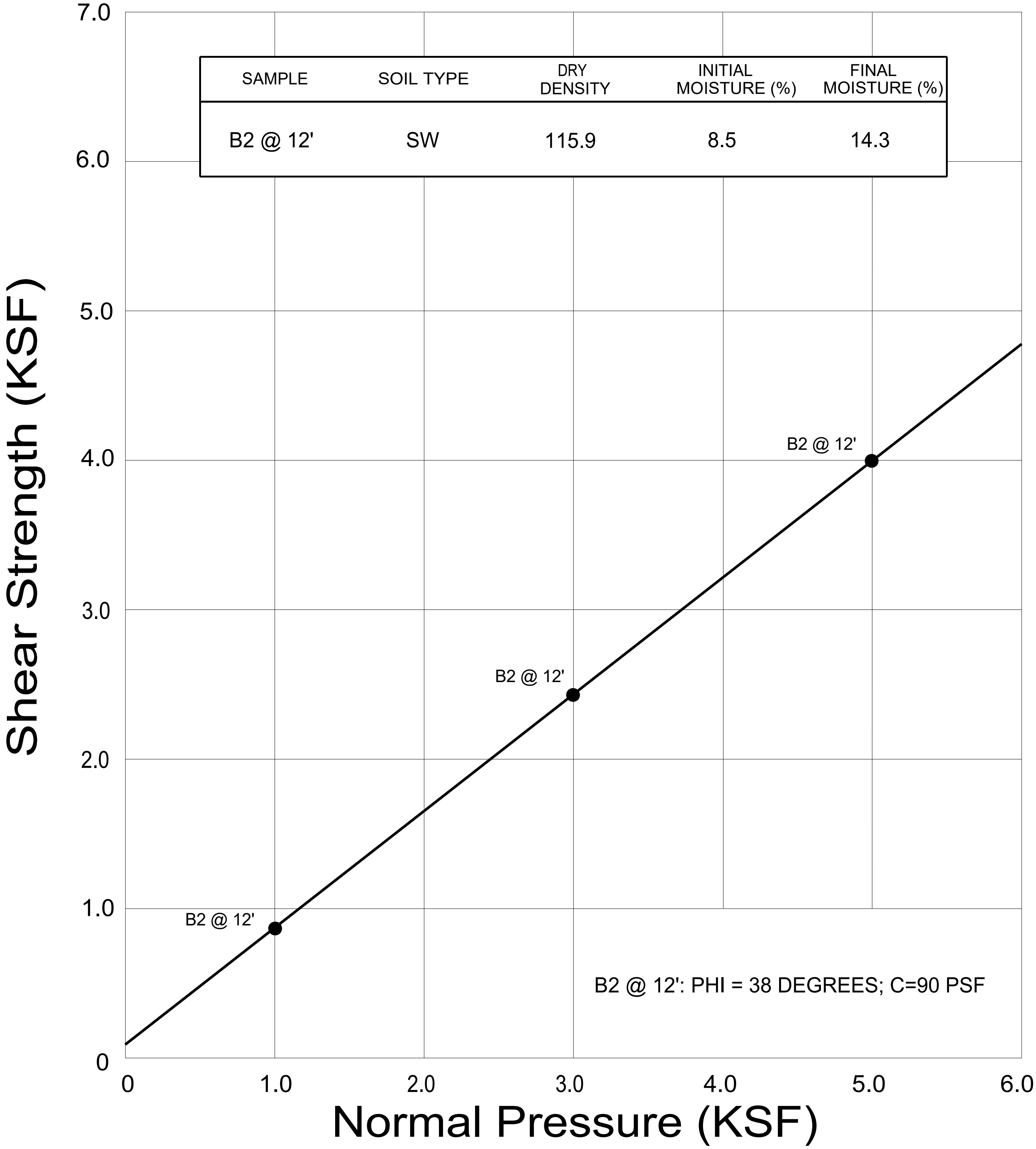
APPENDIX B

APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the International ASTM, or other suggested procedures. Selected samples were tested for direct shear strength, consolidation, corrosivity, compaction characteristics, and in-place dry density and moisture content. The results of the laboratory tests are summarized in Figures B1 through B6. The in-place dry density and moisture content of the samples tested are presented on the boring log, Appendix A.

SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2 @ 12'	SW	115.9	8.5	14.3



● Direct Shear, Saturated

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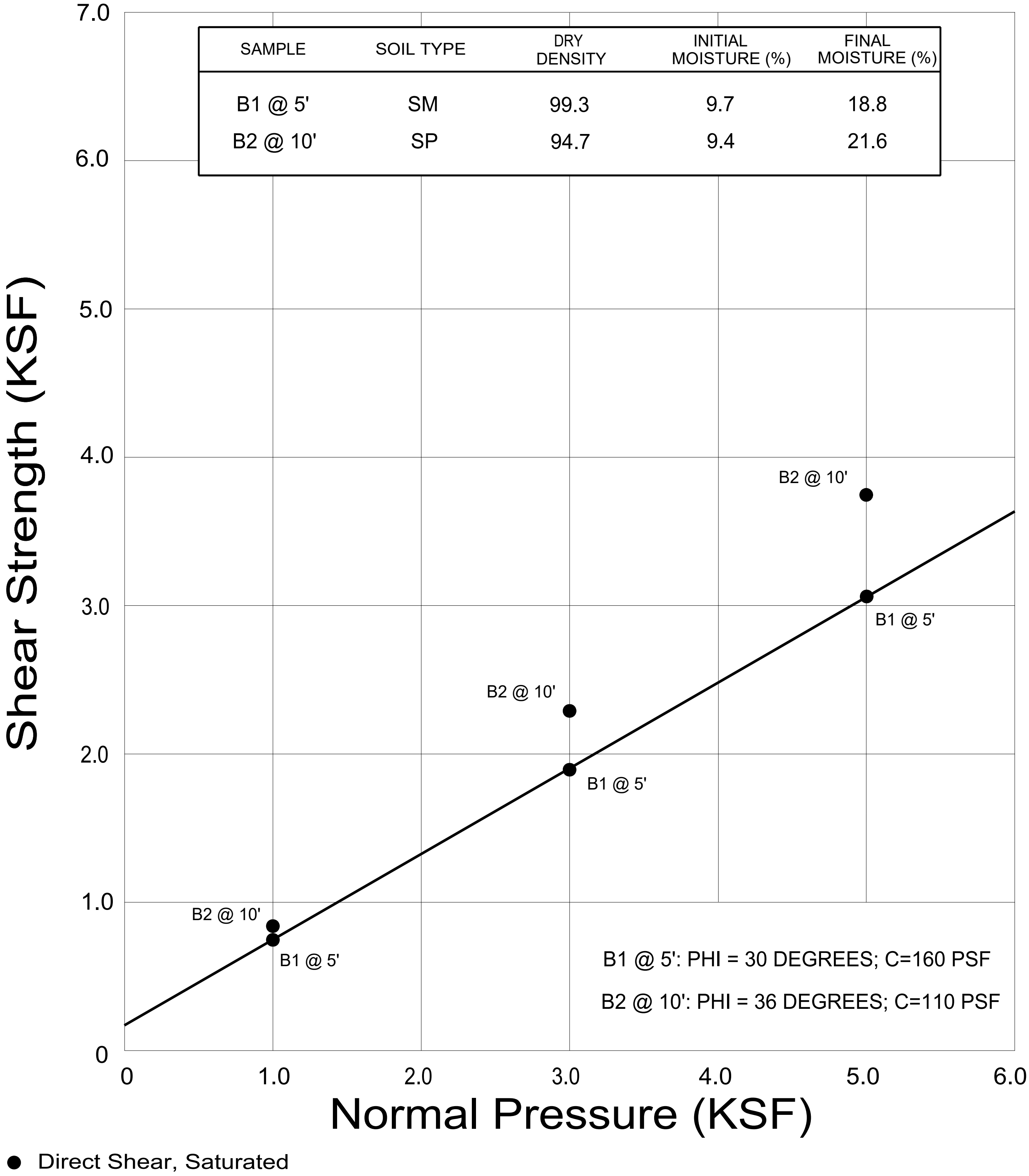
DRAFTED BY: RP | CHECKED BY: JTA/NDB

DIRECT SHEAR TEST RESULTS

2053 EAST 7TH STREET
LOS ANGELES, CALIFORNIA

AUG 2018 | PROJECT NO. A9815-06-01 | FIG. B1

SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1 @ 5'	SM	99.3	9.7	18.8
B2 @ 10'	SP	94.7	9.4	21.6



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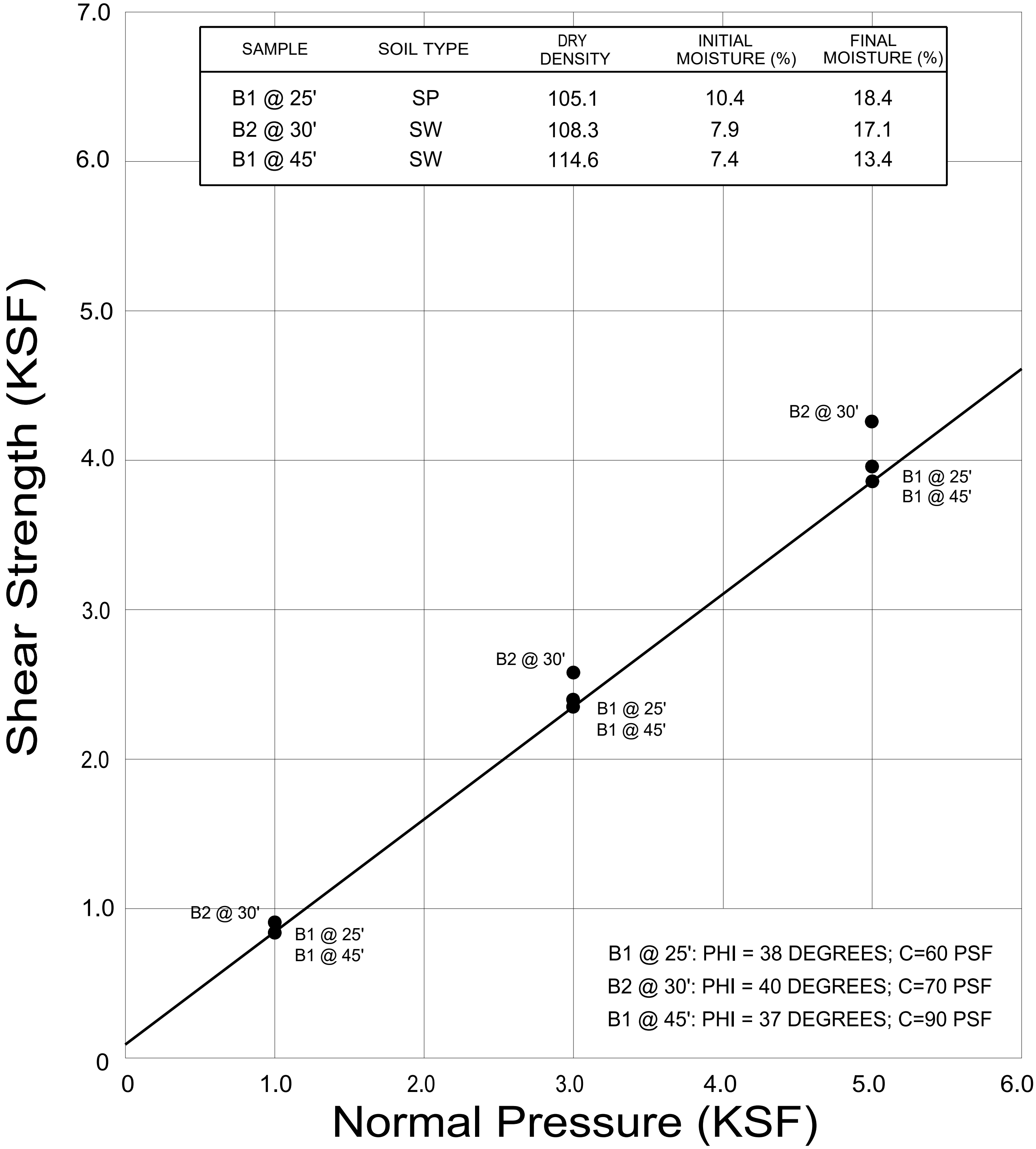
DIRECT SHEAR TEST RESULTS

2053 EAST 7TH STREET
LOS ANGELES, CALIFORNIA

DRAFTED BY: RP	CHECKED BY: JTA/NDB	AUG 2018	PROJECT NO. A9815-06-01	FIG. B2



SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1 @ 25'	SP	105.1	10.4	18.4
B2 @ 30'	SW	108.3	7.9	17.1
B1 @ 45'	SW	114.6	7.4	13.4

● Direct Shear, Saturated

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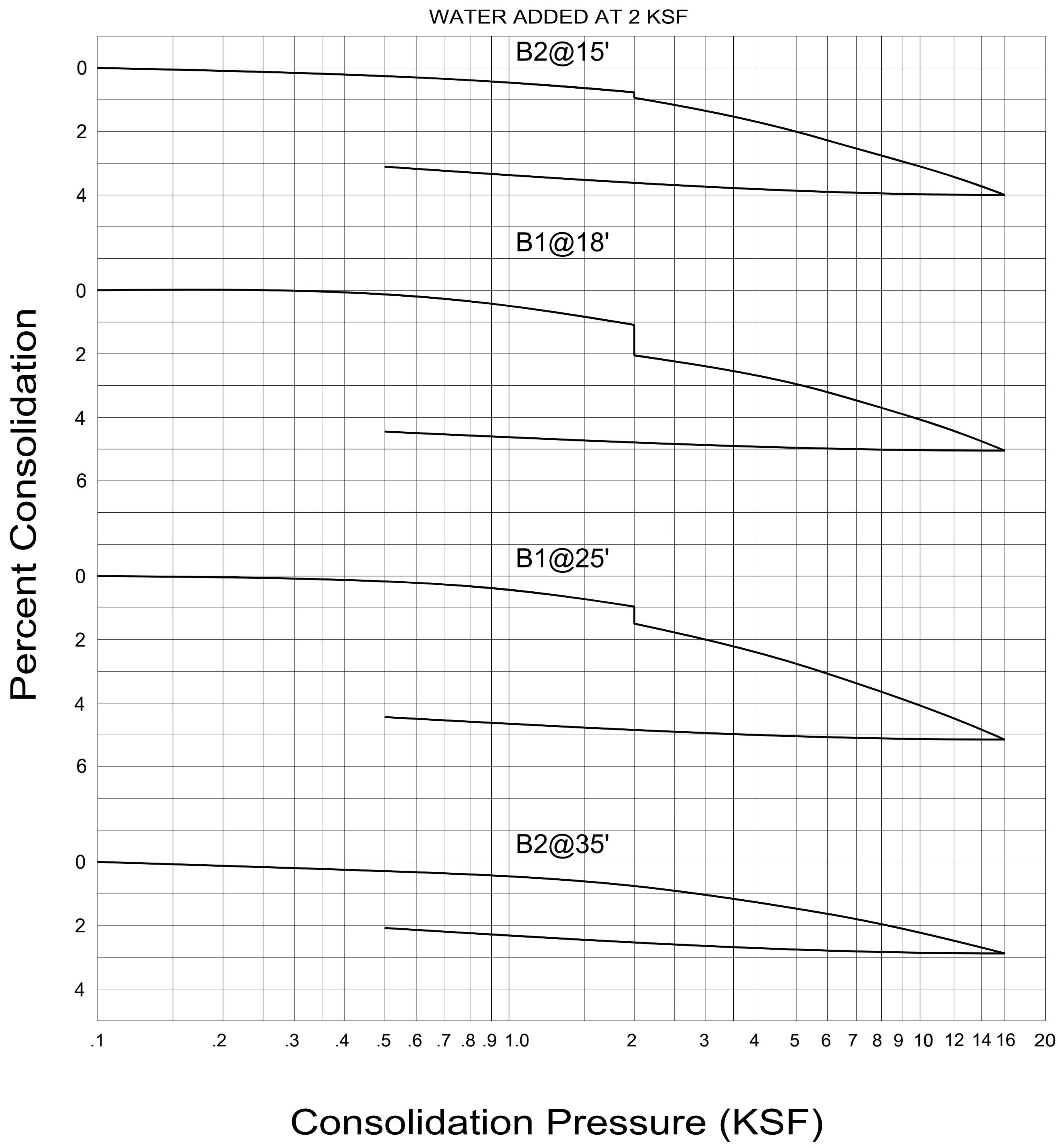
DIRECT SHEAR TEST RESULTS

2053 EAST 7TH STREET
LOS ANGELES, CALIFORNIA

DRAFTED BY: RP	CHECKED BY: JTA/NDB	AUG 2018	PROJECT NO. A9815-06-01	FIG. B3



GEOCON WEST, INC.

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DRAFTED BY: RP	CHECKED BY: JTA

CONSOLIDATION TEST RESULTS

2053 EAST 7TH STREET

LOS ANGELES, CALIFORNIA

AUG 2018	PROJECT NO. A9815-06-01	FIG. B4

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

Sample No.	Soil Description	Maximum Dry Density (pcf)	Optimum Moisture (%)
B2 @ 0-5'	Dark Brown Silty Sand	121.5	10.5

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DRAFTED BY: RP | CHECKED BY: JMT/NDB

LABORATORY TEST RESULTS

2053 EAST 7TH STREET
LOS ANGELES, CALIFORNIA

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

Sample No.	pH	Resistivity (ohm centimeters)
B2 @ 10-15'	8.41	8400 (Moderately Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS EPA NO. 325.3

Sample No.	Chloride Ion Content (%)
B2 @ 10-15'	0.006

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

Sample No.	Water Soluble Sulfate (% SO_4)	Sulfate Exposure*
B2 @ 10-15'	0.000	Negligible

*Reference: 2016 California Building Code, Section 1904 and ACI 318-11 Section 4.3.

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WEST, INC.
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DRAFTED BY: RP	CHECKED BY: JTA/NDB

CORROSIVITY TEST RESULTS

2053 EAST 7TH STREET
LOS ANGELES, CALIFORNIA

AUG 2018	PROJECT NO. A9815-06-01	FIG. B6

CITY OF LOS ANGELES

CALIFORNIA

ERIC GARCETTI
MAYOR

DEPARTMENT OF
BUILDING AND SAFETY
201 NORTH FIGUEROA STREET
LOS ANGELES, CA 90012

FRANK M. BUSH
GENERAL MANAGER
SUPERINTENDENT OF BUILDING

OSAMA YOUNAN, P.E.
EXECUTIVE OFFICER

SOILS REPORT APPROVAL LETTER

August 16, 2018

LOG # 104628
SOILS/GEOLOGY FILE - 2

1711 Lincoln LLC
1880 Century Park East, Suite 200
Los Angeles, Ca 90067

TRACT:	WINGERTER TRACT (M R 15-52)
BLOCK:	---
LOT(S):	213
LOCATION:	2053 E. 7TH ST

CURRENT REFERENCE REPORT/LETTER(S)	REPORT No.	DATE OF DOCUMENT	PREPARED BY
Soils Report	A9815-06-01	08/01/2018	Geocon West, Inc.

The Grading Division of the Department of Building and Safety has reviewed the referenced report(s) that provide(s) recommendations for the proposed 13-story hotel structure over 1-level of subterranean parking (14 levels total). Subsurface exploration performed by the consultant consisted of two hollow-stem auger borings to a maximum depth of 45 feet. The earth materials at the subsurface exploration locations consist of up to 4 feet of uncertified fill underlain by alluvium. The consultants recommend to support the proposed structure(s) on conventional, mat-type and/or drilled-pile foundations bearing on native undisturbed soils. Adjacent offsite strucre is underlain by two subterranean levels. The proposed structure shall provide a deepen foundation in which not to surcharge offsite structures.

The referenced report is acceptable, provided the following conditions are complied with during site development:

(Note: Numbers in parenthesis ( ) refer to applicable sections of the 2017 City of LA Building Code. P/BC numbers refer the applicable Information Bulletin. Information Bulletins can be accessed on the internet at LADBS.ORG.)

1. Provide a notarized letter from all adjoining property owners allowing tie-back anchors on their property (7006.6).

2. This approval does not extend to the use of an on-site infiltration systems. If an on-site infiltration system is proposed, the consultant shall provide an evaluation on the items discussed in Information Bulletin P/BC 2017-118 in a supplemental report with plans drawn to scale and suitable for reproduction and archiving purposes that clearly shows the location of the infiltration facility, all property lines, proposed and existing grades and structures, and the location of the proposed infiltration system. The plan shall be provided on the soils consultant's stationary or shall be signed and stamped by the soils engineer. Note: On-site infiltration systems are required to be a minimum

of 10 feet (in any direction) from any foundation, and a minimum of 10 feet horizontally from private property lines.

3. The soils engineer shall review and approve the detailed plans prior to issuance of any permit. This approval shall be by signature on the plans that clearly indicates the soils engineer has reviewed the plans prepared by the design engineer; and, that the plans included the recommendations contained in their reports (7006.1).

4. All recommendations of the report(s) that are in addition to or more restrictive than the conditions contained herein shall be incorporated into the plans.

5. A copy of the subject and appropriate referenced reports and this approval letter shall be attached to the District Office and field set of plans (7006.1). Submit one copy of the above reports to the Building Department Plan Checker prior to issuance of the permit.

6. A grading permit shall be obtained for all structural fill and retaining wall backfill (106.1.2).

7. All man-made fill shall be compacted to a minimum 90 percent of the maximum dry density of the fill material per the latest version of ASTM D 1557. Where cohesionless soil having less than 15 percent finer than 0.005 millimeters is used for fill, it shall be compacted to a minimum of 95 percent relative compaction based on maximum dry density. Placement of gravel in lieu of compacted fill is only allowed if complying with LAMC Section 91.7011.3.

8. Existing uncertified fill shall not be used for support of footings, concrete slabs or new fill (1809.2, 7011.3).

9. Drainage in conformance with the provisions of the Code shall be maintained during and subsequent to construction (7013.12).

10. Grading shall be scheduled for completion prior to the start of the rainy season, or detailed temporary erosion control plans shall be filed in a manner satisfactory to the Grading Division of the Department and the Department of Public Works, Bureau of Engineering, B-Permit Section, for any grading work in excess of 200 cubic yards (7007.1).

    201 N. Figueroa Street 3rd Floor, LA (213) 482-7045

11. All loose foundation excavation material shall be removed prior to commencement of framing. Slopes disturbed by construction activities shall be restored (7005.3).

12. Controlled Low Strength Material, CLSM (slurry) proposed to be used for backfill shall satisfy the requirements specified in P/BC 2014-121.

13. The applicant is advised that the approval of this report does not waive the requirements for excavations contained in the General Safety Orders of the California Department of Industrial Relations (3301.1).

14. Temporary excavations that remove lateral support to the public way, adjacent property, or adjacent structures shall be supported by shoring or constructed using ABC slot cuts. Note: Lateral support shall be considered to be removed when the excavation extends below a plane projected downward at an angle of 45 degrees from the bottom of a footing of an existing structure, from the edge of the public way or an adjacent property. (3307.3.1)

15. Prior to the issuance of any permit that authorizes an excavation where the excavation is to be of a greater depth than are the walls or foundation of any adjoining building or structure and located closer to the property line than the depth of the excavation, the owner of the subject site shall

provide the Department with evidence that the adjacent property owner has been given a 30-day written notice of such intent to make an excavation (3307.1).

16. The soils engineer shall review and approve the shoring and/or underpinning plans prior to issuance of the permit (3307.3.2).

17. Prior to the issuance of the permits, the soils engineer and/or the structural designer shall evaluate the surcharge loads used in the report calculations for the design of the retaining walls and shoring. If the surcharge loads used in the calculations do not conform to the actual surcharge loads, the soil engineer shall submit a supplementary report with revised recommendations to the Department for approval.

18. Unsurcharged temporary excavation may be cut vertical up to 5 feet. Excavations over 5 feet shall be trimmed back at a uniform gradient not exceeding 1:1, from top to bottom of excavation, as recommended.

19. Shoring shall be designed for the lateral earth pressures specified in the section titled "Shoring" starting on page 31 of the 08/01/2018 report; all surcharge loads shall be included into the design.

20. Shoring shall be designed for a maximum lateral deflection of 1 inch, provided there are no structures within a 1:1 plane projected up from the base of the excavation. Where a structure is within a 1:1 plane projected up from the base of the excavation, shoring shall be designed for a maximum lateral deflection of ½ inch, or to a lower deflection determined by the consultant that does not present any potential hazard to the adjacent structure.

21. A shoring monitoring program shall be implemented to the satisfaction of the soils engineer.

22. All foundations shall derive entire support from native undisturbed soils, as recommended and approved by the soils engineer by inspection.

23. Foundations on the north and west side of the proposed structure shall utilized deepened foundations in which not to surcharge on to offsite structures.

24. Footings supported on approved compacted fill or expansive soil shall be reinforced with a minimum of four (4), ½-inch diameter (#4) deformed reinforcing bars. Two (2) bars shall be placed near the bottom and two (2) bars placed near the top of the footing.

25. The foundation/slab design shall satisfy all requirements of the Information Bulletin P/BC 2014-116 "Foundation Design for Expansive Soils" (1803.5.3).

26. Pile caisson and/or isolated foundation ties are required by LAMC Sections 91.1809.13 and/or 91.1810.3.13. Exceptions and modification to this requirement are provided in Information Bulletin P/BC 2014-030.

27. When water is present in drilled pile holes, the concrete shall be tremied from the bottom up to ensure minimum segregation of the mix and negligible turbulence of the water (1808.8.3).

28. Existing uncertified fill shall not be used for lateral support of deep foundations (1810.2.1).

29. The seismic design shall be based on a Site Class D as recommended. All other seismic design parameters shall be reviewed by LADBS building plan check.

30. Retaining/Basement walls shall be designed for the lateral earth pressures specified in the section titled "Retaining Wall Design" starting on page 27 of the 08/01/2018 report. Note: All surcharge loads shall be included into the design.

31. Retaining walls higher than 6 feet shall be designed for lateral earth pressure due to earthquake motions as specified on page 29 of the 08/01/2018 report (1803.5.12).

32. All retaining walls shall be provided with a standard surface backdrain system and all drainage shall be conducted in a non-erosive device to the street in an acceptable manner (7013.11).

33. With the exception of retaining walls designed for hydrostatic pressure, all retaining walls shall be provided with a subdrain system to prevent possible hydrostatic pressure behind the wall. Prior to issuance of any permit, the retaining wall subdrain system recommended in the soils report shall be incorporated into the foundation plan which shall be reviewed and approved by the soils engineer of record (1805.4).

34. Installation of the subdrain system shall be inspected and approved by the soils engineer of record and the City grading/building inspector (108.9).

35. Basement walls and floors shall be waterproofed/damp-proofed with an LA City approved "Below-grade" waterproofing/damp-proofing material with a research report number (104.2.6).

36. Prefabricated drainage composites (Miradrain, Geotextiles) may be only used in addition to traditionally accepted methods of draining retained earth.

37. All roof, pad and deck drainage shall be conducted to the street in an acceptable manner in non-erosive devices or other approved location in a manner that is acceptable to the LADBS and the Department of Public Works (7013.10).

38. All concentrated drainage shall be conducted in an approved device and disposed of in a manner approved by the LADBS (7013.10).

39. The soils engineer shall inspect all excavations to determine that conditions anticipated in the report have been encountered and to provide recommendations for the correction of hazards found during grading (7008 & 1705.6).

40. All friction pile or caisson drilling and installation shall be performed under the inspection and approval of the geologist and soils engineer. The geologist shall indicate the distance that friction piles or caissons penetrate into competent material in a written field memorandum. (1803.5.5, 1704.9)

41. Prior to pouring concrete, a representative of the consulting soils engineer shall inspect and approve the footing excavations. The representative shall post a notice on the job site for the LADBS Inspector and the Contractor stating that the work inspected meets the conditions of the report. No concrete shall be poured until the LADBS Inspector has also inspected and approved the footing excavations. A written certification to this effect shall be filed with the Grading Division of the Department upon completion of the work. (108.9 & 7008.2)

42. Prior to excavation an initial inspection shall be called with the LADBS Inspector. During the initial inspection, the sequence of construction; shoring; pile installation; protection fences; and, dust and traffic control will be scheduled (108.9.1).

43. Installation of shoring and/or pile excavations shall be performed under the inspection and approval of the soils engineer (1705.8).

44. The installation and testing of tie-back anchors shall comply with the recommendations included in the report or the standard sheets titled "Requirement for Tie-back Earth Anchors", whichever is more restrictive.

45. Prior to the placing of compacted fill, a representative of the soils engineer shall inspect and approve the bottom excavations. The representative shall post a notice on the job site for the LADBS Inspector and the Contractor stating that the soil inspected meets the conditions of the report. No fill shall be placed until the LADBS Inspector has also inspected and approved the bottom excavations. A written certification to this effect shall be included in the final compaction report filed with the Grading Division of the Department. All fill shall be placed under the inspection and approval of the soils engineer. A compaction report together with the approved soil report and Department approval letter shall be submitted to the Grading Division of the Department upon completion of the compaction. In addition, an Engineer's Certificate of Compliance with the legal description as indicated in the grading permit and the permit number shall be included (7011.3).

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