APPENDIX C: GEOTECHNICAL INVESTIGATION

Geocon West, Inc.,
Geotechnical Investigation, Proposed High-Rise Development "Olympic and Hill"
1000-1034 Hill Street and 220 & 226 West Olympic Boulevard, Los Angeles, California,
February 28, 2017.

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EXECUTIVE OFFICER

SOILS REPORT APPROVAL LETTER

June 6, 2017

LOG # 98134
SOILS/GEOLOGY FILE - 2

Onni Broadway Hill Development LP
315 W 9th St, #801
Los Angeles, CA 90015

TRACT: E.H. WORKMAN TRACT (M R 5-36) // TR 1814
LOT(S): 9 / 10 / FR12 / FR13 / FR14 / FR15 // A
LOCATION: 220, 226 W Olympic Blvd, 1000-1034 S Hill St

CURRENT REFERENCE REPORT/LETTER(S)	REPORT No.	DATE(S) OF DOCUMENT	PREPARED BY
Soils Report	A9549-06-01	02/28/2017	Geocon West, Inc.

The Grading Division of the Department of Building and Safety has reviewed the referenced report that provides recommendations for the proposed construction of a 67-story tower over up to 7-level subterranean parking.

The earth materials at the subsurface exploration locations consist of up to 10 feet of uncertified fill underlain by alluvium. The consultants recommend to support the subterranean parking underlying the podium levels on conventional spread footings, while the tower structure will be supported on mat-type foundations. All foundations shall bear on native undisturbed soils.

Traffic and other uniform surcharges on retaining walls and temporary shoring shall be designed in accordance with Information Bulletin P/BC 2017-141. An active or at-rest coefficient of lateral earth pressure shall be used for cantilever or restrained walls, respectively. These uniform surcharges shall be applied for the entire height of the surcharged retaining wall.

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. 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 his report. (7006.1)

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

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

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

6. All man-made fill shall be compacted to a minimum 90 percent of the maximum dry density of the fill material per the latest version of ASTM D 1557. Where cohesionless soil having less than 15 percent finer than 0.005 millimeters is used for fill, it shall be compacted to a minimum of 95 percent relative compaction based on maximum dry density (D1556). Placement of gravel in lieu of compacted fill is allowed only if complying with Section 91.7011.3 of the Code. (7011.3)

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

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

9. The applicant is advised that the approval of this report does not waive the requirements for excavations contained in the State Construction Safety Orders enforced by the State Division of Industrial Safety. (3301.1)

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

11. Prior to the issuance of any permit which 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)

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

13. 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.

14. Unsurcharged temporary excavation may be cut vertical up to 5 feet. For excavations over 5 feet to a maximum height of 12 feet, the portion of the excavation above the vertical cut shall be trimmed back at a uniform gradient not exceeding 1:1 (horizontal to vertical), without a vertical portion, as recommended.

15. Shoring shall be designed for the lateral earth pressures specified in the sections 7.18.18 - 7.18.21 starting on page 26 of the report; all surcharge loads shall be included into the design. Total lateral load on shoring piles shall be determined by multiplying the recommended EFP by the pile spacing.

16. Shoring shall be designed for a maximum lateral deflection of ½ inch where a structure is within a 1:1 plane projected up from the base of the excavation, and 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, as recommended.

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

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

19. Footings supported on alluvial soils 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.

20. The building design shall incorporate provisions for total anticipated differential settlement between conventional spread footing and the mat foundation in the range of 1 inch. While the conventional footing settlement was estimated to be negligible, the mat foundation was estimated to settle less than 1 inch below the central portion of the mat. The differential settlement between center and corners of the mat was estimated to be less than 0.75 inch.

21. A supplemental report shall be submitted including additional analyses required to evaluate the anticipated total and differential settlement using the final building loads and foundation depths.

22. Special provisions such as flexible or swing joints shall be made for buried utilities and drain lines to allow for differential vertical displacement.

23. Slabs placed on alluvial material or compacted fill shall be at least 5 inches thick and shall be reinforced with (#3) reinforcing bars spaced maximum of 18 inches on center each way.

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

25. Cantilever retaining walls with a level backfill shall be designed for a minimum EFP of 42 PCF, as specified on page 20 of the report. All other surcharge loads shall be incorporated into the design.

26. Retaining walls higher than 6 feet shall be designed for lateral earth pressure due to earthquake motions. A triangular pressure distribution with an equivalent fluid pressure of 7 PCF used in addition to the active lateral earth pressure shall be utilized, as specified on page 21 of the report (1803.5.12).

27. Basement walls and other walls in which horizontal movement is restricted at the top shall be designed for a triangular pressure distribution with an equivalent fluid pressure of 62 PCF as specified on page 20 of the report (1610.1).

28. Traffic and other uniform surcharges on retaining walls and temporary shoring shall be designed in accordance with Information Bulletin P/BC 2017-141. An active or at-rest coefficient of lateral earth pressure shall be used for cantilever or restrained walls, respectively.

29. Surcharges from line and point loads shall be calculated as specified in the section titled "Surcharge from Adjacent Structures and Improvements".

30. 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", whatever is more restrictive. (Research Report #23835)

31. All roof and pad drainage shall be conducted to the street in an acceptable manner. (7013.10)

32. An on-site storm water infiltration system at the subject site shall not be implemented, as recommended.

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

34. 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)

35. Prior to the pouring of concrete, a representative of the consulting soils engineer shall inspect and approve the footing excavations. He/She shall post a notice on the job site for the LADBS Building Inspector and the Contractor stating that the work so inspected meets the conditions of the report, but that no concrete shall be poured until the City Building 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)

36. Prior to excavation, an initial inspection shall be called with LADBS Inspector at which time sequence of construction, shoring, pile installation, protection fences and dust and traffic control will be scheduled. (108.9.1)

37. Installation of shoring, and/or pile installation shall be performed under the inspection and approval of the soils engineer and deputy grading inspector. (1705.6)

38. Prior to the placing of compacted fill, a representative of the soils engineer shall inspect and approve the bottom excavations. He/She shall post a notice on the job site for the City Grading Inspector and the Contractor stating that the soil inspected meets the conditions of

the report, but that no fill shall be placed until the LADBS Grading 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)

39. No footing/slab shall be poured until the compaction report is submitted and approved by the Grading Division of the Department.

Dan

DAN L. STOICA
Geotechnical Engineer I

DLS/dls
Log No. 98134
213-482-0480

cc: Geocon West, Inc., Project Consultant
LA District Office

GEOTECHNICAL INVESTIGATION

PROPOSED HIGH-RISE DEVELOPMENT
"OLYMPIC AND HILL"
1000-1034 HILL STREET AND 220 & 226 WEST OLYMPIC BOULEVARD
LOS ANGELES, CALIFORNIA

TRACT: 1814, LOT: A
TRACT: THE E.H. WORKMAN TRACT, LOT: 9, 10, 12, FR 14, & FR15

GEOCON
WEST, INC.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

ONNI CONTRACTING (CALIFORNIA) INC.
LOS ANGELES, CALIFORNIA

PROJECT NO. A9549-06-01

FEBRUARY 28, 2017

GEOCON
WEST, INC.
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. A9549-06-01
February 28, 2017

Mr. Mark Spector
ONNI Contracting (California), Inc.
315 W 9th Street, Suite 801
Los Angeles, California 90015

Subject: GEOTECHNICAL INVESTIGATION
PROPOSED HIGH-RISE DEVELOPMENT
"OLYMPIC AND HILL"
1000-1034 HILL STREET AND 220 & 226 WEST OLYMPIC BOULEVARD
LOS ANGELES, CALIFORNIA
TRACT: 1814, LOT: A
TRACT: THE E.H. WORKMAN TRACT, LOTS: 9, 10, 12, FR14, & FR15

Dear Mr. Spector:

In accordance with your authorization of our proposal dated November 7, 2016 (Revised November 17, 2016), we have performed a geotechnical investigation for the proposed high-rise development located at West Olympic Boulevard and Hill Street in the downtown area 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 project can proceed 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.

JELISA THOMAS ADAMS
GE3092

SUSAN FRANZEN KIRKGARD
No. 1754

NEAL D. BERLINER
2576

Jelisa Thomas Adams
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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 high-rise development located at 1000-1034 Hill Street and 220 & 226 West Olympic Boulevard in the City of Los Angeles, California (see Vicinity Map, Figure 1). The purpose of the investigation was to evaluate subsurface soil and geologic conditions underlying the site and, based on conditions encountered, to provide conclusions and recommendations pertaining to the geotechnical aspects of 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 January 20 and 23, 2017 by excavating two exploratory borings using a truck-mounted hollow-stem auger drilling machine to depths of 123 and 125 feet below existing ground surface. 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 1000-1034 Hill Street and 220 & 226 West Olympic Boulevard. The site is currently occupied by a parking lot. The site is bounded by West Olympic Boulevard to the northeast, by Hill Street to the northwest, by a parking lot and a four-story theater to the southwest, and an alley, several single-story buildings and a twelve-story building to the southeast. The site is relatively level, with to pronounced highs or lows. Surface water drainage at the site appears to be by sheet flow along the existing ground contours to the city streets.

Geocon previously performed a geotechnical investigation for the 12-story structure to the southeast, located at 1031 Broadway (Geocon Project No. A9421-06-01).

Based on the information provided by the Client, it is our understanding that the proposed development consists of the construction of a 67-story tower underlain by up to seven levels of subterranean parking. Due to the preliminary nature of the project, formal plans depicting the proposed development are not available for inclusion in this report. The existing site conditions are depicted on the Site Plan (see Figure 2). The proposed structure is depicted on Figure 3, Geologic Section A-A'.

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 7,500 kips, and that a bearing pressure of 15,000 psf may be required for support of the proposed tower.

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 northern portion of the Los Angeles Basin, approximately 1.5 mile west of the Los Angeles River. The Los Angeles Basin is a coastal plain between the Santa Monica Mountains to the north, the Elysian and Repetto Hills to the northeast, the Puente Hills and Whittier fault to 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, northwest-trending structural depression that has been filled by both marine and continental sedimentary deposits underlain by a basement complex of igneous and metamorphic composition. 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 5.8 miles to the west-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 unconsolidated Holocene age alluvium consisting of gravel, sand, silt and clay derived from the Elysian and Repetto Hills to the north and the Los Angeles River to the east (Dibblee, 1991; California Geological Survey, 2010). Detailed stratigraphic profiles of the materials encountered at the site are provided on the boring logs in Appendix A.

4.1 Artificial Fill

Artificial fill was encountered in our explorations to a maximum depth of 10 feet below existing ground surface. The artificial fill generally consists of brown to light yellowish brown silty sand and sandy silt with fine to coarse gravel and abundant brick fragments. The artificial fill is characterized as fine- to medium-grained, slightly moist, and loose to medium dense or stiff. The fill is likely the result of past grading and 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 alluvium was encountered beneath the fill. The alluvium generally consists of yellowish brown to grayish, brown poorly and well graded sand and silty sand with varying amounts of silt, fine to coarse gravel and cobbles. The alluvial soils are primarily fine- to coarse-grained, slightly moist and very dense.

5. GROUNDWATER

Review of the Seismic Hazard Zone Report for the Hollywood Quadrangle (California Division of Mines and Geology [CDMG], 1998), the historically highest groundwater level in the area is approximately 110 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 excavated to a maximum depth of 125 feet below the existing ground surface. Based on the historic high groundwater levels in the site vicinity, the lack of groundwater in our borings, and the depth of proposed construction, groundwater is neither expected to be encountered during construction, nor have a detrimental effect on the project. 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 7.25).

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, 2017; Bryant and Hart, 2007). By definition, an active fault is one that has had surface displacement within Holocene time (about the last 11,000 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 (Bryant and Hart, 2007) or a city-designated Preliminary Fault Rupture Study Area (City of Los Angeles, 2017) for surface fault rupture hazards. No active or potentially active faults with the potential for surface fault rupture are known to pass directly beneath the site. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed development is considered low. However, the site is located in the seismically active Southern California region, and could be subjected to moderate to strong ground shaking in the event of an earthquake on one of the many active Southern California faults. The faults in the vicinity of the site are shown in Figure 4, Regional Fault Map.

The closest surface trace of an active fault to the site is the Hollywood Fault located approximately 4.9 miles to the north (Ziony and Jones, 1989). Other nearby active faults include the Newport-Inglewood Fault Zone, the Raymond Fault, the Eagle Rock Fault, and the Santa Monica Fault located approximately 5.8 miles west-southwest, 5.9 miles north-northeast, 7.3 miles northeast, and 9.2 miles west of the site, respectively (Ziony and Jones, 1989). The active San Andreas Fault Zone is located approximately 34 miles northeast of the site.

The closest potentially active fault to the site is the MacArthur Fault located approximately 0.6 mile to the north (Ziony and Jones, 1989). Other nearby potentially active faults are the Coyote Pass Fault, the Overland Avenue Fault, and the Charnock Fault located approximately 3.1 miles east, 8.2 miles west, and 9.5 miles west of the site, respectively (Ziony and Jones, 1989).

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

however, these deep thrust faults are considered active features capable of generating future earthquakes that could result in moderate to significant ground shaking at the site.

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 5, Regional Seismicity Map. A partial list of moderate to major magnitude earthquakes that have occurred in the Southern California area within the last 100 years is included in the following table.

List of Historic Earthquakes

Earthquake (Oldest to Youngest)	Date of Earthquake	Magnitude	Distance to Epicenter (Miles)	Direction to Epicenter
San Jacinto-Hemet area	April 21, 1918	6.8	75	ESE
Near Redlands	July 23, 1923	6.3	58	E
Long Beach	March 10, 1933	6.4	34	SE
Tehachapi	July 21, 1952	7.5	79	NW
San Fernando	February 9, 1971	6.6	27	NNW
Whittier Narrows	October 1, 1987	5.9	10	E
Sierra Madre	June 28, 1991	5.8	21	NE
Landers	June 28, 1992	7.3	105	E
Big Bear	June 28, 1992	6.4	82	E
Northridge	January 17, 1994	6.7	20	NW
Hector Mine	October 16, 1999	7.1	120	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 Site-Specific Shear Wave Velocity

Geocon performed a geotechnical investigation for 1031 South Broadway (Geocon Project No. A9421-06-01; City of Los Angeles Soils Report Approval Log No 94554 dated September 19, 2016). The location of 1031 Broadway with respect to the subject site is indicated on the Site Plan (see Figure 2). As a part of that investigation, a geophysical subcontractor performed a multi-channel analysis of surface waves (MASW) survey. Based on the proximity of the survey to the subject site, it is our opinion that the measured shear wave velocity may be used for this project.

Based on the results of the MASW survey, the site-specific soil shear wave velocity for the upper 30 meters of soil (Vs30) is calculated as 416 meters/second. In accordance with Section 1613.3.2 of the 2016 California Building Code and Table 20.3-1 of ASCE 7-10, the calculated soil shear wave velocity falls within the boundaries of a Site Class "C".

6.4 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. 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	C	Table 1613.3.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	2.311g	Figure 1613.3.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.812g	Figure 1613.3.1(2)
Site Coefficient, F_A	1.0	Table 1613.3.3(1)
Site Coefficient, F_V	1.3	Table 1613.3.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	2.311g	Section 1613.3.3 (Eqn 16-37)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	1.056g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.540g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.704g	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	0.864g	Figure 22-7
Site Coefficient, F_{PGA}	1.0	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.864g	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 2008 Probabilistic Seismic Hazard Analysis (PSHA) Interactive Deaggregation online tool. The result of the deaggregation analysis indicates that the predominant earthquake contributing to the MCE peak ground acceleration is characterized as a 6.67 magnitude event occurring at a hypocentral distance of 5.9 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 6.63 magnitude occurring at a hypocentral distance of 9.4 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.

6.5 Liquefaction Potential

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

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

The State of California Seismic Hazard Zone Map for the Hollywood Quadrangle (1999) indicates that the site is not located in an area identified as having a potential for liquefaction. In addition, a review of the County of Los Angeles Safety Element (Leighton, 1990) indicates that the site is not located within an area identified as having a potential for liquefaction. Also, as previously discussed, the historic high groundwater level beneath the site is at a depth of approximately 110 feet below the existing ground surface and groundwater was not encountered in our borings (drilled to a maximum depth of 125 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.

6.6 Slope Stability

The topography at the site is relatively level and the topography in the immediate site vicinity slopes gently to the southeast. The site is not located within a City of Los Angeles Hillside Grading Area or a Hillside Ordinance Area (City of Los Angeles, 2017). The County of Los Angeles Safety Element (Leighton, 1990), indicates the site is not within 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). 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.

6.7 Earthquake-Induced Flooding

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

6.8 Tsunamis, Seiches, and Flooding

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

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

The site is within an area of minimal flooding (Zone X) as defined by the Federal Emergency Management Agency (LACDPW, 2017b).

6.9 Oil Fields & Methane Potential

Based on a review of the California Division of Oil, Gas and Geothermal Resources (DOGGR) Oil and Gas Well Location Map 119 (2006), the site is located within the Los Angeles Downtown Oil Field. The nearest well to the site is the 1211 S. Olive Street Well Number 1-2014, a plugged oil and gas production well, located approximately 0.2 mile to the southwest (DOGGR, 2017). However, due to the voluntary nature of record reporting by the oil well drilling companies, wells may be improperly located or not shown on the location map and undocumented wells could be encountered during construction. Any wells encountered will need to be properly abandoned in accordance with the current requirements of the DOGGR.

The site is within a city-designated Methane Zone (City of Los Angeles, 2017) and methane study will be required prior to development. We recommend that a qualified methane consultant be retained to perform the study and provide mitigation measures as necessary.

6.10 Subsidence

Subsidence occurs when a large portion of land is displaced vertically, usually due to the withdrawal of groundwater, oil, or natural gas. Soils that are particularly subject to subsidence include those with high silt or clay content. 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 the investigation that would preclude the construction of the proposed development provided the recommendations presented herein are followed and implemented during design and construction. The geotechnical design parameters presented herein should be reviewed and updated once subterranean elevations and structural loads are established.

7.1.2 Up to 10 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. The existing fill and site soils are suitable for re-use as engineered fill provided the recommendations in the *Grading* section of this report are followed (see Section 7.4).

7.1.3 Excavation for the subterranean portion of the structure is anticipated to penetrate through the existing artificial fill and expose undisturbed alluvial soils throughout the excavation bottom.

7.1.4 Due to the preliminary nature of the project at this time, development plans depicting the proposed towers, podium levels (if any) and the extent of the subterranean levels were not available. It is anticipated that the tower structure will be supported on reinforced concrete mat foundations, and the subterranean parking underlying the podium levels, if any, supported on conventional spread foundations. Recommendations for mat foundations and conventional spread foundations are provided herein as Sections 7.6 through 7.9. All foundations should derive support in the competent undisturbed alluvial soils generally found at or below the anticipated foundation depth. For the purposes of this report, the foundation depth has been assumed to be 80 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.).

7.1.5 Once proposed building loads become available and elevations are established, additional analyses will be required to evaluate the anticipated total and differential settlements between the foundation elements for verification that the settlements are in conformance with the City of Los Angeles policy. Updated foundation design recommendations will be provided as necessary in an addendum report.

7.1.6 The historic high groundwater level is reported at a depth of 120 feet below the ground surface. Based on the depth of proposed construction, groundwater is neither expected to be encountered during construction, nor have a detrimental effect on the project.

7.1.7 Due to the depth of the excavation and the proximity to the property lines, city streets and adjacent offsite structures, excavations will require sloping and/or 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 *Temporary Excavations* are provided in Section 7.17 of this report.

7.1.8 Due to the nature of the proposed design and intent for subterranean levels, waterproofing of subterranean walls and slabs is recommended. 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.

7.1.9 Based on the results of percolation testing performed at the site, a stormwater infiltration system is likely not feasible for this project. A summary of the percolation test results are provided in the *Stormwater Infiltration* section of this report (see Section 7.24).

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

7.2 Soil and Excavation Characteristics

7.2.1 The in-situ soils can be excavated with moderate effort using conventional excavation equipment. Caving should be anticipated in unshored excavations, especially where granular soils are encountered.

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 and shoring. Excavation recommendations are provided in the *Temporary Excavations* section of this report (see Section 7.17).

7.2.4 Based on depth of the proposed subterranean levels, the proposed structure would not be prone to the effects of expansive soils.

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

7.4.2 Earthwork should be observed, and compacted fill tested by representatives of Geocon West, Inc. The existing fill and alluvial soil 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 are removed.

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

7.4.4 All foundations should derive support in the competent undisturbed alluvial soils generally found at or below the anticipated foundation depth. For the purposes of this report, the foundation depth has been assumed to be 80 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.).

7.4.5 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). It is anticipated that the soils encountered by this firm would require the minimum 95 percent compaction requirement; however additional laboratory testing can be performed during construction to verify the compaction requirement. 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).

7.4.6 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).

7.4.7 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 corrosivity properties that are equally or less detrimental to that of the existing onsite soils (see Figure B6).

7.4.8 Utility trenches should be properly backfilled in accordance with the requirements of the Green Book (latest edition). The pipe should be bedded with clean sands (Sand Equivalent greater than 30) to a depth of at least 1 foot over the pipe, and the bedding material must be inspected and approved in writing by the Geotechnical Engineer (a representative of Geocon). The use of gravel is not acceptable unless used in conjunction with filter fabric to prevent the gravel from having direct contact with soil. The remainder of the trench backfill may be derived from onsite soil or approved import soil, compacted as necessary, until the required compaction is obtained. The use of minimum 2-sack slurry is also acceptable as backfill (see Section 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).

7.4.9 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 material, 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 It is anticipated that the tower structure will be supported on reinforced concrete mat foundations, and the subterranean parking underlying the podium levels, if any, will be supported on conventional spread foundations. All foundations should derive support in the competent undisturbed alluvial soils generally found at or below the anticipated foundation depth. For the purposes of this report, the foundation depth has been assumed to be 80 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.).

7.6.2 Once proposed foundation depths and building loads are available, additional analyses will be required to evaluate the anticipated total and differential settlements between the foundation elements for verification that the settlements are in conformance with the City of Los Angeles policy. Updated foundation design recommendations will be provided as necessary in an addendum report.

7.6.3 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.

7.6.4 Waterproofing of subterranean walls and slabs is recommended for this project for any portions of the structure that will be constructed below the groundwater table. Particular care should be taken in the design and installation of waterproofing to avoid moisture problems, or actual water seepage into the structure through any normal shrinkage cracks which may develop in the concrete walls, floor slab, foundations and/or construction joints. The design and inspection of the waterproofing is not the responsibility of the geotechnical engineer. A waterproofing consultant should be retained in order to recommend a product or method, which would provide protection to subterranean walls, floor slabs and foundations.

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

7.6.6 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 3,500 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 material.

7.7.2 Isolated spread foundations may be designed for an allowable bearing capacity of 4,300 psf, and should be a minimum of 24 inches in width, 18 inches in depth below the lowest adjacent grade, and 12 inches into the recommended bearing material.

7.7.3 The allowable soil bearing pressure above may be increased by 500 psf and 1,000 psf for each additional foot of foundation width and depth, respectively, up to a maximum allowable soil bearing pressure of 9,000 psf.

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

7.7.5 If depth increases are utilized for the perimeter foundations, 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.

7.7.6 Continuous footings should be reinforced with 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.

7.7.7 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 constructed for support of the tower will impart an average pressure of approximately 10,000 psf to 15,000 psf. The recommended maximum allowable bearing value is 15,000 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 100 pounds per cubic inch (pci) may be used in the design of mat foundations deriving support in competent alluvial soils generally found at or below the anticipated foundation depth. For the purposes of this report, the foundation depth has been assumed to be 80 feet below the existing ground surface. This value takes into consideration the estimated mat foundation size, but should be reevaluated once foundation loads and dimensions become available.

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 0.45 may be utilized between the concrete mat and alluvium without a moisture barrier, and 0.15 for slabs underlain by a moisture barrier.

7.9 Foundation Settlement

7.9.1 The maximum static settlement for conventional foundations deriving support in the recommended bearing materials and designed with a maximum bearing pressure of 9,000 psf is estimated to be negligible based on the overburden to be removed for the subterranean excavation.

7.9.2 The maximum expected static settlement for a mat foundation deriving support in competent alluvial soils and utilizing a maximum allowable bearing pressure of 15,000 psf is estimated to be less than 1 inch and occur below the central portion of the mat. The differential settlement between the center and corner of the mat is estimated to be less than ¾ inch.

7.9.3 Differential settlement between the mat foundations and conventional foundations is expected to be less than 1 inch.

7.9.4 A majority of the settlement of the foundation system is expected to occur on initial application of loading; however, minor additional settlements are expected within the first 12 months.

7.9.5 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. Based on the final foundation loading configuration, the potential for settlement should be reevaluated by this office.

7.10 Lateral Design

7.10.1 Resistance to lateral loading may be provided by friction acting at the base of foundations, slabs and by passive earth pressure. An allowable coefficient of friction of 0.45 may be used with the dead load forces in the competent alluvial soils or newly placed engineered fill.

7.10.2 Passive earth pressure for the sides of foundations and slabs poured against alluvial soils or newly placed engineered fill may be computed as an equivalent fluid having a density of 300 pcf with a maximum earth pressure of 3,000 pcf. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

7.11 Concrete Slabs-on-Grade

7.11.1 The project structural engineer may determine and design the necessary slab thickness and reinforcing for this structure. Unless specifically analyzed and designed by the project structural engineer, the slab-on-grade and ramp for the subterranean parking garage should be a minimum of 5 inches concrete reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions and positioned vertically near the slab midpoint. The concrete slab-on-grade may bear directly on competent alluvial soils. Any disturbed soils should be properly compacted for slab support.

7.11.2 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; recycled content or woven materials are not recommended. The material 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 Los Angeles 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 Los Angeles 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.

7.11.3 Waterproofing of subterranean walls and slabs is recommended for this project for any portions of the structure that will be constructed below the groundwater table. Particular care should be taken in the design and installation of waterproofing to avoid moisture problems, or actual water seepage into the structure through any normal shrinkage cracks which may develop in the concrete walls, floor slab, foundations and/or construction joints. The design and inspection of the waterproofing is not the responsibility of the geotechnical engineer. A waterproofing consultant should be retained in order to recommend a product or method, which would provide protection to subterranean walls, floor slabs and foundations.

7.11.4 For seismic design purposes, a coefficient of friction of 0.45 may be utilized between concrete slabs and soil without a moisture barrier and 0.15 for slabs underlain by a vapor retarder or methane barrier.

7.11.5 Exterior slabs, not subject to traffic loads, should be at least 4 inches thick and reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions, positioned near the slab midpoint. Prior to construction of slabs, the upper 12 inches of subgrade should be 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. The project structural engineer should design construction joints as necessary.

7.11.6 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 or concrete shrinkage. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics. Their occurrence may be reduced 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.

7.12 Retaining Walls Design

7.12.1 The recommendations presented below are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 80 feet. In the event that walls significantly higher than 80 feet are planned, Geocon should be contacted for additional recommendations.

7.12.2 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 7.9).

7.12.3 Assuming that proper drainage and permanent dewatering is maintained, 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) of 42 pcf.

7.12.4 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) of 62 pcf. Calculation of the recommended earth pressures is provided as Figure 6.

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

7.12.6 Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic, or adjacent structures. Recommendations for the incorporation of surcharges are provided in section 7.23 of this report. Once the design becomes more finalized, an addendum letter can be prepared revising recommendations and addressing specific surcharge conditions throughout the project, if necessary.

7.12.7 In addition to the recommended earth pressure, the upper ten feet of the subterranean wall adjacent to the street and parking lot should be designed to resist a uniform lateral pressure of 100 pounds per square foot, 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.

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

7.13 Dynamic (Seismic) Lateral Forces

7.13.1 The structural engineer should determine the seismic design category for the project in accordance with Section 1613 of the CBC. If the project possesses a seismic design category of D, E, or F, proposed retaining walls in excess of 6 feet in height should be designed with seismic lateral pressure (Section 1803.5.12 of the 2016 CBC).

7.13.2 A seismic load of 7 pcf should be used for design of walls that support more than 6 feet of backfill in accordance with Section 1803.5.12 of the 2016 CBC. The seismic load is applied as an equivalent fluid pressure along the height of the wall and the calculated loads result in a maximum load exerted at the base of the wall and zero at the top of the wall. This seismic load should be applied in addition to the active earth pressure. The earth pressure is based on half of two thirds of PGA_M calculated from ASCE 7-10 Section 11.8.3.

7.14 Retaining Wall Drainage

7.14.1 Retaining walls should be provided with a drainage system. At the base of the drain system, a subdrain covered with a minimum of 12 inches of gravel should be installed, and a compacted fill blanket or other seal placed at the surface (see Figure 7). The clean bottom and subdrain pipe, behind a retaining wall, should be observed by the Geotechnical Engineer (a representative of Geocon), prior to placement of gravel or compacting backfill.

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

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

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

7.15 Elevator Pit Design

7.15.1 The elevator pit slab and retaining wall should be designed by the project structural engineer. Elevator pit walls may be designed in accordance with the recommendations in the *Retaining Wall Design* section of this report (see Section 7.12).

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

7.15.3 If retaining wall drainage is to be provided, the drainage system should be designed in accordance with the *Retaining Wall Drainage* section of this report (see Section 7.14).

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

7.16 Elevator Piston

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

7.16.2 Due to the preliminary nature of the project at this time, it is unknown if a plunger-type elevator piston will be included for this project. If in the future it is determined that a plunger-type elevator piston will be constructed, the location of the proposed elevator should be reviewed by the Geotechnical Engineer to evaluate the setback from foundations and shoring piles. Additional recommendations will be provided as necessary.

7.16.3 Casing may be required in the drilled excavation. The contractor should be prepared to use casing and should have it readily available at the commencement of drilling activities. Continuous observation of the drilling and installation of the elevator piston by the Geotechnical Engineer (a representative of Geocon West, Inc.) is required.

7.16.4 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.

7.17 Temporary Excavations

7.17.1 Excavations on the order of 85 feet in height are anticipated for excavation and construction of the proposed subterranean level, including foundation excavations. The excavations are expected to expose alluvial soils, which are suitable for vertical excavations up to 5 feet where loose soils or caving sands are not present or where not surcharged by adjacent traffic or structures.

7.17.2 Vertical excavations greater than 5 feet will require sloping and/or shoring measures in order to provide a stable excavation. Where sufficient space is available, temporary unsurcharged embankments could be sloped back at a uniform 1:1 slope gradient or flatter, up to a maximum of 12 feet in height. A uniform slope does not have a vertical portion. Where space is limited, shoring measures will be required. *Shoring* data is provided in Section 7.18 of this report.

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

7.18 Shoring – Soldier Pile Design and Installation

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

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

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

7.18.4 All piles utilized for shoring can also be incorporated into a permanent retaining wall system (shotcrete wall) provided they are designed in accordance with the earth pressure provided in the *Retaining Wall Design* section of this report (see Section 7.12).

7.18.5 Drilled cast-in-place soldier piles should be placed no closer than two 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 400 pounds per square foot per. Where piles are installed by vibration techniques, the passive pressure may be assumed to mobilize across a width equal to the two times the dimension of the beam flange. The allowable passive value may be doubled for isolated piles spaced a minimum of three the pile diameter. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed soils.

7.18.6 Groundwater was not encountered during site exploration. 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.

7.18.7 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 pounds per square inch (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.

7.18.8 Caving is anticipated to occur where granular soils are encountered and the contractor should have casing available prior to commencement of pile excavation. When 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 piles by the Geotechnical Engineer (a representative of Geocon West, Inc.), is required.

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

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

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

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

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

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

7.18.15 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 0.45 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 1,000 psf per foot.

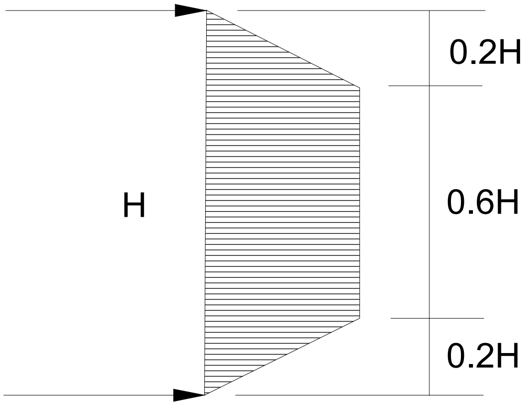
7.18.16 Due to the nature of the site soils, it is expected that continuous lagging between soldier piles will be required. However, it is recommended that the exposed soils be observed by the Geotechnical Engineer (a representative of Geocon West, Inc.), to verify the presence of any competent, cohesive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

7.18.17 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 pounds per square foot.

7.18.18 For the design of shoring, it is recommended that an equivalent fluid pressure based on the following table, be utilized for design. A diagram depicting the trapezoidal pressure distribution of lateral earth pressure is provided below the table. Calculation of the recommended shoring pressures is provided as Figure 9.

HEIGHT OF SHORING (FEET)	**EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (ACTIVE PRESSURE)**	**EQUIVALENT FLUID PRESSURE (Pounds Per Square Foot per Foot) Active Trapezoidal (Where H is the height of the shoring in feet)**
Up to 85	35	22H

Trapezoidal Distribution of Pressure



7.18.19 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, or the pile is restrained from movement by bracing or a tie back anchor, an at-rest pressure of 54 pcf should be considered for design purposes.

7.18.20 Where a combination of sloped embankment and shoring is utilized, the pressure will be greater and must be determined for each combination. Additional active pressure should be added for a surcharge condition due to slopes, vehicular traffic or adjacent structures and should be designed for each condition. The surcharge pressure should be evaluated in accordance with the recommendations in Section 7.23 of this report.

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

7.18.22 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.

7.18.23 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.

7.18.24 Due to the depth of the depth of the excavation and proximity to adjacent structures, it is suggested that prior to excavation the existing improvements be inspected to document the present condition. For documentation purposes, photographs should be taken of preconstruction distress conditions and level surveys of adjacent grade and pavement should be considered. During excavation activities, the adjacent structures and pavement should be periodically inspected for signs of distress. In the even that distress or settlement is noted, an investigation should be performed and corrective measures taken sot 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.

7.19 Tie-Back Anchors

7.19.1 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.

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

- 15 feet below the top of the excavation – 2,000 pounds per square foot
- 30 feet below the top of the excavation – 3,100 pounds per square foot
- 60 feet below the top of the excavation – 4,500 pounds per square foot

7.19.3 Depending on the techniques utilized, and the experience of the contractor performing the installation, a maximum allowable friction capacity of 5.0 kips per linear foot for post-grouted anchors (for a minimum 20-foot length beyond the active wedge) may be assumed for design purposes. Only the frictional resistance developed beyond the active wedge should be utilized in resisting lateral loads. Higher capacity assumptions may be acceptable, but must be verified by testing.

7.20 Anchor Installation

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

7.21 Anchor Testing

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

7.21.2 At least ten 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.

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

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

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

7.22 Internal Bracing

7.22.1 Rakers may be utilized to brace the soldier piles in lieu of tieback anchors. The raker bracing could be supported laterally by temporary concrete footings (deadmen) or by the permanent, interior footings. For design of such temporary footings or deadmen, poured with the bearing surface normal to rakers inclined at 45 degrees, a bearing value of 3,500 psf may be used, provided the shallowest point of the footing is at least 1 foot below the lowest adjacent grade. The structural engineer should review the shoring plans to determine if raker footings conflict with the structural foundation system. 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.

7.23 Surcharge from Adjacent Structures and Improvements

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

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

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

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

and

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

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

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

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

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

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

and

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

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

then

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

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

7.24 Stormwater Infiltration

7.24.1 During site exploration, boring 1 was utilized to perform percolation testing. The boring was advanced to the depth listed in the table below. Slotted casing was 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 January 24, 2017 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 average infiltration rate (adjusted percolation rate), for the earth materials encountered, is provided in the following table.

Boring	Infiltration Depth (ft.)	Average Infiltration Rate (in / hour)
B1	85-123	0.2

7.24.2 The results of the percolation testing indicate that the soils at depths in the above table are not conductive to infiltration. Based on these considerations, a stormwater infiltration system is likely not feasible for this development. It is suggested that stormwater be retained, filtered and discharged in accordance with the requirements of the local governing agency.

7.25 Surface Drainage

7.25.1 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.

7.25.2 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 five 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.

7.25.3 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.

7.25.4 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.

7.26 Plan Review

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

LIMITATIONS AND UNIFORMITY OF CONDITIONS

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

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

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

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

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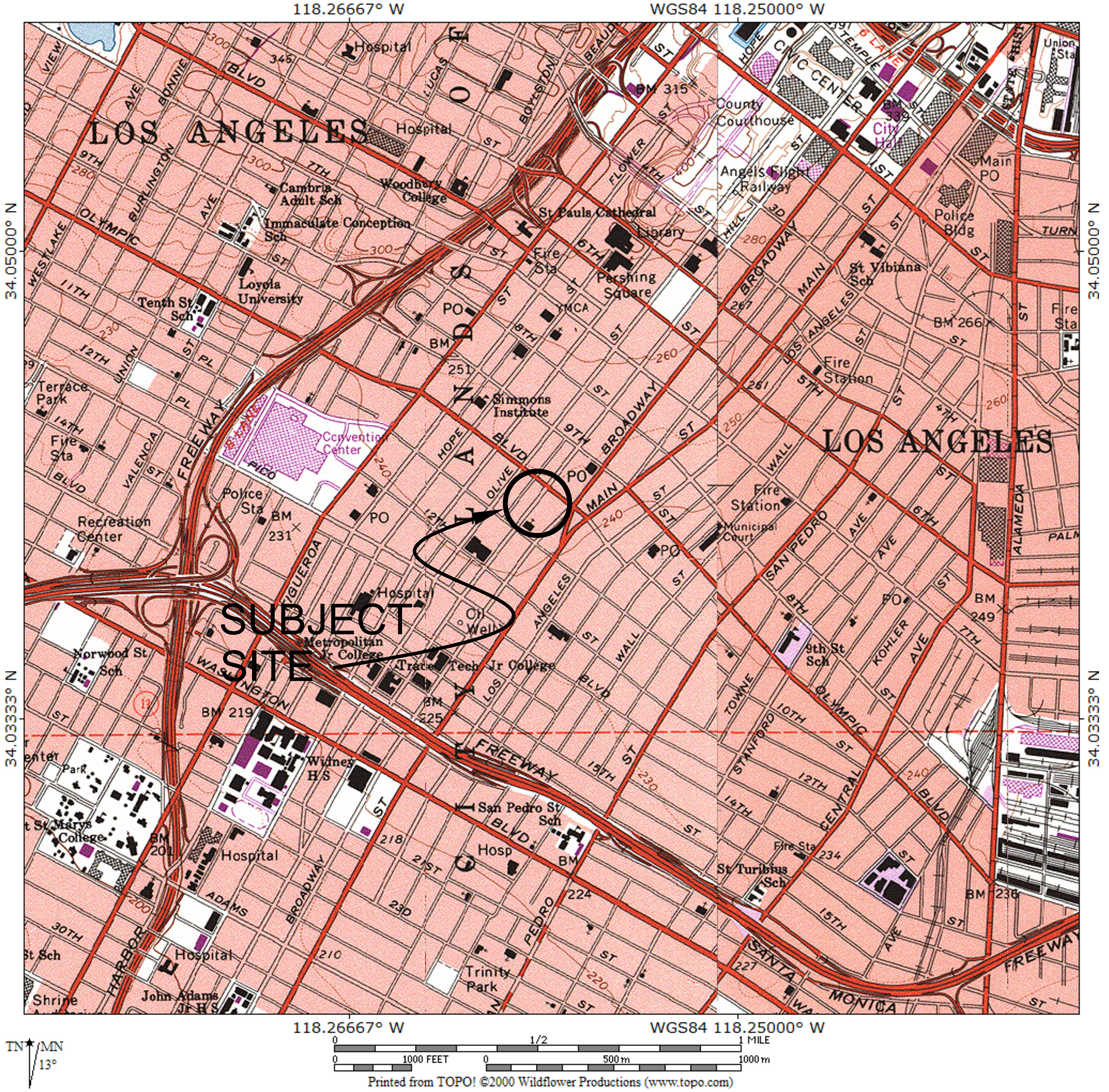

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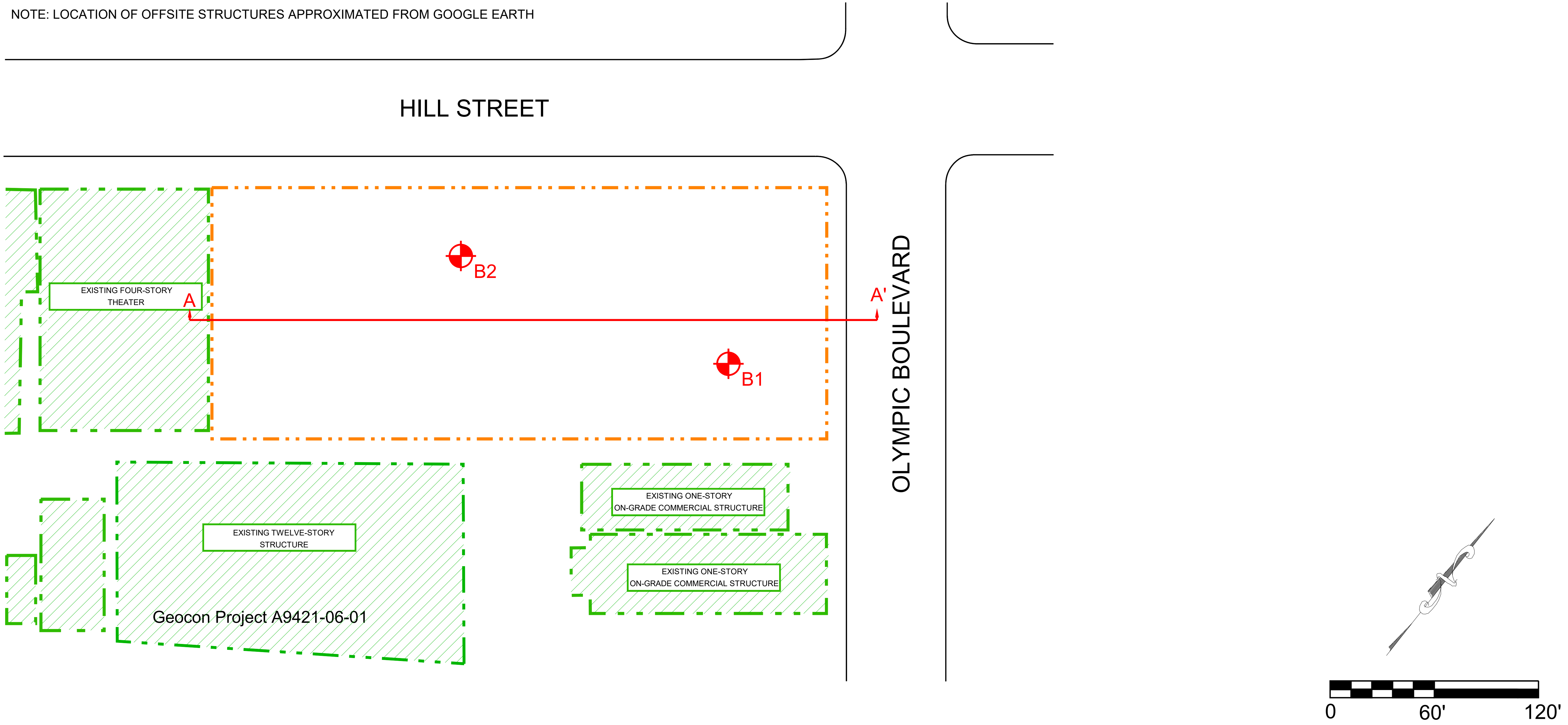
VICINITY MAP

ONNI CONTRACTING (CALIFORNIA) INC.
HILL STREET AND OLYMPIC BOULEVARD
LOS ANGELES, CALIFORNIA

FEB 2017	PROJECT NO. A9549-06-01	FIG. 1



NOTE: LOCATION OF OFFSITE STRUCTURES APPROXIMATED FROM GOOGLE EARTH

LEGEND

⊕ B2	Approximate Location of Boring
- · · —	Approximate Limits of Project Site
▨	Approximate Location of Existing Offsite Structures

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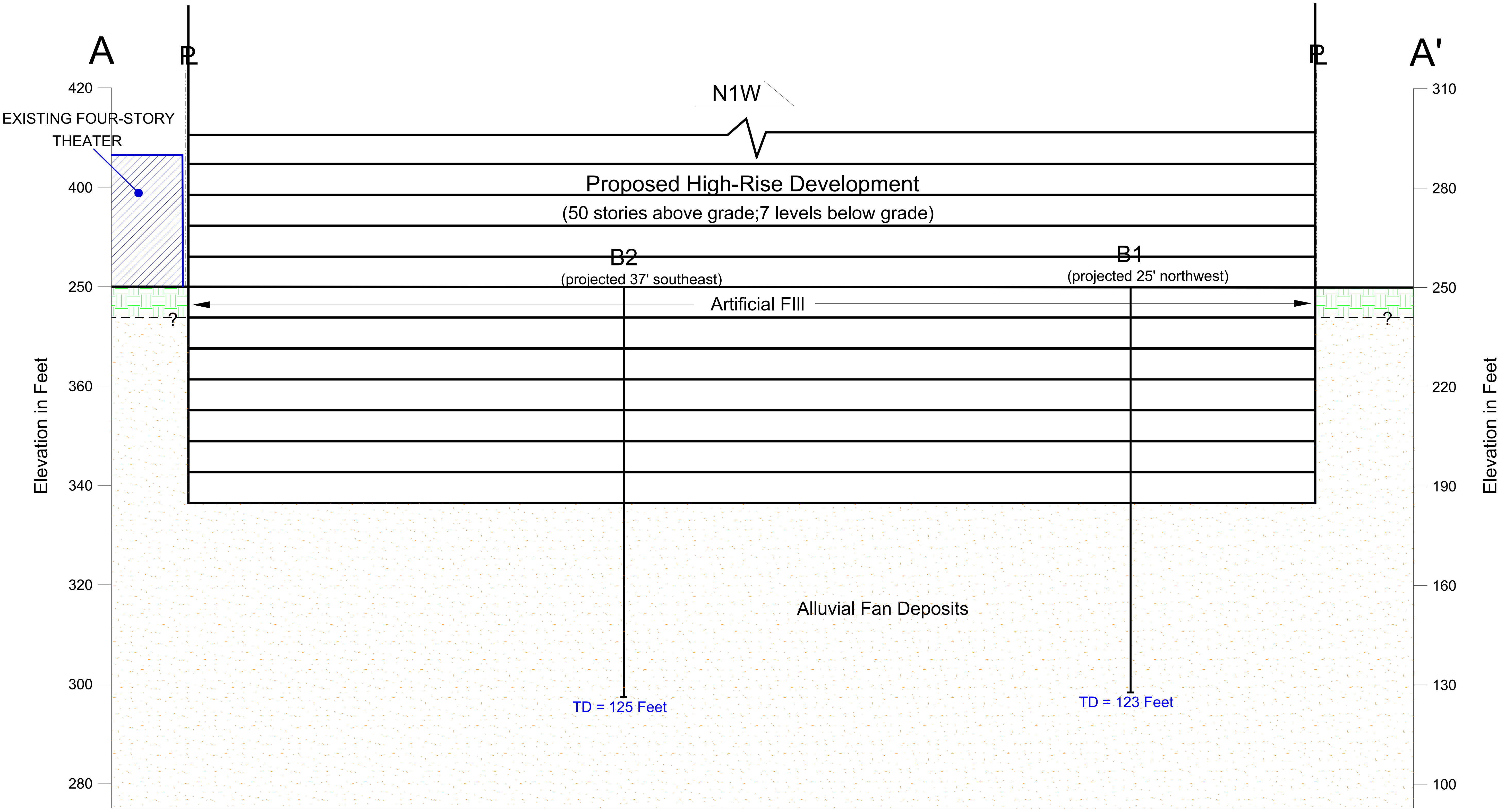
SITE PLAN

ONNI CONTRACTING (CALIFORNIA) INC.
HILL STREET AND OLYMPIC BOULEVARD
LOS ANGELES, CALIFORNIA

FEB 2017	PROJECT NO. A99549-06-01	FIG. 2



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PHONE (818) 841-8388 - FAX (818) 841-1704

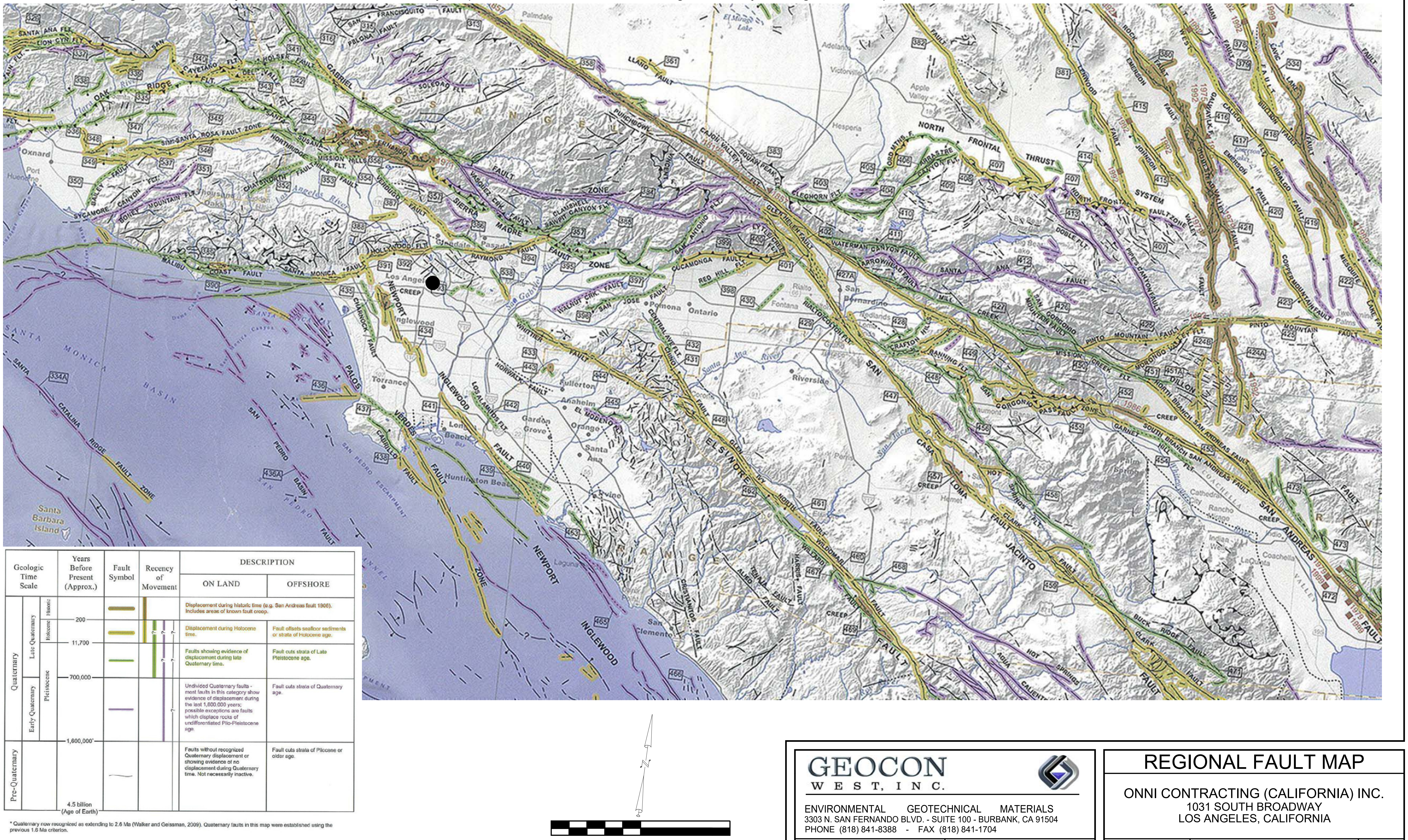
Drafted by: MDS
Checked by: SFK

GEOLOGIC SECTION A - A'

ONNI CONTRACTING (CALIFORNIA) INC.
HILL STREET AND OLYMPIC BOULEVARD
LOS ANGELES, CALIFORNIA

FEB 2017
PROJECT NO. A9549-06-01
FIG. 3

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



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

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

N

0 12 24 Miles

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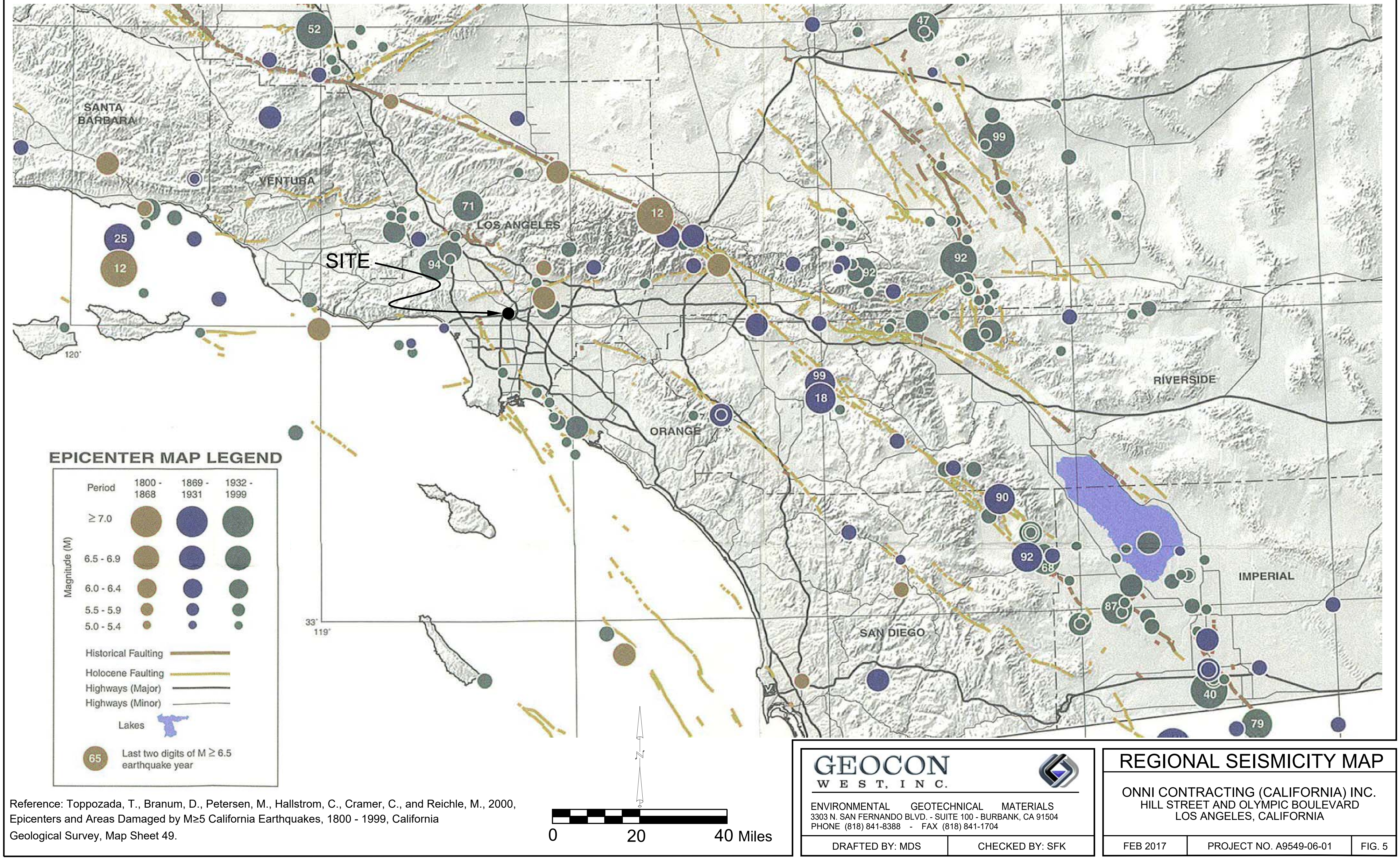
REGIONAL FAULT MAP

ONNI CONTRACTING (CALIFORNIA) INC.
1031 SOUTH BROADWAY
LOS ANGELES, CALIFORNIA

DRAFTED BY: MDS	CHECKED BY: SFK	FEB 2017	PROJECT NO. A9549-06-01	FIG. 4



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

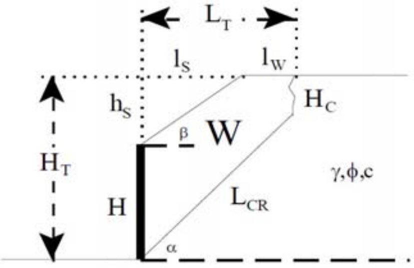
ONNI CONTRACTING (CALIFORNIA) INC.
HILL STREET AND OLYMPIC BOULEVARD
LOS ANGELES, CALIFORNIA

FEB 2017	PROJECT NO. A9549-06-01	FIG. 5

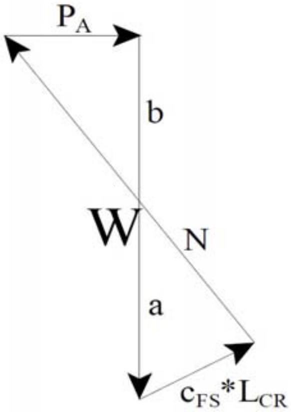
Retaining Wall Design with Transitioned Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	80.00	feet
Slope Angle of Backfill	(b)	0.0	degrees
Height of Slope above Wall	(h_s)	0.0	feet
Horizontal Length of Slope	(l_s)	0.0	feet
Total Height (Wall + Slope)	(H_T)	80.0	feet
Unit Weight of Retained Soils	(g)	115.0	pcf
Friction Angle of Retained Soils	(f)	37.0	degrees
Cohesion of Retained Soils	(c)	100.0	psf
Factor of Safety	(FS)	1.50	
Factored Parameters:	(f_{FS})	26.7	degrees
	(c_{FS})	66.7	psf



Failure Angle (a) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
45	2.3	3197	367687.9	109.8	20810.6	346877.2	114896.3
46	2.3	3088	355091.5	108.1	19454.8	335636.7	117712.3
47	2.2	2982	342909.2	106.4	18246.4	324662.8	120266.8
48	2.1	2879	331114.1	104.8	17164.2	313949.8	122570.9
49	2.1	2780	319681.6	103.2	16190.8	303490.8	124634.3
50	2.0	2683	308588.8	101.8	15311.7	293277.1	126465.6
51	2.0	2590	297814.6	100.4	14514.9	283299.7	128072.2
52	2.0	2499	287339.3	99.0	13790.3	273549.0	129460.6
53	1.9	2410	277144.7	97.7	13129.2	264015.5	130636.3
54	1.9	2324	267213.7	96.5	12524.4	254689.3	131603.9
55	1.9	2239	257530.5	95.3	11969.6	245561.0	132367.2
56	1.9	2157	248080.4	94.2	11459.4	236621.0	132929.2
57	1.9	2077	238849.5	93.1	10989.1	227860.3	133292.0
58	1.9	1998	229824.9	92.1	10554.8	219270.1	133457.0
59	1.9	1922	220994.5	91.1	10152.9	210841.6	133424.8
60	1.9	1846	212346.8	90.2	9780.2	202566.7	133195.3
61	1.9	1773	203871.3	89.3	9434.0	194437.3	132767.7
62	1.9	1701	195557.7	88.4	9111.9	186445.9	132140.3
63	1.9	1630	187396.7	87.6	8811.7	178584.9	131310.6
64	1.9	1560	179379.1	86.8	8531.7	170847.4	130275.5
65	2.0	1491	171496.5	86.1	8269.9	163226.6	129031.0
66	2.0	1424	163740.8	85.4	8025.0	155715.8	127572.0
67	2.0	1357	156104.3	84.7	7795.4	148308.9	125892.8
68	2.1	1292	148579.8	84.0	7580.1	140999.7	123986.6
69	2.1	1227	141160.2	83.4	7377.7	133782.5	121845.6
70	2.2	1164	133839.1	82.8	7187.4	126651.7	119460.9



Design Equations (Vector Analysis):

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

$b = W-a$

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

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\ max}$ 133457.0 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 41.7 pcf 61.3 pcf

Design Wall for an Equivalent Fluid Pressure: 42 pcf 62 pcf

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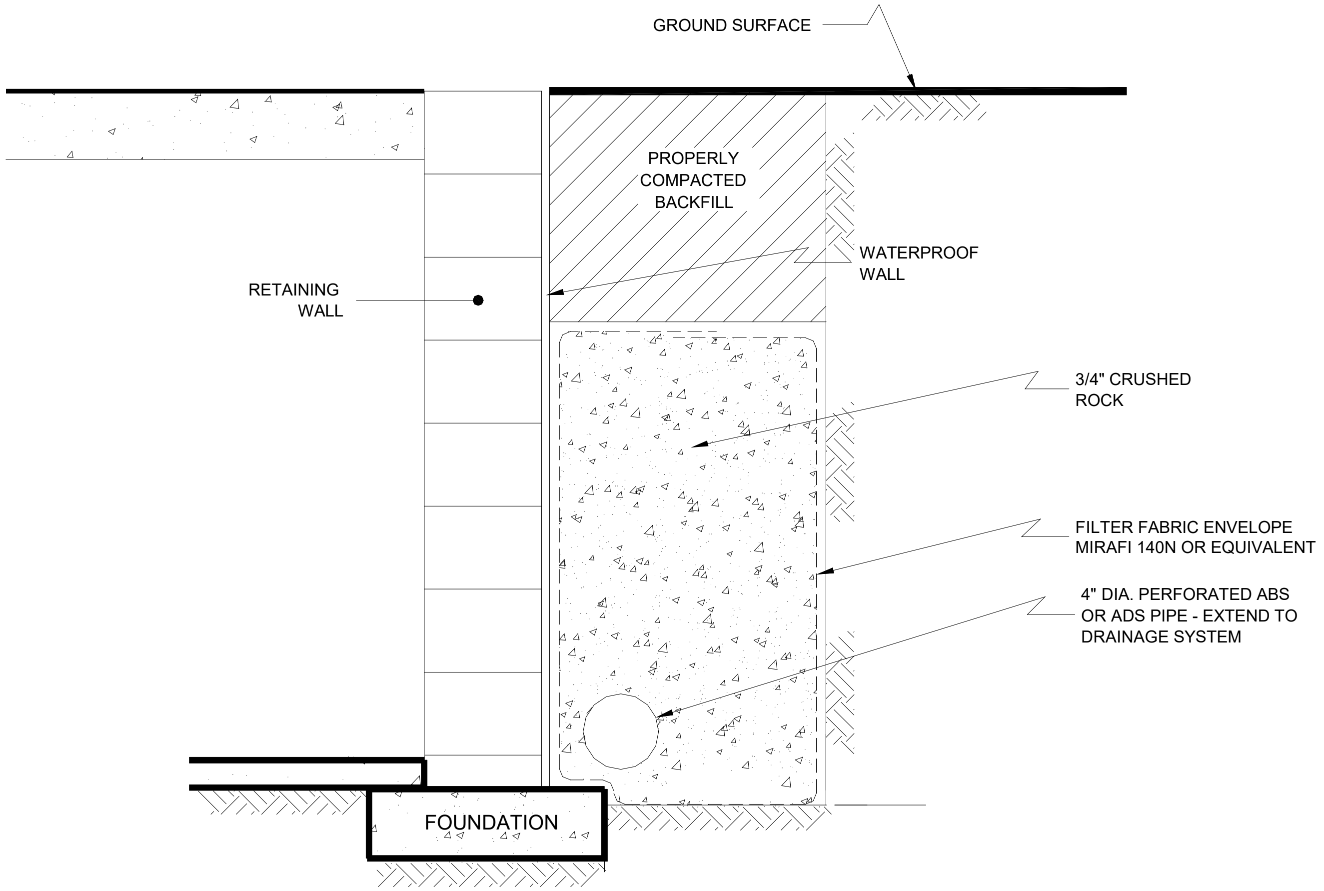
RETAINING WALL PRESSURE CALCULATION

ONNI CONTRACTING (CALIFORNIA) INC.
HILL STREET AND OLYMPIC BOULEVARD
LOS ANGELES, CALIFORNIA

FEB 2017
PROJECT NO. A9549-06-01
FIG. 6



NO SCALE

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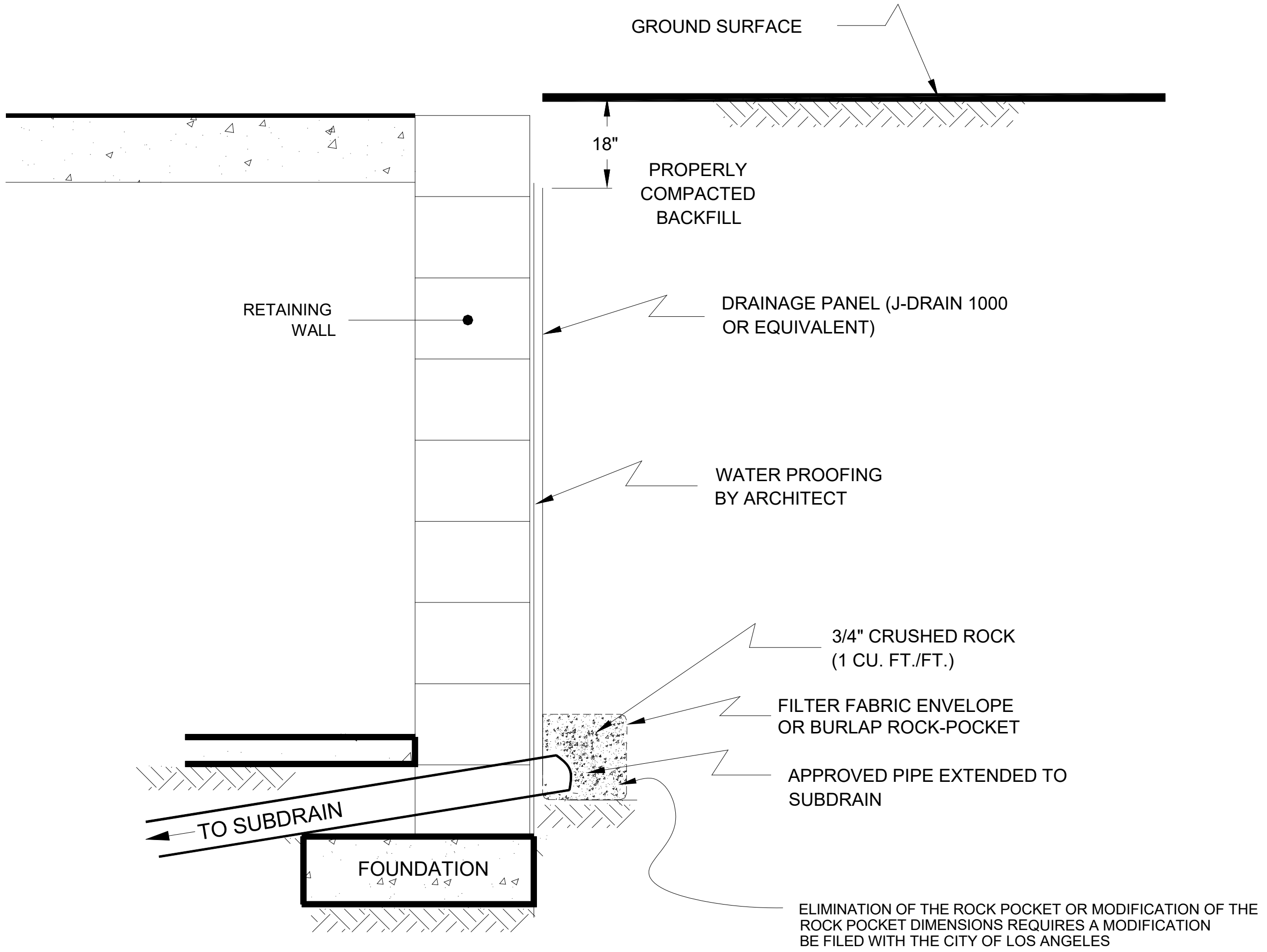
RETAINING WALL DRAIN DETAIL

ONNI CONTRACTING (CALIFORNIA) INC.
HILL STREET AND OLYMPIC BOULEVARD
LOS ANGELES, CALIFORNIA

DRAFTED BY: JTA	CHECKED BY: NDB	FEB 2017	PROJECT NO. A9549-06-01	FIG. 7



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RETAINING WALL DRAIN DETAIL

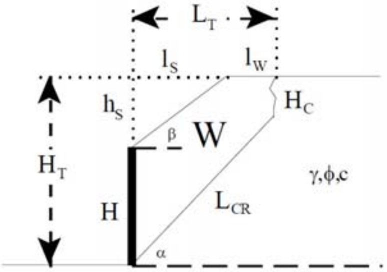
ONNI CONTRACTING (CALIFORNIA) INC.
HILL STREET AND OLYMPIC BOULEVARD
LOS ANGELES, CALIFORNIA

FEB 2017 | PROJECT NO. A9549-06-01 | FIG. 8

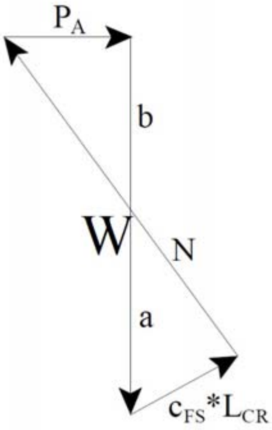
Shoring Design with Transitioned Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	85.00	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)	85.0	feet
Unit Weight of Retained Soils	(γ)	115.0	pcf
Friction Angle of Retained Soils	(ϕ)	37.0	degrees
Cohesion of Retained Soils	(c)	100.0	psf
Factor of Safety	(FS)	1.25	
Factored Parameters:	(ϕ_{FS})	31.1	degrees
	(c_{FS})	80.0	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	3.5	3606	414731.9	115.3	32832.1	381899.7	94628.1
46	3.3	3483	400566.9	113.5	30217.9	370349.1	98657.2
47	3.2	3364	386857.7	111.9	27948.0	358909.7	102350.6
48	3.1	3248	373576.9	110.3	25961.9	347615.0	105723.6
49	3.0	3137	360698.8	108.7	24212.1	336486.7	108790.0
50	2.9	3028	348199.1	107.2	22661.0	325538.1	111562.0
51	2.8	2922	336055.0	105.8	21278.6	314776.4	114050.7
52	2.7	2820	324245.3	104.4	20040.5	304204.8	116265.6
53	2.7	2720	312749.8	103.1	18926.5	293823.3	118215.2
54	2.6	2622	301550.0	101.8	17920.2	283629.7	119906.8
55	2.6	2527	290628.2	100.6	17007.7	273620.4	121346.8
56	2.5	2435	279968.1	99.5	16177.6	263790.6	122540.5
57	2.5	2344	269554.4	98.4	15419.8	254134.5	123492.2
58	2.5	2255	259372.6	97.3	14726.2	244646.3	124205.4
59	2.5	2169	249409.1	96.3	14089.6	235319.5	124682.6
60	2.5	2084	239651.4	95.3	13503.9	226147.5	124925.7
61	2.5	2001	230087.3	94.4	12963.6	217123.6	124935.4
62	2.5	1919	220705.5	93.5	12464.3	208241.3	124711.8
63	2.5	1839	211495.5	92.6	12001.8	199493.7	124254.1
64	2.5	1760	202447.0	91.8	11572.6	190874.4	123560.7
65	2.5	1683	193550.5	91.0	11173.6	182377.0	122629.0
66	2.6	1607	184797.0	90.2	10801.9	173995.1	121455.7
67	2.6	1532	176177.9	89.5	10455.2	165722.6	120036.4
68	2.6	1458	167684.8	88.8	10131.2	157553.6	118366.0
69	2.7	1385	159310.0	88.1	9828.0	149482.0	116438.2
70	2.8	1313	151046.0	87.5	9543.6	141502.3	114245.7



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

$P_{A,\ max}$ 124935.41 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 34.6 pcf 53.5 pcf

Design Shoring for an Equivalent Fluid Pressure: 35 pcf 54 pcf

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SHORING PRESSURE CALCULATION

ONNI CONTRACTING (CALIFORNIA) INC.
HILL STREET AND OLYMPIC BOULEVARD
LOS ANGELES, CALIFORNIA

FEB 2017 PROJECT NO. A9549-06-01 FIG. 9

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

The site was explored on January 20 and 23, 2017, by excavating two exploratory borings using a truck-mounted hollow-stem auger drilling machine to depths of 123 and 125 feet below existing ground surface. Representative and relatively undisturbed samples were obtained by driving a 3-inch O. D., California Modified Sampler into the "undisturbed" soil mass with blows from a 140-pound auto-hammer falling 30 inches (auto-hammer). 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. Standard Penetration Tests were performed in Boring B2.

The soil conditions encountered in the test pits were visually examined, classified, and logged in general accordance with the Unified Soil Classification System (USCS). Logs of the test pits 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 locations of the borings are shown on Figure 2.

PROJECT NO. A9549-06-01

BORING 1

ELEV. (MSL.) **- -** DATE COMPLETED **1/20/17**

EQUIPMENT **HOLLOW STEM AUGER** BY: **MDS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**AC: 3" BASE: NONE** **ARTIFICIAL FILL** Silty Sand with Gravel, loose, slightly moist, light yellowish brown, fine- to coarse-grained, gravel (to 3"), abundant brick fragments.			
2								
4								
6	B1@5.5' ■				Sandy Silt, stiff, slightly moist, brown, fine- to medium-grained, brick fragments.	30	101.4	18.2
8								
10	B1@10' ■				**ALLUVIUM** Sand with Silt, poorly graded, very dense, slightly moist, light brown, fine- to medium-grained, trace gravel (to 2.5").	50 (4")	97.5	4.2
12								
14				SP-SM				
16	B1@15' ■				- increase in silt content	50 (5")	104.2	16.8
18								
20	B1@20' ■				Sand with Gravel, poorly graded, very dense, slightly moist, grayish brown, fine- to medium-grained, some coarse-grained, gravel (to 3").	--	92.5	1.6
22								
24				SP				
26								
28								

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

A9549-06-01 BORING LOGS.GPJ

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

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

PROJECT NO. A9549-06-01

BORING 1

ELEV. (MSL.) **- -** DATE COMPLETED **1/20/17**

EQUIPMENT **HOLLOW STEM AUGER** BY: **MDS**

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' ■					50 (6")	100.9	15.4
32								
				SP				
34								
36								
					Silty Sand with Gravel, very dense, slightly moist, fine- to coarse-grained, gravel (to 3").			
38								
40	B1@40' ■					50 (5")	108.6	5.8
42								
				SM				
44								
46								
48								
50	B1@50' ■				Sand, poorly graded, very dense,s lightly moist, yellowish brown, fine- to medium-grained, some coarse-grained, trace fine gravel.	50 (6")	99.5	19.8
52								
54								
				SP				
56								
58								

A9549-06-01 BORING LOGS.GPJ

Figure A1,
Log of Boring 1, Page 2 of 5

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL

◧ ... STANDARD PENETRATION TEST

■ ... DRIVE SAMPLE (UNDISTURBED)

☒ ... DISTURBED OR BAG SAMPLE

◪ ... CHUNK SAMPLE

▼ ... WATER TABLE OR SEEPAGE

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

GEOCON

BORING 1

ELEV. (MSL.) - - DATE COMPLETED 1/20/17

EQUIPMENT **HOLLOW STEM AUGER** BY: **MDS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
60	B1@60' ■			SW	Sand with Gravel, well-graded, very dense, slightly moist, grayish brown, fine- to coarse-grained, gravel (to 3").	50 (6")	110.3	4.2
62								
64								
66								
68								
70	B1@70' ■			SW	Sand, well-graded, very dense, slightly moist, grayish brown, fine- to coarse-grained, trace gravel.	50 (5.5")	90.0	18.9
72								
74								
75	B1@75' ■			SW	Sand with Gravel, well-graded, very dense, slightly moist, grayish brown, fine- to coarse-grained, gravel (to 3").	50 (5")	106.7	2.9
76								
78								
80	B1@80' ■					50 (6")	101.7	4.7
82								
84								
85	B1@85' ■					50 (6")	92.2	2.8
86								
88								

Figure A1,
Log of Boring 1, Page 3 of 5

A9549-06-01 BORING LOGS.GPJ

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

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

PROJECT NO. A9549-06-01

BORING 1

ELEV. (MSL.) **- -** DATE COMPLETED **1/20/17**

EQUIPMENT **HOLLOW STEM AUGER** BY: **MDS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
90	B1@90' ■					50 (4")	116.3	2.5
92				SW				
94								
	B1@95' ■				Silty Sand, very dense, slightly moist, yellowish brown, fine-grained.	50 (6")	104.6	20.1
96								
98								
100	B1@100' ■					50 (6")	106.6	3.8
102				SM				
104								
106								
108								
110	B1@110' ■				Sand, well-graded, very dense, slightly moist, light brown, fine- to coarse-grained.	50 (6")	99.6	5.9
112								
114				SW				
116								
118								

Figure A1,
Log of Boring 1, Page 4 of 5

A9549-06-01 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

PROJECT NO. A9549-06-01

BORING 1

ELEV. (MSL.) - - DATE COMPLETED 1/20/17

EQUIPMENT **HOLLOW STEM AUGER** BY: **MDS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
120	B1@120' ■	⁙		SP	Sand, poorly graded, very dense, slightly moist, grayish brown, fine- to medium-granied, some coarse-grained, trace silt, some gravel (to 3").	50 (5")	116.1	3.6
122		⁙						
					Total depth of boring: refusal at 123 feet Fill to 10 feet. No groundwater encountered. Backfilled with soil cuttings and tamped. Asphalt patched. *Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.			

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

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. A9549-06-01

BORING 2

ELEV. (MSL.) **- -** DATE COMPLETED **1/23/17**

EQUIPMENT **HOLLOW STEM AUGER** BY: **MDS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL** Silty Sand with Gravel, medium dense, slightly moist, brown to light brown, fine- to medium-grained, fine gravel (to 4"), abundant debris.			
2								
					Sandy Silt, stiff, slightly moist, dark brown, fine-grained, abundant debris.			
4								
6	B2@5.5' ■					23	92.2	14.3
8								
10	B2@10' ■				**ALLUVIUM** Silty Sand with Gravel, very dense, slightly moist, yellowish brown, fine- to medium-grained, trace coarse-grained, gravel (to 1.5").	50 (5")	135.0	5.5
12								
14								
16	B2@15' ■				- dark yellowish brown, gravel (to 2.5")	50 (6")	118.1	5.0
18								
20	B2@20' ■			SM	- increase in gravel content	50 (6")	115.2	0.9
22								
24								
26								
28								

Figure A2,
Log of Boring 2, Page 1 of 5

A9549-06-01 BORING LOGS.GPJ

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

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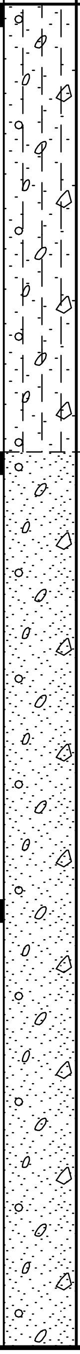
GEOCON

BORING 2

ELEV. (MSL.) - - DATE COMPLETED 1/23/17

EQUIPMENT **HOLLOW STEM AUGER** BY: **MDS**



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' ■					50 (6")	102.5	15.1
32								
					- abundant rig chatter			
34								
				SM				
36								
38								
40	B2@40' ■				Sand with Gravel, well-graded, very dense, (moisture), brown to reddish brown, fine- to coarse-grained, gravel (to 2.5").	50 (3.5")	110.3	1.3
42								
44								
46								
48								
				SW				
50	B2@50' ■				- cobble in sampler	76	112.4	7.9
					- dense			
52								
54								
56								
58								

Figure A2,
Log of Boring 2, Page 2 of 5

A9549-06-01 BORING LOGS.GPJ

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
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BORING 2

ELEV. (MSL.) - - DATE COMPLETED 1/23/17

EQUIPMENT **HOLLOW STEM AUGER** BY: **MDS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
60	B2@60'				- very dense, rock in sampler	50 (5.5")	133.0	3.4
62								
64								
66								
68								
70	B2@70'					50 (2.5")	128.4	3.0
72				SW				
	B2@72.5'					--	--	--
74								
	B2@75'					50 (6")	--	2.3
76								
	B2@77.5'					50 (3")	--	--
78								
80	B2@80'					50 (6")	--	1.9
82	B2@82'					--	--	--
84					Sand, poorly graded, very dense, slightly moist, light grayish brown, fine- to medium-grained, trace coarse-grained, trace fine gravel (to .75").			
	B2@85'					50 (4")	--	3.7
86								
				SP				
88	B2@87.5'					50 (5")	103.1	5.7

Figure A2,
Log of Boring 2, Page 3 of 5

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
⊠ ... DISTURBED OR BAG SAMPLE
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PROJECT NO. A9549-06-01

BORING 2

ELEV. (MSL.) **- -** DATE COMPLETED **1/23/17**

EQUIPMENT **HOLLOW STEM AUGER** BY: **MDS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
90	B2@90'					53	--	6.0
92				SP				
94					Silty Sand, very dense, slightly moist, yellowish brown, fine-grained.			
	B2@95'					50 (4")	109.4	16.7
96				SM				
98								
100	B2@100'				Sand, well-graded, very dense, slightly moist, grayish brown, fine- to coarse-grained.	50 (6")	--	4.1
102								
104				SW				
	B2@105'					50 (5.5")	105.8	7.0
106								
108								
					Sandy Silt, hard, slightly moist, brown, fine-grained, oxidation staining.			
110	B2@110'					32	--	21.6
112								
114				ML				
	B2@115'				- trace gravel (to 2.5")	50 (4.5")	110.3	16.0
116								
118								

Figure A2,
Log of Boring 2, Page 4 of 5

A9549-06-01 BORING LOGS.GPJ

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

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GEOCON

PROJECT NO. A9549-06-01

BORING 2

ELEV. (MSL.) **- -** DATE COMPLETED **1/23/17**

EQUIPMENT **HOLLOW STEM AUGER** BY: **MDS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
120	B2@120' ■				Sand with Gravel, well-graded, very dense, slightly moist, light gray, fine- to coarse-grained, gravel (to 1.5").	50 (6")	--	2.6
122				SW				
124								
	B2@125' ■					50 (3")	109.2	3.2

Total depth of boring: 125 feet
Fill 10 feet.
No groundwater encountered.
Prepared for percolation testing.
Backfilled with soil cuttings and tamped.

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

Figure A2,
Log of Boring 2, Page 5 of 5

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

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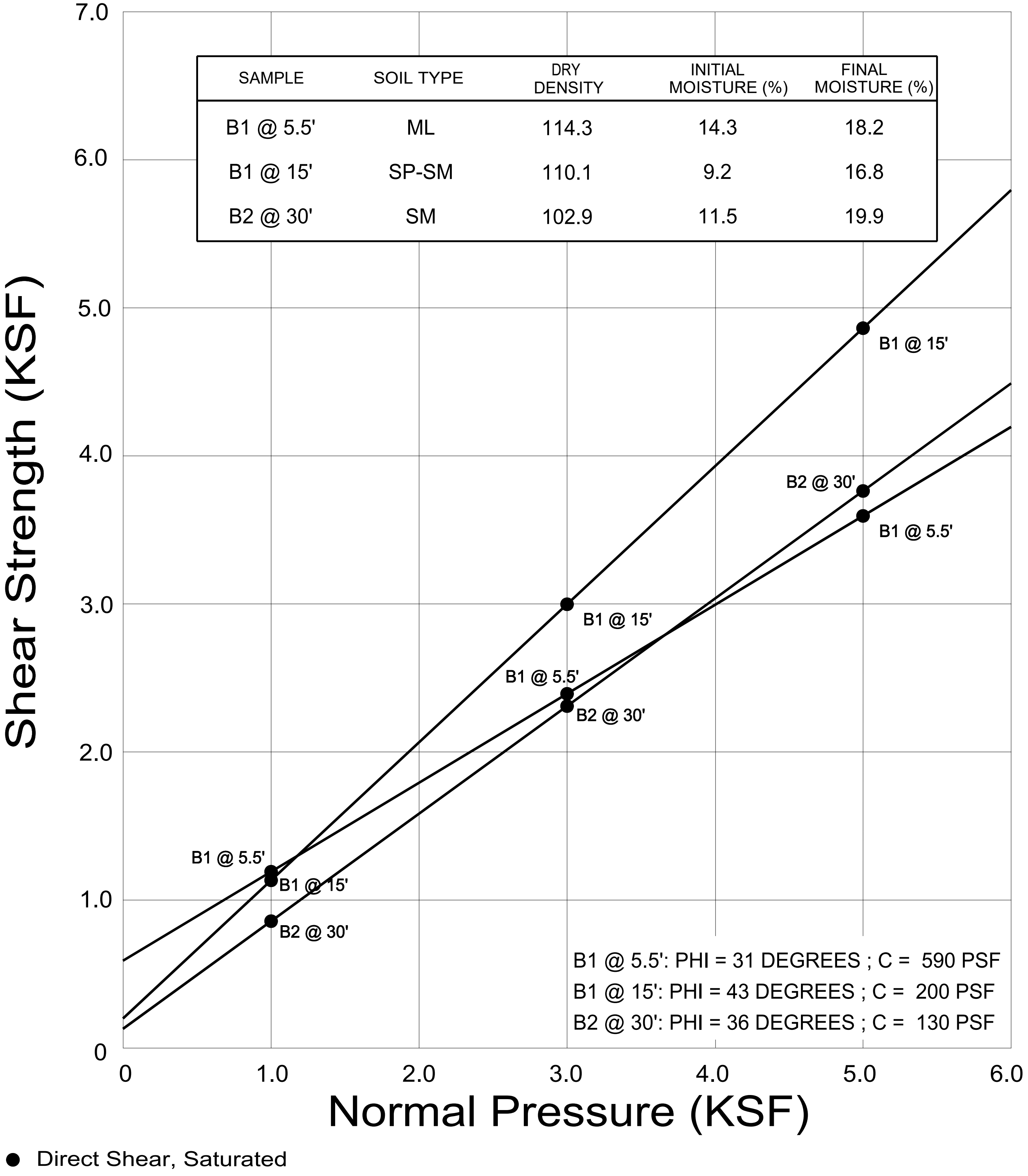
APPENDIX B

APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the "American Society for Testing and Materials (ASTM)", or other suggested procedures. Selected samples were tested for direct shear strength, consolidation, corrosivity, 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 logs in Appendix A.



SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1 @ 5.5'	ML	114.3	14.3	18.2
B1 @ 15'	SP-SM	110.1	9.2	16.8
B2 @ 30'	SM	102.9	11.5	19.9

B1 @ 5.5': PHI = 31 DEGREES ; C = 590 PSF
B1 @ 15': PHI = 43 DEGREES ; C = 200 PSF
B2 @ 30': PHI = 36 DEGREES ; C = 130 PSF

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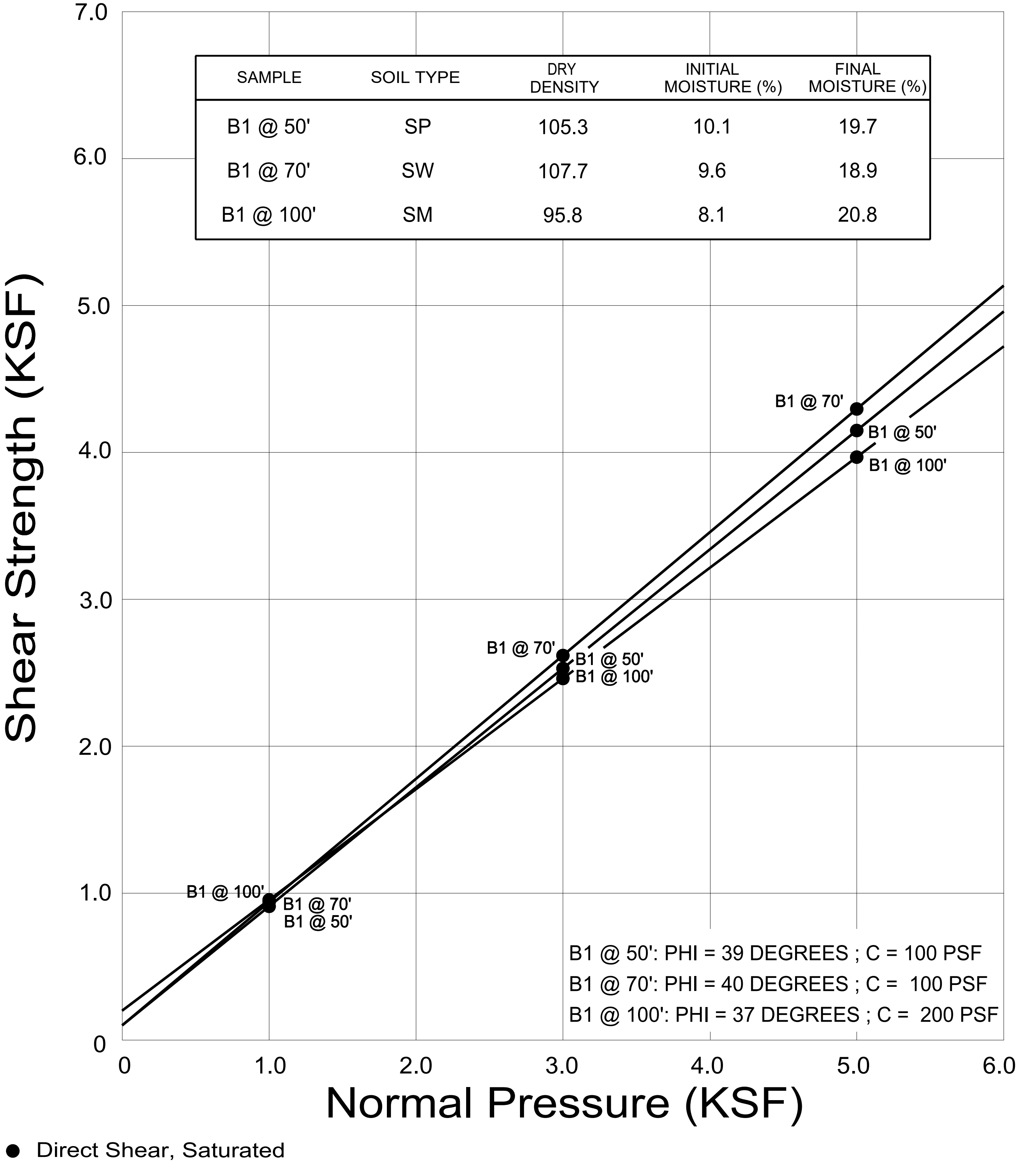
DIRECT SHEAR TEST RESULTS

ONNI CONTRACTING (CALIFORNIA) INC.
HILL STREET AND OLYMPIC BOULEVARD
LOS ANGELES, CALIFORNIA

DRAFTED BY: JTA	CHECKED BY: NDB	FEB 2017	PROJECT NO. A9549-06-01	FIG. B1



SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1 @ 50'	SP	105.3	10.1	19.7
B1 @ 70'	SW	107.7	9.6	18.9
B1 @ 100'	SM	95.8	8.1	20.8

B1 @ 50': PHI = 39 DEGREES ; C = 100 PSF
B1 @ 70': PHI = 40 DEGREES ; C = 100 PSF
B1 @ 100': PHI = 37 DEGREES ; C = 200 PSF

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DIRECT SHEAR TEST RESULTS

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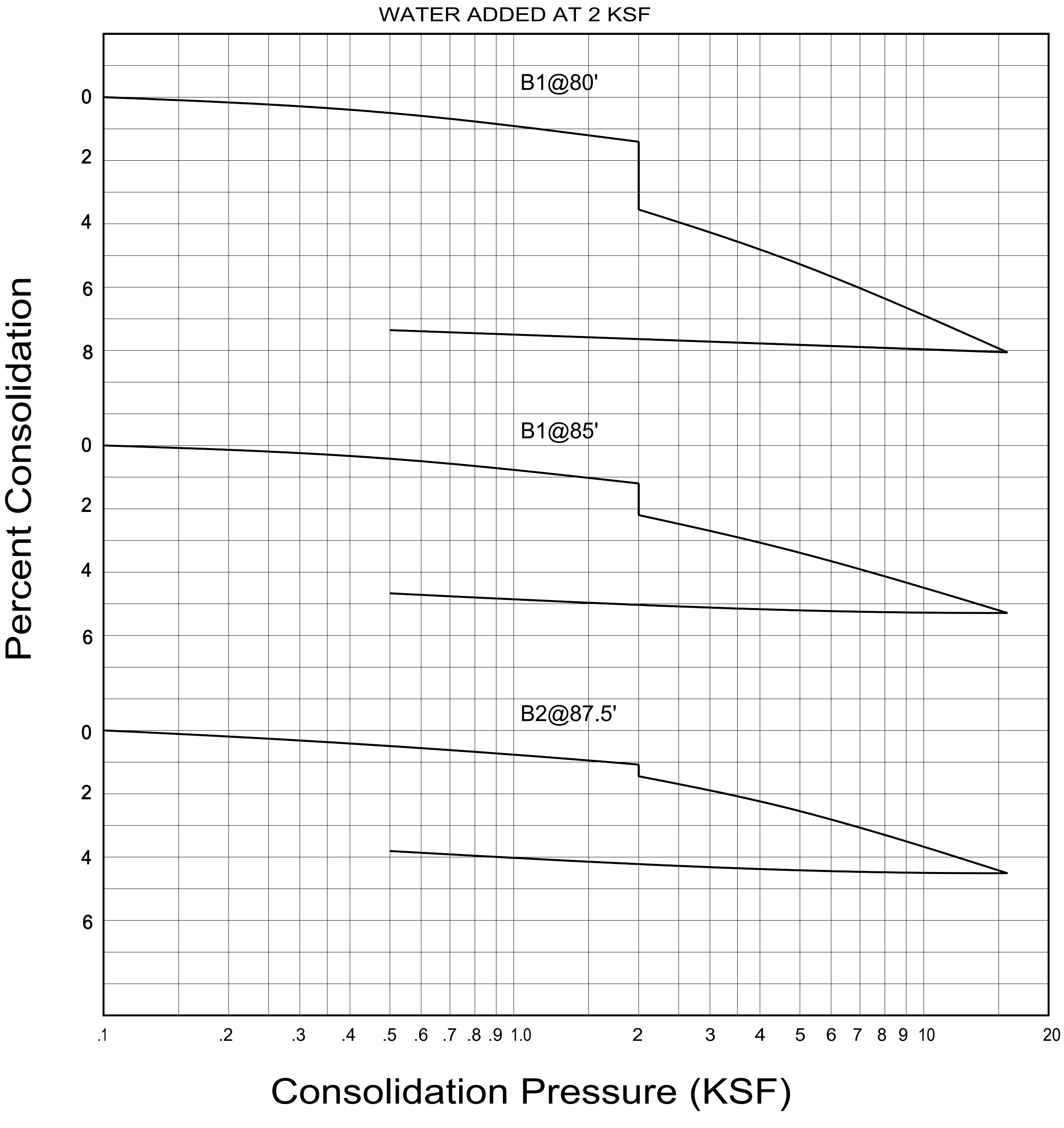
DRAFTED BY: JTA	CHECKED BY: NDB	FEB 2017	PROJECT NO. A9549-06-01	FIG. B2



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CONSOLIDATION TEST RESULTS

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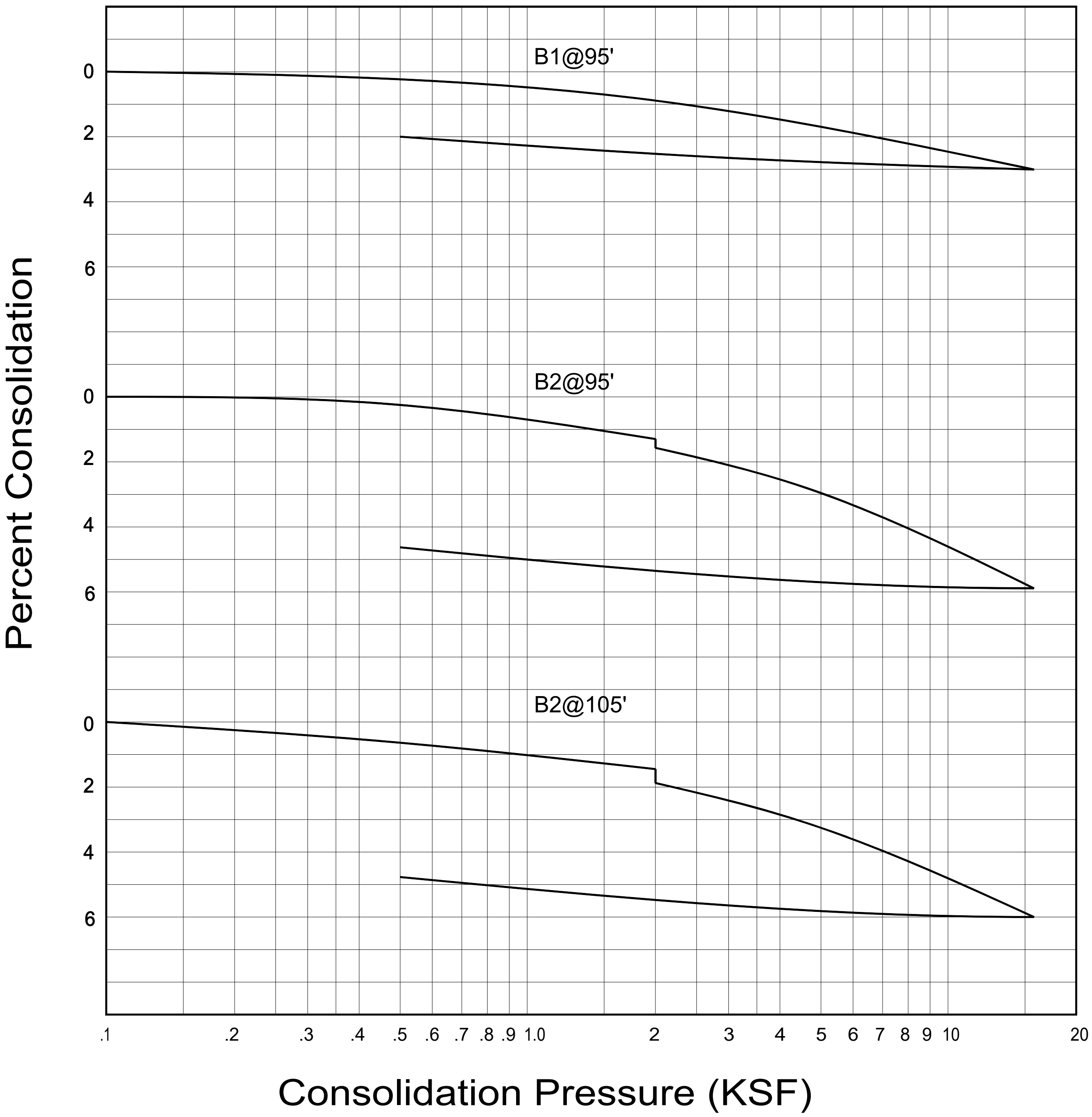
LOS ANGELES, CALIFORNIA

Drafted by: JTA	Checked by: NDB	FEB 2017	PROJECT NO. A9549-06-01	FIG. B3

WATER ADDED AT 2 KSF



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CONSOLIDATION TEST RESULTS

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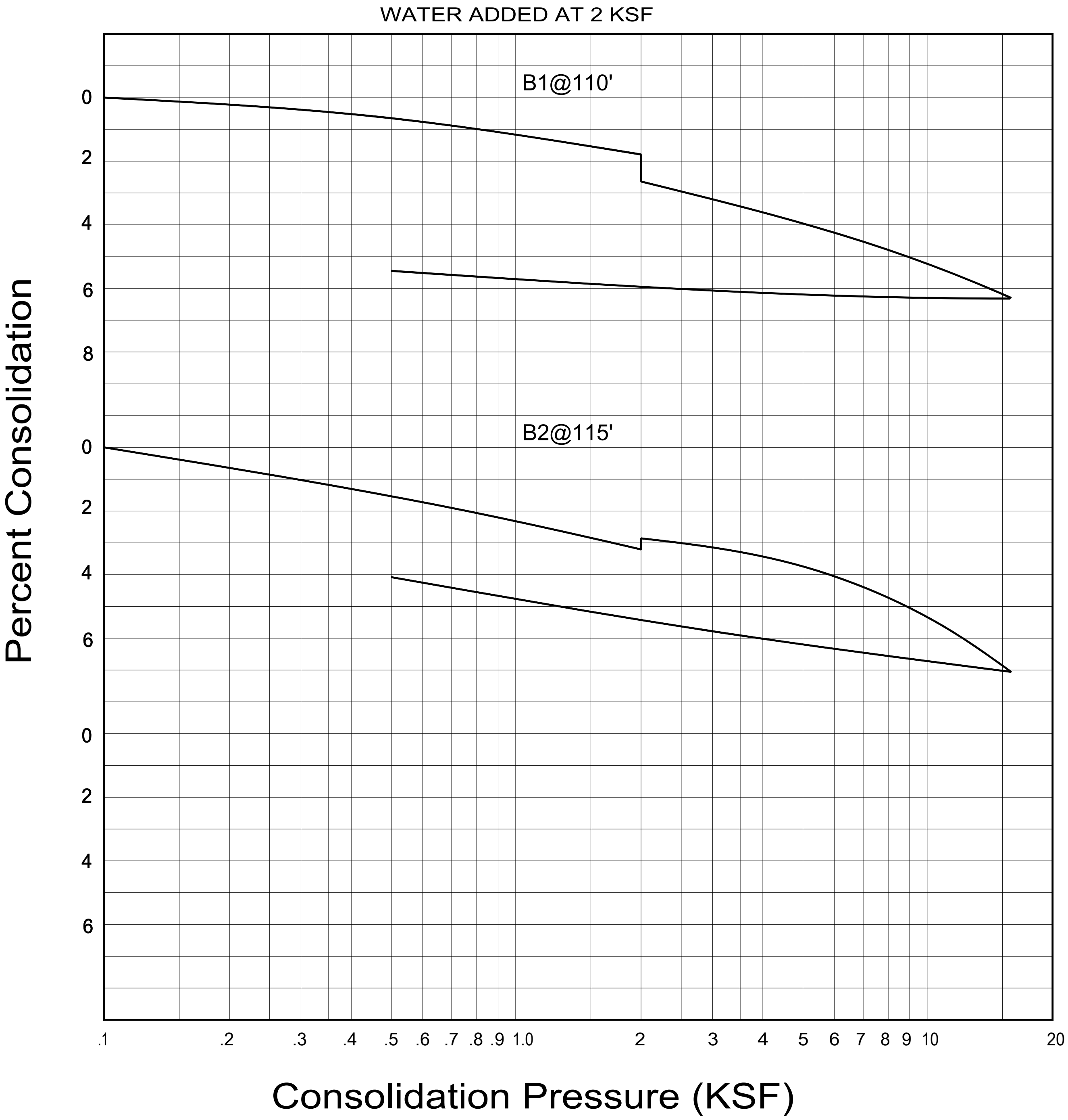
Drafted by: JTA	Checked by: NDB	FEB 2017	PROJECT NO. A9549-06-01	FIG. B4



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CONSOLIDATION TEST RESULTS

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FEB 2017 | PROJECT NO. A9549-06-01 | FIG. B5

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

Sample No.	**pH**	**Resistivity (ohm centimeters)**
B2 @ 77.5'	8.15	25,000 (Mildly Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS
EPA NO. 325.3

Sample No.	Chloride Ion Content (%)
B2 @ 77.5'	0.001

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

Sample No.	Water Soluble Sulfate (% SO_4)	Sulfate Exposure*
B2 @ 77.5'	0.007	Negligible

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

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CORROSIVITY TEST RESULTS

ONNI CONTRACTING (CALIFORNIA) INC.
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FEB 2017 PROJECT NO. A9549-06-01 FIG. B6