APPENDIX F
GEOTECHNICAL REPORT

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GEOTECHNICAL INVESTIGATION

PROPOSED MULTI-FAMILY RESIDENTIAL DEVELOPMENT
1625 SOUTH MAGNOLIA AVENUE
MONROVIA, CALIFORNIA

GEOCON
W E S T, I N C.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

TRAMMELL CROW RESIDENTIAL
CARLSBAD, CALIFORNIA

PROJECT NO. A9621-06-01

JULY 12, 2017

GEOCON
WEST, INC.
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. A9621-06-01
July 12, 2017

Trammell Crow Residential
5790 Fleet Street, Suite 140
Carlsbad, California 92008

Attention: Mr. Bryce Tabb

Subject: GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-FAMILY RESIDENTIAL DEVELOPMENT
1625 SOUTH MAGNOLIA AVENUE
MONROVIA, CALIFORNIA

Dear Mr. Tabb:

In accordance with your authorization of our proposal dated March 23, 2017, we have performed a geotechnical investigation for the proposed multi-family residential development located at 1625 South Magnolia Avenue in the City of Monrovia, California. The accompanying report presents the findings of our study, and our conclusions and recommendations pertaining to the geotechnical aspects of proposed design and construction. Based on the results of our investigation, it is our opinion that the site can be developed as proposed, provided the recommendations of this report are followed and implemented during design and construction.

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

Very truly yours,

GEOCON WEST, INC.

Arnold Gastelum
PE 81553

Jelisa Thomas Adams
GE 3092

Susan F. Kirkgard
CEG 1754

(Email) Addressee

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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a geotechnical investigation for the proposed multi-family residential development located at 1625 South Magnolia Avenue in the City of Monrovia, California (see Vicinity Map, Figure 1). The purpose of the investigation was to evaluate subsurface soil and geologic conditions underlying the site and, based on conditions encountered, to provide conclusions and recommendations pertaining to the geotechnical aspects of design and construction.

The scope of this investigation included a site reconnaissance, field exploration, laboratory testing, engineering analysis, and the preparation of this report. The site was explored on June 12, 2017, by excavating six 8-inch diameter borings to depths of approximately 25½ feet below the existing ground surface utilizing a truck-mounted hollow-stem auger drilling machine. The approximate locations of the exploratory borings are depicted on the Site Plan (see Figure 2). A detailed discussion of the field investigation, including boring logs, is presented in Appendix A.

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

The recommendations presented herein are based on analysis of the data obtained during the investigation and our experience with similar soil and geologic conditions. References reviewed to prepare this report are provided in the *List of References* section.

If project details vary significantly from those described above, 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 1625 South Magnolia Avenue in the City of Monrovia, California. The site consists of a 5.73-acre irregularly shaped parcel which is currently an automobile distribution lot. The site is occupied by four existing commercial/industrial structures, paved surface parking lots, and associated hardscape and landscaping. Two single-story residential structures also occupy a portion of the site adjacent to Evergreen Avenue. The site is bounded by Magnolia Avenue to the east, by a railroad right-of-way to the south, by residential and industrial structures to the west, by Evergreen Avenue to the north and northwest, and by commercial structures and associated surface parking lots to the east. The site is relatively level, with no pronounced highs or lows. The topography at the site and in the general site vicinity slopes downward toward the south and southwest. Surface water drainage at the site appears to be by sheet flow along the existing ground contours to the city streets. Vegetation consists of some isolated trees and sparse grass and shrubbery in isolated planter areas.

Based on the information provided by the Client, it is our understanding that the proposed project will consist of a Type III wrap development including five-story residential buildings and a seven-level parking structure. The development will include approximately 436 residential housing units. It is our further understanding that the proposed structures will be constructed at or near present site grade. The footprints of the proposed structures are shown on Figure 2 (Site Plan).

Due to preliminary nature of the design at this time, wall and column loads were not available. It is anticipated that column loads for the proposed residential buildings will be up to 200 kips, and wall loads will be up to 5 kips per linear foot. It is anticipated that column loads for the proposed parking structure will be up to 800 kips, and wall loads will be up to 10 kips per linear foot.

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

3. GEOLOGIC SETTING

The site is located in the northeastern San Gabriel Valley. The San Gabriel Valley is an alluvial filled valley bounded by the Sierra Madre Fault Zone and San Gabriel Mountains on the north, by the Puente Hills on the south, by the Covina and Indian Hills on the east, and by the Raymond Basin on the west. The alluvial deposits are derived from erosion of the San Gabriel Mountains to the north and subsequent deposition by the San Gabriel River and other local drainages. The alluvium is estimated to be approximately 200 feet thick at the base of the mountains, extending to hundreds of feet thick in the central portion of the valley.

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 Whittier Fault located approximately 7.2 miles to the southwest. The active Raymond Fault, located approximately 1.4 miles to the northwest of the site, forms the boundary between the Peninsular Ranges Geomorphic Province and the Transverse Ranges Geomorphic Province to the north.

4. SOIL AND GEOLOGIC CONDITIONS

Based on our field investigation and published geologic maps of the area, the site is underlain by artificial fill and Holocene are young alluvial fan deposits consisting of varying amounts of sand, silt, clay and gravel (California Geological Survey [CGS], 2010). Detailed stratigraphic profiles are provided on the boring logs in Appendix A.

4.1 Artificial Fill

Artificial fill was encountered in the exploratory borings to a maximum depth of 2½ feet below existing ground surface. The artificial fill generally consists of light yellowish brown, grayish brown and dark brown silty sand with lesser amounts of sandy silt and varying amounts of fine gravel and trace roots. The fill is characterized as moist and loose to medium dense. The fill is likely the result of past grading or construction activities at the site. Deeper fill may exist between excavations and in other portions of the site that were not directly explored.

4.2 Alluvium

Holocene age young alluvial deposits were encountered beneath the artificial fill and consist primarily of light gray, yellowish brown, dark brown, and reddish brown interbedded silty sand, poorly graded and well-graded sand, sandy silt, and silt with varying amounts of fine gravel. The soil is characterized as slightly moist to moist and loose to very dense or firm.

5. GROUNDWATER

Based on a review of the Seismic Hazard Evaluation of the Mount Wilson Quadrangle, Los Angeles County, California (California Division of Mines and Geology [CDMG], 1998), the historically highest groundwater level in the area is approximately 145 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.

Considering the historic high groundwater level (CDMG, 1998), the lack of groundwater encountered in our borings, and the depth of the proposed construction, it is unlikely that groundwater will be encountered during construction. However, it is not uncommon for groundwater levels to vary seasonally or for groundwater seepage conditions to develop where none previously existed, especially in impermeable fine-grained soils which are heavily irrigated or after seasonal rainfall. In addition, recent requirements for stormwater infiltration could result in shallower seepage conditions in the immediate site vicinity. Proper surface drainage of irrigation and precipitation will be critical for future performance of the project. Recommendations for drainage are provided in the S*urface Drainage* section of this report (see Section 7.21).

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 (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 currently established State of California Alquist-Priolo Earthquake Fault Zone for surface fault rupture hazards (CGS, 2017). In addition, the site is not located within the Raymond Hill Fault Zone as defined by the City of Monrovia Safety Element of the General Plan (2002). No active or potentially active faults with the potential for surface fault rupture are known to pass directly beneath the site. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed development is considered low. However, the site is located in the seismically active Southern California region, and could be subjected to moderate to strong ground shaking in the event of an earthquake on one of the many active Southern California faults. The faults in the vicinity of the site are shown in Figure 3, Regional Fault Map.

The closest active fault to the site is the Raymond Fault located approximately 1.4 miles to the northwest (CDMG, 2007). Other nearby active faults are the Duarte Fault, the Sierra Madre Fault, the Verdugo Fault, the Whittier Fault, the Hollywood Fault, and the Cucamonga Fault, located approximately 1.6 miles northeast, 2.1 miles north-northeast, 7.1 miles west, 7.2 miles southwest, 13½ miles west, and 19 miles east of the site, respectively. (Ziony and Jones, 1989). The active San Andreas Fault Zone is located approximately 23 miles north-northeast of the site.

The closest potentially active faults to the site are the San Jose Fault and the Indian Hill Fault located approximately 7.4 miles to the southeast and 7.5 miles to the southeast, respectively. The potentially active San Gabriel Fault located is located approximately 7.7 miles north of the site (Ziony and Jones, 1989).

Several buried thrust faults, commonly referred to as blind thrusts, underlie the Los Angeles Basin at depth. These faults are not exposed at the ground surface and are typically identified at depths greater than 3.0 kilometers. The October 1, 1987 M_w 5.9 Whittier Narrows earthquake and the January 17, 1994, M_w 6.7 Northridge earthquake were a result of movement on the Puente Hills Blind Thrust and the Northridge Thrust, respectively. These thrust faults and others in the Los Angeles area 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 4, Regional Seismicity Map. A partial list of moderate to major magnitude earthquakes that have occurred in the Southern California area within the last 100 years is included in the following table.

LIST OF HISTORIC EARTHQUAKES

Earthquake (Oldest to Youngest)	Date of Earthquake	Magnitude	Distance to Epicenter (Miles)	Direction to Epicenter
San Jacinto-Hemet area	April 21, 1918	6.8	63	ESE
Near Redlands	July 23, 1923	6.3	44	ESE
Long Beach	March 10, 1933	6.4	36	S
Tehachapi	July 21, 1952	7.5	83	NW
San Fernando	February 9, 1971	6.6	29	NW
Whittier Narrows	October 1, 1987	5.9	7	SW
Sierra Madre	June 28, 1991	5.8	9	N
Landers	June 28, 1992	7.3	90	E
Big Bear	June 28, 1992	6.4	68	E
Northridge	January 17, 1994	6.7	31	W
Hector Mine	October 16, 1999	7.1	104	ENE

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

6.3 Seismic Design Criteria

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

2016 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2016 CBC Reference
Site Class	D	Section 1613.3.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	2.157g	Figure 1613.3.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.834g	Figure 1613.3.1(2)
Site Coefficient, F_A	1.0	Table 1613.3.3(1)
Site Coefficient, F_V	1.5	Table 1613.3.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	2.157g	Section 1613.3.3 (Eqn 16-37)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	1.251g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.438g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.834g	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	0.819g	Figure 22-7
Site Coefficient, F_{PGA}	1.0	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.819g	Section 11.8.3 (Eqn 11.8-1)

The Maximum Considered Earthquake Ground Motion (MCE) is the level of ground motion that has a 2 percent chance of exceedance in 50 years, with a statistical return period of 2,475 years. According to the 2016 California Building Code and ASCE 7-10, the MCE is to be utilized for the evaluation of liquefaction, lateral spreading, seismic settlements, and it is our understanding that the intent of the Building code is to maintain “Life Safety” during a MCE event. The Design Earthquake Ground Motion (DE) is the level of ground motion that has a 10 percent chance of exceedance in 50 years, with a statistical return period of 475 years.

Deaggregation of the MCE peak ground acceleration was performed using the USGS online BETA Unified Hazard Tool, 2008 Conterminous U.S. Dynamic edition. The result of the deaggregation analysis indicates that the predominant earthquake contributing to the MCE peak ground acceleration is characterized as a 6.7 magnitude event occurring at a hypocentral distance of 6.87 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 6.7 magnitude occurring at a hypocentral distance of 10.44 kilometers from the site.

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

6.4 Liquefaction Potential

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

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

The Seismic Hazards Zone Map for the Mount Wilson 7.5 Minute Quadrangle (CGS, 2017; CDMG, 1999) indicates that the site is not located within a zone of required investigation for liquefaction. In addition, the County of Los Angeles Safety Element (Leighton, 1990), indicates that the site is not located in a liquefiable area. Groundwater was not encountered in our borings drilled to a maximum depth of 25½ feet beneath the existing ground surface and the historic high groundwater level in the area is reported to be approximately 145 feet beneath the existing ground surface (CDMG, 1998). Based on these considerations, it is our opinion that the potential for liquefaction and associated ground deformations beneath the site is very low.

6.5 Slope Stability

The site is relatively level and the topography in the site vicinity slopes downward toward the south and southwest. The County of Los Angeles Safety Element (Leighton, 1990), indicates that the site is not located in a hillside area. Also, the State of California Seismic Hazard Zone Map for the Mount Wilson Quadrangle (CGS, 2017; CDMG, 1999) indicates that the site is not located within a zone of required investigation for earthquake-induced landslides. 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 impact the site is considered low.

6.6 Earthquake-Induced Flooding

Earthquake-induced flooding is inundation caused by failure of dams or other water-retaining structures due to earthquakes. The City of Monrovia Safety Element (2002) and the Los Angeles County Safety Element (Leighton, 1990) indicates that the site is located within the potential inundation area for Big Santa Anita Dam, Sierra Madre Dam, and Saw Pit Dam. However, these reservoirs, as well as others in California, are continually monitored by various governmental agencies (such as the State of California Division of Safety of Dams and the U.S. Army Corps of Engineers) to guard against the threat of dam failure. Current design, construction practices, and ongoing programs of review, modification, or total reconstruction of existing dams are intended to ensure that all dams are capable of withstanding the maximum considered earthquake (MCE) for the site. Therefore, the potential for inundation at the site as a result of an earthquake-induced dam failure is considered low.

6.7 Tsunamis, Seiches, and Flooding

The site is not located within a coastal area. Therefore, tsunamis, 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 from a seismically-induced seiche is considered unlikely.

The majority of the site is within an area of minimal flooding (zone X) as defined by the Federal Emergency Management Agency, however the eastern portion of the site (adjacent to Magnolia Avenue) is within a flood zone D for possible but undetermined flood hazards (FEMA, 2017; LACDPW, 2017b).

6.8 Oil Fields & Methane Potential

Information on the California Division of Oil, Gas and Geothermal Resources (DOGGR) Well Finder Website (DOGGR, 2017) indicates the site is not located within the limits of an oilfield and oil or gas wells are not located within a mile of the site vicinity. 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 during construction will need to be properly abandoned in accordance with the current requirements of the DOGGR.

As previously indicated, the site is not located within an oilfield. Therefore, the potential for methane or other volatile gases to occur at the site is considered very low. However, should it be determined that a methane study is required for the proposed development it is recommended that a qualified methane consultant be retained to perform the study and provide mitigation measures as necessary.

6.9 Subsidence

Subsidence occurs when a large portion of land is displaced vertically, usually due to the withdrawal of groundwater, oil, or natural gas. Soils that are particularly subject to subsidence include those with high silt or clay content. The site is not located within an area of known ground subsidence. No large-scale extraction of groundwater, gas, oil, or geothermal energy is occurring or planned at the site or in the general site vicinity. There appears to be little or no potential for ground subsidence due to withdrawal of fluids or gases at the site.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

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

7.1.2 Up to 2½ feet of existing artificial fill was encountered during site exploration. The existing fill encountered is believed to be the result of past grading and construction activities at the site. Deeper fill may exist in other areas of the site that were not directly explored. It is our opinion that the existing fill, in its present condition, is not suitable for direct support of proposed foundations or slabs. The existing fill and site soils are suitable for re-use as engineered fill provided the recommendations in the Grading section of this report are followed (see Section 7.5).

7.1.3 As a minimum, the upper 5 feet of existing soils within the footprint areas of the proposed residential buildings and parking structure should be excavated and properly compacted for foundation and slab support. The engineered fill blanket should extend at least 5 feet beyond the edge of foundations or for a distance equal to the depth of fill below the foundations, whichever is greater. Where the recommended grading cannot be performed, such as adjacent to property lines, alternate grading and foundation design recommendations may be required. Proposed foundations should be underlain by at least 3 feet of newly compacted engineered fill. Foundations with an embedment greater than 2 feet will require deeper grading in order to maintain the required 3-foot-thick fill blanket beneath foundations. It is recommended that the grading contractor verify the depth of all building foundations prior to commencement of site grading activities in order to correctly determine the required excavation depth. Deeper fill or soft soils encountered during site grading operations should be excavated as necessary at the direction of the Geotechnical Engineer. The limits of existing fill and/or soft soil removal will be verified by the Geocon representative during site grading operations.

7.1.4 Subsequent to the recommended grading, the proposed residential buildings and parking structure may be supported on conventional foundation systems deriving support the newly placed engineered fill. All foundation excavations must be observed and approved in writing by the Geotechnical Engineer prior to placement of steel or concrete. Recommendations for *Conventional Foundation Design* are provided in Sections 7.7 through 7.9 of this report.

7.1.5 It is recommended that a seismic separation or flexible connection be utilized where adjacent structures abut. The design of the connection is at the discretion of the project structural engineer.

7.1.6 It is recommended that flexible utility connections be utilized for all rigid utilities to minimize or prevent damage to utilities from minor differential movements.

7.1.7 It is anticipated that stable excavations for the recommended grading associated with the proposed structures can be achieved with sloping measures. However, if excavations in close proximity to an adjacent property line and/or existing structure are required, special excavation measures such as slot-cutting may be necessary in order to maintain lateral support of offsite improvements. Excavation recommendations for *Temporary Excavations* and *Slot Cutting* are provided in Sections 7.18 and 7.19.

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

7.1.9 Where new paving is to be placed, it is recommended that all existing fill and soft alluvial soils be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all existing fill and soft soils in the area of new paving is not required; however, paving constructed over existing uncertified fill or unsuitable alluvium may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper 12 inches of soil should be scarified and properly compacted for paving support. *Preliminary Pavement Recommendations* are provided in Section 7.13.

7.1.10 Based on the results of percolation testing performed at the site, a stormwater infiltration system is considered feasible for this project. Recommendations for *Stormwater Infiltration* are provided in Section 7.20.

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

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

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

7.2 Soil and Excavation Characteristics

7.2.1 The in-situ soils can be excavated with moderate effort using conventional excavation equipment. Due to the loose and granular nature of the soils, minor caving should be anticipated in unshored excavations. In addition, the contractor should be aware that formwork may be required to prevent caving of shallow spread foundation excavations.

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 7.18).

7.2.4 The existing site soils encountered during the field investigation near the ground surface are considered to have a "very low" (EI = 4) expansive potential and are classified as "non-expansive" in accordance with the 2016 California Building Code (CBC) Section 1803.5.3. The recommendations presented herein assume that the building foundations and slabs will derive support in these materials.

7.3 Hydroconsolidation

7.3.1 Hydroconsolidation is the tendency of unsaturated soil structure to collapse upon saturation resulting in the overall settlement of the effected soil and overlying foundations or improvements supported thereon. Potentially compressible soils underlying the site are typically removed and recompacted during remedial site grading. However, if compressible soil is left in-place, a potential for settlement due to hydroconsolidation of the soil exists.

7.3.2 The laboratory test results presented herein, indicate that the potential for hydroconsolidation for the soils located near Borings B1 and B6, between 10 and 14 feet in depth ranges from about 1.7 to 2.2 percent. Therefore, the potential for settlement due to hydroconsolidation is up to 1 inch in those areas, should those soils become saturated.

7.3.3 Minimal infiltration of surface runoff is anticipated in property paved and maintained parking areas. Provided proper drainage measures are designed and implemented, the potential for saturation of the soils and subsequent hydroconsolidation will be minimized. Maintaining proper site drainage is discussed in Section 7.21 of the referenced report.

7.4 Minimum Resistivity, pH, and Water-Soluble Sulfate

7.4.1 Potential of Hydrogen (pH) and resistivity testing, as well as chloride content testing, were performed on representative samples of on-site material to generally evaluate the corrosion potential to surface utilities. The tests were performed in accordance with California Test Method Nos. 643 and 422 and indicate that the soils are considered “moderately corrosive” with respect to corrosion of buried ferrous metals on site. The results are presented in Appendix B (Figure B9) and should be considered for design of underground structures.

7.4.2 Laboratory tests were performed on representative samples of the on-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 B9) and indicate that the on-site materials possess “not applicable” sulfate exposure to concrete structures as defined by 2016 CBC Section 1904 and ACI 318-11 Sections 4.2 and 4.3.

7.4.3 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.

7.5 Grading

7.5.1 Grading is anticipated to include excavation of site soils for the proposed residential buildings, parking structure, foundations, and utility trenches, as well as placement of backfill for foundations, walls, and trenches.

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

7.5.3 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.

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

7.5.5 As a minimum, the upper 5 feet of existing soils within the footprint areas of the proposed residential buildings and parking structure should be excavated and properly compacted for foundation and slab support. The engineered fill blanket should extend at least 5 feet beyond the edge of foundations or for a distance equal to the depth of fill below the foundations, whichever is greater. Where the recommended grading cannot be performed, such as adjacent to property lines, alternate grading and foundation design recommendations may be required. Proposed foundations at or near existing grade should be underlain by at least 3 feet of newly compacted engineered fill. Foundations with an embedment greater than 2 feet will require deeper grading in order to maintain the required 3-foot-thick fill blanket beneath foundations. It is recommended that the grading contractor verify the depth of all building foundations prior to commencement of site grading activities in order to correctly determine the required excavation depth. Deeper fill or soft soils encountered during site grading operations should be excavated as necessary at the direction of the Geotechnical Engineer. The limits of existing fill and/or soft soil removal will be verified by the Geocon representative during site grading operations.

7.5.6 It is anticipated that stable excavations for the recommended grading associated with the proposed structures can be achieved with sloping measures. However, if excavations in close proximity to an adjacent property line and/or existing structure are required, special excavation measures such as slot-cutting may be necessary in order to maintain lateral support of offsite improvements. Excavation recommendations for *Temporary Excavations* and *Slot Cutting* are provided in Sections 7.18 and 7.19.

7.5.7 Excavated soil generally free of deleterious debris can be placed as fill and compacted in layers to the design finish grade elevations. Fill and backfill soil should be placed in horizontal loose layers approximately 6 to 8 inches thick, moisture conditioned to optimum moisture content, and compacted to a dry density of at least 90 percent of the laboratory maximum dry density as determined by ASTM D 1557 (latest edition).

7.5.8. Where new paving is to be placed, it is recommended that all existing fill and soft alluvium be excavated and properly compacted for paving support. As a minimum, the upper 12 inches of soil should be scarified, moisture conditioned to optimum moisture content, and compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition). Paving recommendations are provided in *Preliminary Pavement Recommendations* section of this report (see Section 7.13).

7.5.9 Foundations for small outlying structures, such as block walls up to 6 feet high, planter walls or trash enclosures, which will not be tied to the proposed structure, may be supported on conventional foundations deriving support on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and proper compaction cannot be performed or is undesirable, foundations may derive support directly in the undisturbed alluvial soils found at or below a depth of 12 inches, and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials. If the soils exposed in the excavation bottom are soft or loose, compaction of the soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved in writing by a Geocon representative

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

7.5.11 Although not anticipated for this project, all imported fill shall be observed, tested, and approved by Geocon West, Inc. prior to bringing soil to the site. Rocks larger than 6 inches in diameter shall not be used in the fill. If necessary, import soils used as structural fill should have an expansion index less than 20 and soil corrosivity properties that are equally or less detrimental to that of the existing onsite soils (see Figure B9).

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

7.6 Earthwork Grading Factors

7.6.1 Shrinkage results when a volume of material removed at one density is compacted to a higher density. A shrinkage factor of between 5 and 10 percent should be anticipated when excavating and compacting the existing earth materials on the site to an average relative compaction of 92 percent.

7.6.2 If import soils will be utilized in the building pad, the soils must be placed uniformly and at equal thickness at the direction of the Geotechnical Engineer (a representative of Geocon West, Inc.). Soils can be borrowed from non-building pad areas and later replaced with imported soils.

7.7 Conventional Foundation Design – Residential Buildings

7.7.1 Subsequent to the recommended grading, a conventional foundation system may be utilized for support of the proposed residential buildings provided foundations derive support in newly placed engineered fill. Foundations should be underlain by a minimum of 3 feet of newly placed engineered fill.

7.7.2 Continuous footings may be designed for an allowable bearing capacity of 2,200 pounds per square foot (psf), and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade, and 12 inches into the recommended bearing materials.

7.7.3 Isolated spread foundations may be designed for an allowable bearing capacity of 2,500 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 materials.

7.7.4 The allowable soil bearing pressure above may be increased by 500 psf and 800 psf for each additional foot of foundation width and depth, respectively, up to maximum allowable bearing value of 3,300 psf should be utilized.

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

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

7.7.7 If depth increases are utilized for the exterior wall footings, this office should be provided a copy of the final construction plans so that the excavation recommendations presented herein could be properly reviewed and revised if necessary. Additional grading should be performed as necessary in order to maintain the required 3-foot-thick engineered fill blanket beneath building foundations.

7.7.8 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.

7.7.9 No special subgrade presaturation is required prior to placement of concrete. However, the foundation subgrade should be sprinkled as necessary to maintain a moist condition at the time of concrete placement.

7.7.10 Foundation excavations should be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.), 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.

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

7.8 Conventional Foundation Design – Parking Structure

7.8.1 Subsequent to the recommended grading, a conventional foundation system may be utilized for support of the proposed residential structures provided foundations derive support in newly placed engineered fill. Foundations should be underlain by a minimum of 3 feet of newly placed engineered fill.

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

7.8.3 Isolated spread foundations may be designed for an allowable bearing capacity of 3,500 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 materials.

7.8.4 The allowable soil bearing pressure above may be increased by 500 psf and 800 psf for each additional foot of foundation width and depth, respectively, up to maximum allowable bearing value of 4,500 psf should be utilized.

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

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

7.8.7 If depth increases are utilized for the exterior wall footings, this office should be provided a copy of the final construction plans so that the excavation recommendations presented herein could be properly reviewed and revised if necessary. Additional grading should be performed as necessary in order to maintain the required 3-foot-thick engineered fill blanket beneath building foundations.

7.8.8 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.

7.8.9 No special subgrade presaturation is required prior to placement of concrete. However, the foundation subgrade should be sprinkled as necessary to maintain a moist condition at the time of concrete placement.

7.8.10 Foundation excavations should be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.), 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.

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

7.9 Conventional Foundation Settlement

7.9.1 The maximum expected total settlement for the residential buildings supported on a conventional foundation system with a maximum allowable bearing pressure of 3,300 psf is estimated to be less than 1 inch and occur below the heaviest loaded structural element. Settlement of the foundation system is expected to occur on initial application of loading. Differential settlement is expected to be less than ½ inch over a distance of 20 feet.

7.9.2 The maximum expected total settlement for the parking structure supported on a conventional foundation system with a maximum allowable bearing pressure of 4,500 psf deriving support in the recommended bearing material is estimated to be less than 1½ inches and occur below the heaviest loaded structural element. Settlement of the foundation system is expected to occur on initial application of loading. Differential settlement is expected to be less than ¾ inch over a distance of 20 feet.

7.9.3 It is recommended that a seismic separation or flexible connection be utilized where adjacent structures abut. The design of the connection is at the discretion of the project structural engineer.

7.9.4 If side by side construction is planned for the residential structure and parking structure it is recommended that the parking structure be constructed prior to the adjacent residential structure in order to allow the majority of the static settlement to occur. This will help to minimize differential settlements between the two structures. The utilization of a lesser bearing value would further reduce the anticipated settlements and could be evaluated further once the design becomes more finalized.

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

7.10 Miscellaneous Foundations

7.10.1 Foundations for small outlying structures, such as block walls up to 6 feet in height, planter walls or trash enclosures, which will not be structurally supported by the proposed building, may be supported on conventional foundations deriving support on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and compaction cannot be performed, such as adjacent to property lines, foundations may derive support in the undisturbed alluvial soils found at or below a depth of 12 inches, and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials.

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

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

7.11 Lateral Design

7.11.1 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 0.40 may be used with the dead load forces in the undisturbed alluvial soils or newly placed engineered fill.

7.11.2 Passive earth pressure for the sides of foundations and slabs poured against undisturbed alluvial soils may be computed as an equivalent fluid having a density of 270 pounds per cubic foot (pcf) with a maximum earth pressure of 2,700 psf. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

7.12 Concrete Slabs-on-Grade

7.12.1 Concrete slabs-on-grade subject to vehicle loading should be designed in accordance with the recommendations in the *Preliminary Pavement Recommendations* section of this report (Section 7.13).

7.12.2 The project structural engineer may determine and design the necessary slab thickness and reinforcing for this structure. Unless specifically evaluated and designed by a qualified structural engineer, the slab-on-grade in the residential building should be a minimum of 4 inches of concrete reinforced with No. 4 steel reinforcing bars placed 16 inches on center in both horizontal directions and positioned vertically near the slab midpoint. The concrete slab-on-grade may bear directly on the newly placed engineered fill. Any disturbed soils should be properly compacted for slab support.

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

7.12.4 For seismic design purposes, a coefficient of friction of 0.40 may be utilized between concrete slabs and subgrade soils without a moisture barrier, and 0.15 for slabs underlain by a moisture barrier.

7.12.5 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 moistened to optimum moisture content and properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition). Crack control joints should be spaced at intervals not greater than 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.

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

7.13 Preliminary Pavement Recommendations

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

7.13.2 The following pavement sections are based on an assumed R-Value of 35. Once site grading activities are complete, it is recommended that laboratory testing confirm the properties of the soils serving as paving subgrade prior to placing pavement.

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

PRELIMINARY PAVEMENT DESIGN SECTIONS

Location	Estimated Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
Automobile Parking	3.5	3	4
Driveways	5	3	5
Trash Truck & Fire Lanes	7	4	8

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

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

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

7.14 Retaining Wall Design

7.14.1 The recommendations presented below are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 6 feet. In the event that walls significantly higher than 6 feet are planned, Geocon should be contacted for additional recommendations.

7.14.2 Retaining wall foundations may be designed in accordance with the recommendations provided in the *Conventional Foundation Design* sections of this report (see Sections 7.7 through 7.9).

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

RETAINING WALL WITH LEVEL BACKFILL SURFACE

HEIGHT OF RETAINING WALL (Feet)	ACTIVE PRESSURE EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot)	AT-REST PRESSURE EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot)
Up to 6	39	59

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

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

7.14.6 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 to the vertical line-load, H is the distance from the bottom of the footing to the bottom of excavation, z is the depth at which the horizontal pressure is desired, QL is the vertical line-load and σH is the horizontal pressure at depth z.

7.14.7 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 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, σ is the vertical pressure at depth z, Θ is the angle between a line perpendicular to the bulkhead and a line from the point-load to half the pile spacing at the bulkhead, and σ_H is the horizontal pressure at depth z.

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

7.15 Retaining Wall Drainage

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

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

7.15.3 Subdrainage pipes at the base of the retaining wall drainage system should outlet to an acceptable location via controlled drainage structures.

7.15.4 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.

7.16 Elevator Pit Design

7.16.1 The elevator pit slab and retaining wall should be designed by the project structural engineer. As a minimum the slab-on-grade for the elevator pit bottom should be at least 4 inches thick and reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions, positioned near the slab midpoint. The elevator slab and retaining wall footings may derive support in either newly placed engineered fill or the alluvial soils found at or below a depth of 5 feet if exposed in the elevator pit excavation bottom. Elevator pit walls may be designed in accordance with the recommendations in the *Conventional Foundation Design* and *Retaining Wall Design* section of this report (see Sections 7.7 through 7.9, and 7.14).

7.16.2 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.

7.16.3 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 7.15).

7.16.4 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.

7.17 Elevator Piston

7.17.1 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 the drilled excavation could compromise the existing foundation support, especially if the drilling is performed subsequent to the foundation construction.

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

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

7.18 Temporary Excavations

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

7.18.2 Vertical excavations greater than 5 feet or where surcharged by existing structures or traffic loads will require sloping or slot-cutting measures 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 height of 7 feet. A uniform slope does not have a vertical portion.

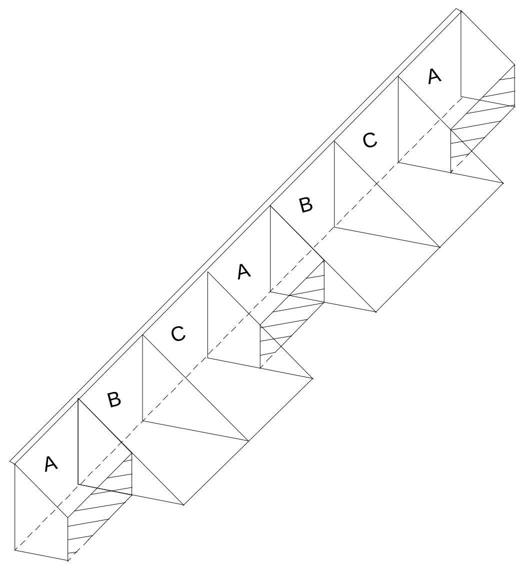
7.18.3 Performing continuous vertical excavations along property lines or adjacent to an existing structure could remove support from the property and/or structure which is not acceptable. Excavations in close proximity to an adjacent property may require special excavation measures, such as slot-cutting. Recommendations for slot-cutting are provided in the following sections.

7.18.4 Where sloped embankments 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 construction embankments 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.

7.19 Slot Cutting

7.19.1 The slot-cutting method employs the earth as a buttress and allows the earth excavation to proceed in phases. Where slot-cutting is used for foundation construction, the proposed construction techniques should be discussed with the structural engineer so that appropriate modifications can be made to the foundation design; such as additional reinforcing or details for doweling.

7.19.2 It is recommended that the initial temporary excavation along the property line be sloped back at a uniform 1:1 (H:V) slope gradient or flatter for excavation of the existing soils to the necessary depth. The temporary excavation should not extend below the surcharge area of any adjacent foundations. The surcharge area may be defined by a 1:1 projection down and away from the bottom of an existing foundation. The temporary slope may then be excavated using the slot-cutting (see illustration below).



7.19.3 Alternate "A" slots of 5½ feet in width may be worked. The remaining earth buttresses ("B" and "C" slots) should also be 5½ feet in width. The wall, foundation, or backfill should be completed in the "A" slots to a point where support of the offsite property and/or any existing structures is restored before the "B" slots are excavated. After completing the wall, foundation, or backfill in the "B" slots, finally the "C" slots may be excavated. Slot-cutting is not recommended for vertical excavations greater than 5 feet in height. The slot-cut calculation should be revised as needed for each surcharge condition as the project progresses. A slot-cut calculation is provided on the following page.

Slot Cut Calculation

Input:

Height of Slots	(H)	5.0	feet
Unit Weight of Soils	(γ)	120.0	pcf
Friction Angle of Soils	(ϕ)	33.0	degrees
Cohesion of Soils	(c)	50.0	psf
Factor of Safety	(FS)	1.50	

Factor of Safety = Resistance Force/Driving Force

Coefficient of Lateral Earth Pressure At-Rest	K_o	0.46

Surcharge Pressure:

Line Load	(q_L)	0.0	plf
Distance Away from Edge of Excavation	(X)	0.0	feet

Design Equations

$b = H/(\tan \alpha)$

$A = 0.5*H*b$

$W = 0.5*H*b*\gamma$ *(per lineal foot of slot width)*

$F_1 = d*W*(\sin \alpha)*(\cos \alpha)$

$F_2 = d*L$

$R_1 = d*[W*(\cos^2 \alpha)*(\tan \phi)+(c*b)]$

$R_2 = 2*\Delta F$

$\Delta F = A*[1/3*\gamma*H*K_o*(\tan \phi)+c]$

FS = Resistance Force/Driving Force

$FS = (R_1+R_2)/(F_1+F_2)$

Failure Angle (α) degrees	Base Width of Failure Wedge (b) feet	Area of Failure Wedge (A) feet^2	Weight of Failure Wedge (W) lbs/lineal foot	**Driving Force** Wedge + Surcharge per lineal foot of Slot Wdith	**Resisting Force** Failure Wedge per lineal foot of Slot Width	**Resisting Force** Side Resistance Force (ΔF) lbs	Allowable Width of Slots* (d) feet
45	5.0	13	1500.0	750.0	737.1	1364.3	7.0
46	4.8	12	1448.5	723.8	695.4	1317.5	6.7
47	4.7	12	1398.8	697.7	655.6	1272.2	6.5
48	4.5	11	1350.6	671.6	617.8	1228.4	6.3
49	4.3	11	1303.9	645.6	581.8	1186.0	6.1
50	4.2	10	1258.6	619.8	547.5	1144.8	6.0
51	4.0	10	1214.7	594.1	514.9	1104.8	5.9
52	3.9	10	1171.9	568.6	483.8	1065.9	5.8
53	3.8	9	1130.3	543.3	454.2	1028.1	5.7
54	3.6	9	1089.8	518.2	426.2	991.2	5.6
55	3.5	9	1050.3	493.5	399.4	955.3	5.6
56	3.4	8	1011.8	469.0	374.1	920.2	5.6
57	3.2	8	974.1	444.9	350.0	886.0	5.6
58	3.1	8	937.3	421.2	327.1	852.5	5.6
59	3.0	8	901.3	397.9	305.5	819.7	5.6
60	2.9	7	866.0	375.0	284.9	787.7	5.7
61	2.8	7	831.5	352.6	265.5	756.2	5.7
62	2.7	7	797.6	330.6	247.1	725.4	5.8
63	2.5	6	764.3	309.2	229.7	695.1	5.9
64	2.4	6	731.6	288.3	213.2	665.4	6.1
65	2.3	6	699.5	267.9	197.7	636.2	6.2
66	2.2	6	667.8	248.2	183.1	607.4	6.4
67	2.1	5	636.7	229.0	169.2	579.1	6.6
68	2.0	5	606.0	210.5	156.2	551.2	6.9
69	1.9	5	575.8	192.6	144.0	523.7	7.2
70	1.8	5	546.0	175.5	132.5	496.6	7.6

*Width of Slots to achieve a minimum of 1.5 Factor of Safety, with a Maximum Allowable Slot Width of 8 feet.

Critical Slot Width with Factor of Safety equal or exceeding 1.5:

d_{allow} = 5.6 feet

7.20 Stormwater Infiltration

7.20.1 During the June 12, 2017 site exploration, boring B6 was excavated and utilized to perform percolation testing. The boring was advanced to the depth listed in the table below. A slotted casing was placed in the boring, and the annular space between the casing and excavation was filled with filter pack. The boring was then filled with water to pre-saturate the soils. On June 13, 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 listed in the following table. Additional correction factors may be required and should be applied by the engineer in responsible charge of the design of the stormwater infiltration system and based on applicable guidelines. Percolation test data sheet is included herein as Figure 8.

Boring	Infiltration Depth (ft.)	Average Infiltration Rate (in / hour)
B6	10-25	0.48

7.20.2 Based on the results of the subsequent laboratory testing, the upper 14 feet of existing site soils may be subject to hydroconsolidation when saturated. Therefore, it is recommended that infiltration of storm water occur below a depth of 15 feet to minimize saturation of the soils supporting the proposed structures. In addition, it is suggested that additional infiltration and laboratory testing be performed in stormwater infiltration is proposed at a location other than near where the above test was performed.

7.20.3 Provided that infiltration occurs below a depth of 15 feet, it is our opinion that the soil encountered at the depths and location as listed in the table above are suitable for infiltration of stormwater and will not create a perched groundwater condition, will not affect soil structure interaction of existing or proposed foundations due to expansive soils, will not saturate soils supported by existing or proposed retaining walls, and will not increase the potential for liquefaction. Resulting settlements are anticipated to be less than ¼ inch, if any.

7.20.4 Where infiltration systems will be utilized, it is recommended that a minimum 10-foot horizontal and vertical setback be maintained from existing or proposed foundations. The boundary of the zone of saturation may be assumed to project downward from the discharge of the infiltration facility at a gradient of 1:1. Additional setbacks may be required by the governing jurisdiction and should be incorporated into the stormwater infiltration system design as necessary.

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

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

7.21 Surface Drainage

7.21.1 Proper surface drainage is critical to the future performance of the project. Uncontrolled infiltration of irrigation excess and storm runoff into the supporting 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.

7.21.2 All site drainage should be collected and controlled in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2016 CBC 1804.4 or other applicable standards. In addition, drainage should not be allowed to flow uncontrolled over any descending slope. Discharge from downspouts, roof drains and scuppers are not recommended onto unprotected soils within 5 feet of the building perimeter. Planters which are located adjacent to foundations should be sealed to prevent moisture intrusion into the soils providing foundation support. Landscape irrigation is not recommended within 5 feet of the building perimeter footings except when enclosed in protected planters.

7.21.3 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.

7.21.4 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.

7.22 Plan Review

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

LIMITATIONS AND UNIFORMITY OF CONDITIONS

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

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

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

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

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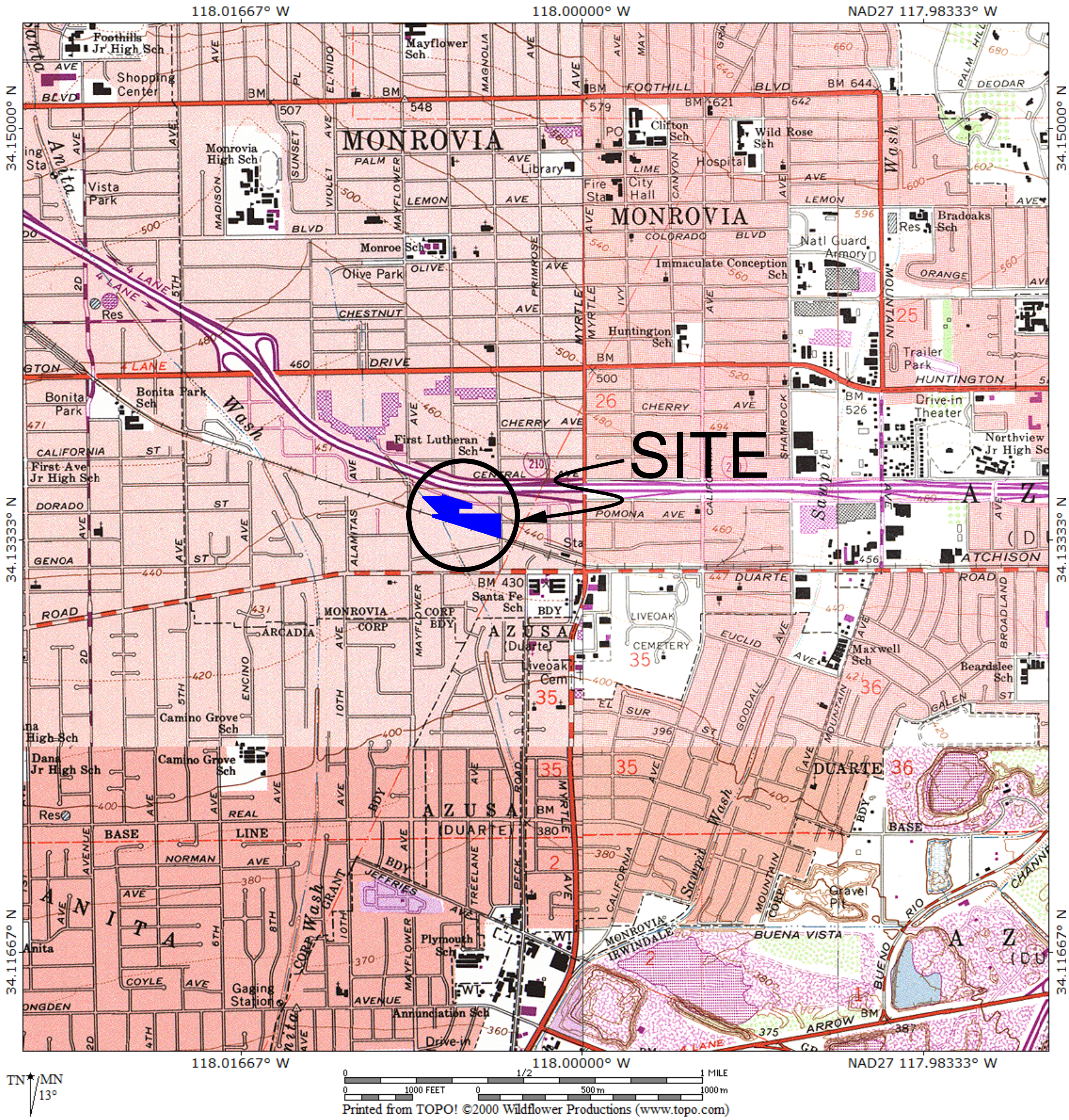

REFERENCE: U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, MOUNT WILSON, CA QUADRANGLE

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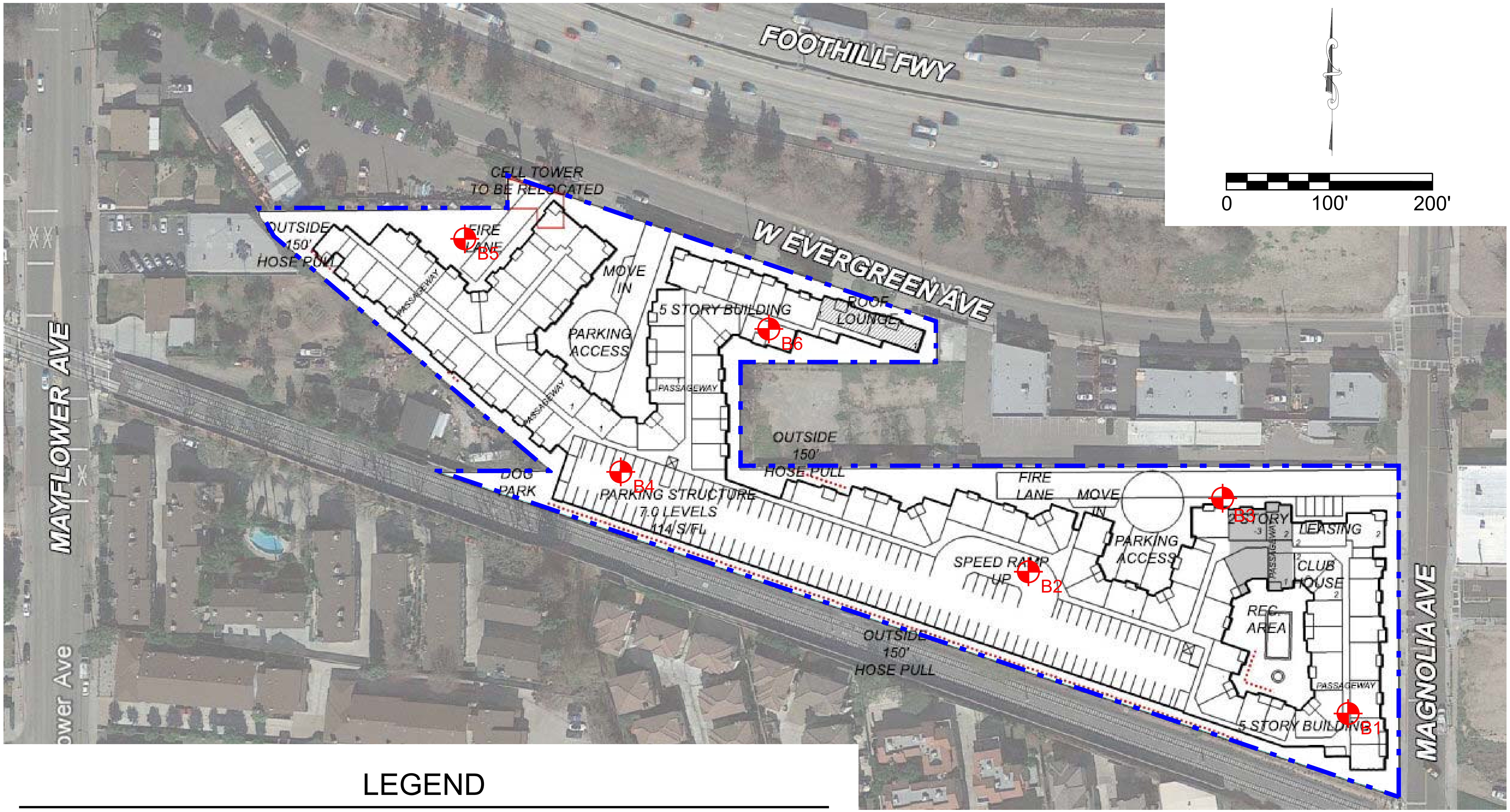
DRAFTED BY: SJB | CHECKED BY: SFK

VICINITY MAP

TRAMMELL CROW RESIDENTIAL
1625 SOUTH MAGNOLIA AVENUE
MONROVIA, CALIFORNIA

JULY 2017 | PROJECT NO. A9621-06-01 | FIG. 1



LEGEND

B6 Boring Location & Number

Approximate Limits of Proposed Development

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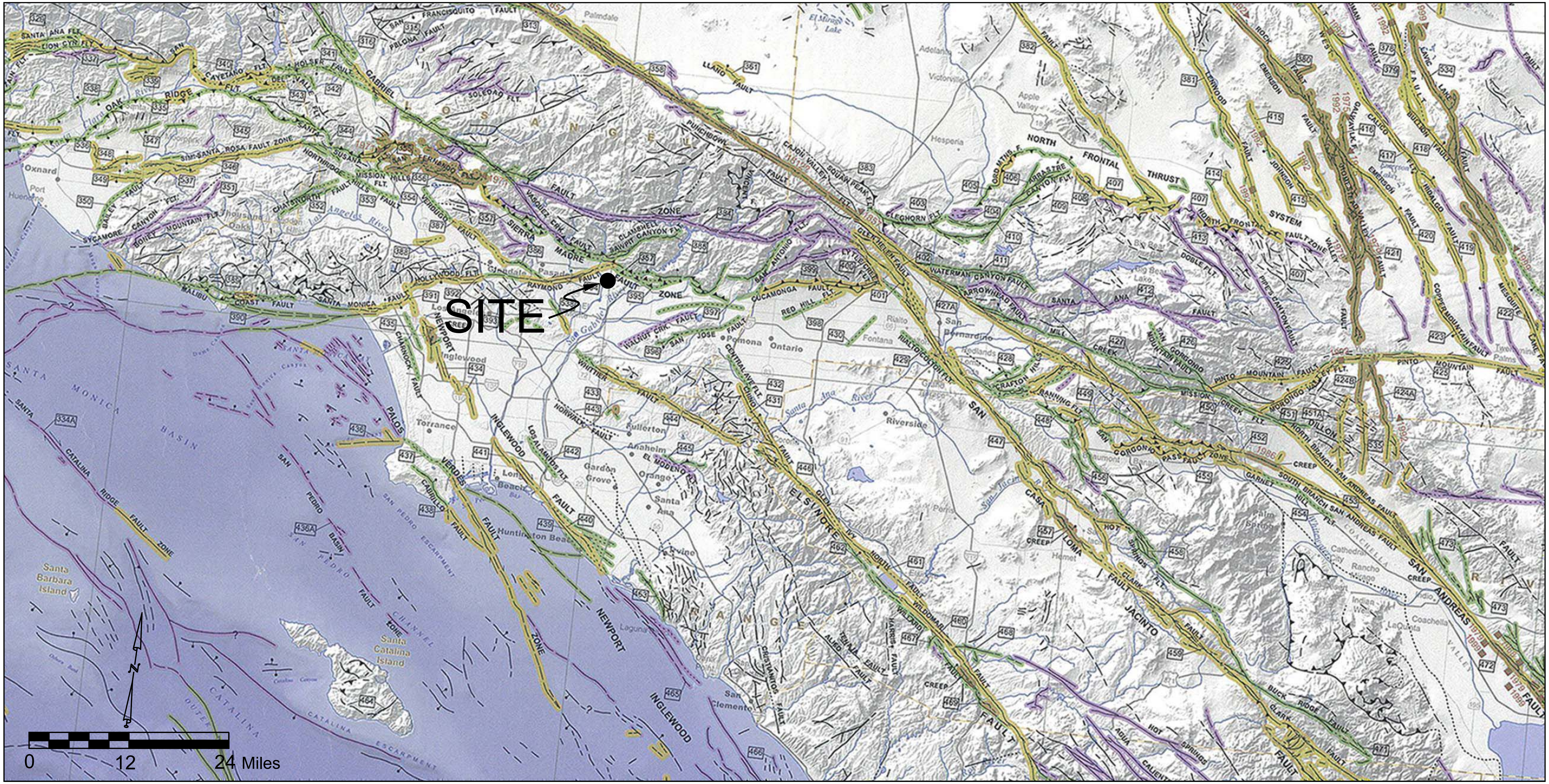
SITE PLAN

TRAMMELL CROW RESIDENTIAL
1625 SOUTH MAGNOLIA AVENUE
MONROVIA, CALIFORNIA

JULY 2017 | PROJECT NO. A9621-06-01 | FIG. 2



Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Frequency of Movement	Description: On Land	Description: Offshore
Quaternary	Late Quaternary	Historic	200			Displacement during histoic 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 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 the map were established using the pervious 1.6 Ma criterion

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

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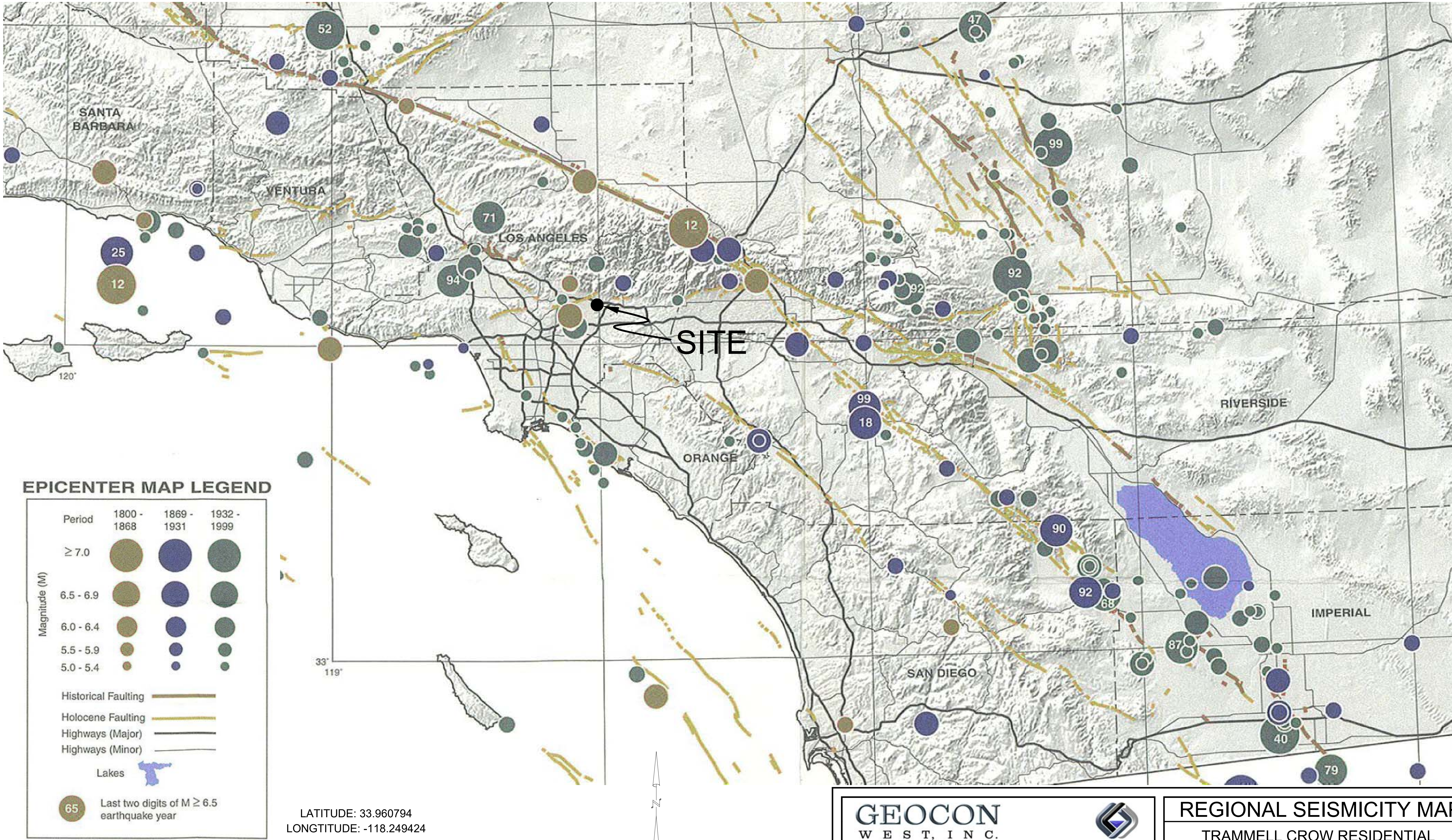
REGIONAL FAULT MAP

TRAMMELL CROW RESIDENTIAL
1625 SOUTH MAGNOLIA AVENUE
MONROVIA, CALIFORNIA

DRAFTED BY: SJB	CHECKED BY: SFK	JULY 2017	PROJECT NO. A9621-06-01	FIG. 3



LATITUDE: 33.960794
LONGTITUDE: -118.249424

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

TRAMMELL CROW RESIDENTIAL
1625 SOUTH MAGNOLIA AVENUE
MONROVIA, CALIFORNIA

JULY 2017 | PROJECT NO. A9621-06-01 | FIG. 4

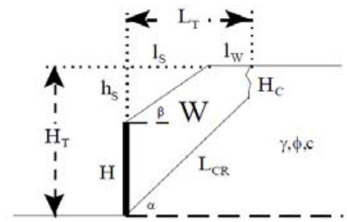
GEOCON

Project Name: 1625 S. Magnolia Ave.
Project No.: A9621-06-01
Description:

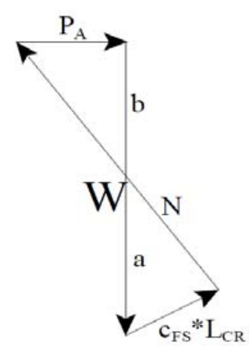
Retaining Wall Design with Transitioned Backfill (Vector Analysis)

Input:

Retaining Wall Height	(H)	6.00	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)	6.0	feet
Unit Weight of Retained Soils	(g)	120.0	pcf
Friction Angle of Retained Soils	(f)	33.0	degrees
Cohesion of Retained Soils	(c)	50.0	psf
Factor of Safety	(FS)	1.50	
Factored Parameters:	(f_{FS})	23.4	degrees
	(c_{FS})	33.3	psf



Failure Angle (a) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
45	1.0	18	2102.4	7.1	590.2	1512.2	598.4
46	1.0	17	2033.0	7.0	558.5	1474.6	613.5
47	0.9	16	1965.4	6.9	529.5	1436.0	627.1
48	0.9	16	1899.6	6.8	502.9	1396.6	639.2
49	0.9	15	1835.5	6.8	478.6	1356.8	649.8
50	0.9	15	1772.9	6.7	456.2	1316.7	659.1
51	0.9	14	1712.0	6.6	435.6	1276.4	667.0
52	0.9	14	1652.5	6.5	416.5	1236.0	673.6
53	0.9	13	1594.4	6.4	398.9	1195.5	678.9
54	0.9	13	1537.7	6.4	382.5	1155.2	682.9
55	0.8	12	1482.2	6.3	367.2	1115.0	685.7
56	0.8	12	1428.0	6.2	353.0	1074.9	687.2
57	0.8	11	1374.8	6.1	339.8	1035.0	687.4
58	0.8	11	1322.8	6.1	327.4	995.4	686.4
59	0.9	11	1271.8	6.0	315.8	956.0	684.2
60	0.9	10	1221.7	5.9	304.9	916.9	680.7
61	0.9	10	1172.6	5.9	294.6	878.0	675.9
62	0.9	9	1124.3	5.8	284.9	839.4	669.9
63	0.9	9	1076.8	5.7	275.8	801.1	662.5
64	0.9	9	1030.1	5.7	267.1	763.0	653.8
65	0.9	8	984.1	5.6	258.9	725.2	643.7
66	0.9	8	938.8	5.6	251.0	687.7	632.2
67	0.9	7	894.1	5.5	243.6	650.5	619.2
68	1.0	7	849.9	5.4	236.4	613.5	604.8
69	1.0	7	806.3	5.4	229.5	576.8	588.8
70	1.0	6	763.2	5.3	222.9	540.3	571.2



Design Equations (Vector Analysis):

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

$b = W-a$

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

$EFP = 2{*}P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\,max}$ 687.4 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2{*}P_A/H^2$

EFP 38.2 pcf 58.8 pcf

Design Wall for an Equivalent Fluid Pressure: 39 pcf 59 pcf

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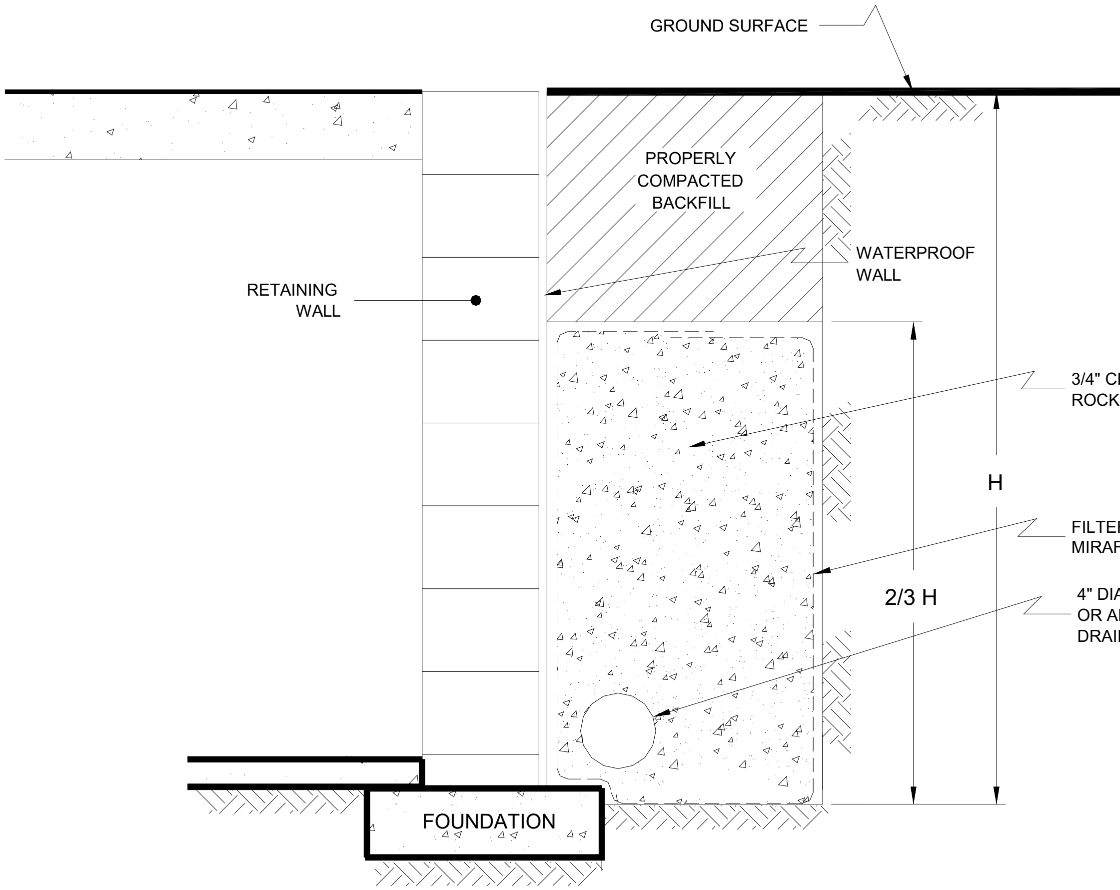
RETAINING WALL PRESSURE CALCULATION

TRAMMELL CROW RESIDENTIAL
1625 SOUTH MAGNOLIA AVENUE
MONROVIA, CALIFORNIA

DRAFTED BY: AG	CHECKED BY: JTA	JULY 2017	PROJECT NO. A9621-06-01	FIG. 5



NO SCALE

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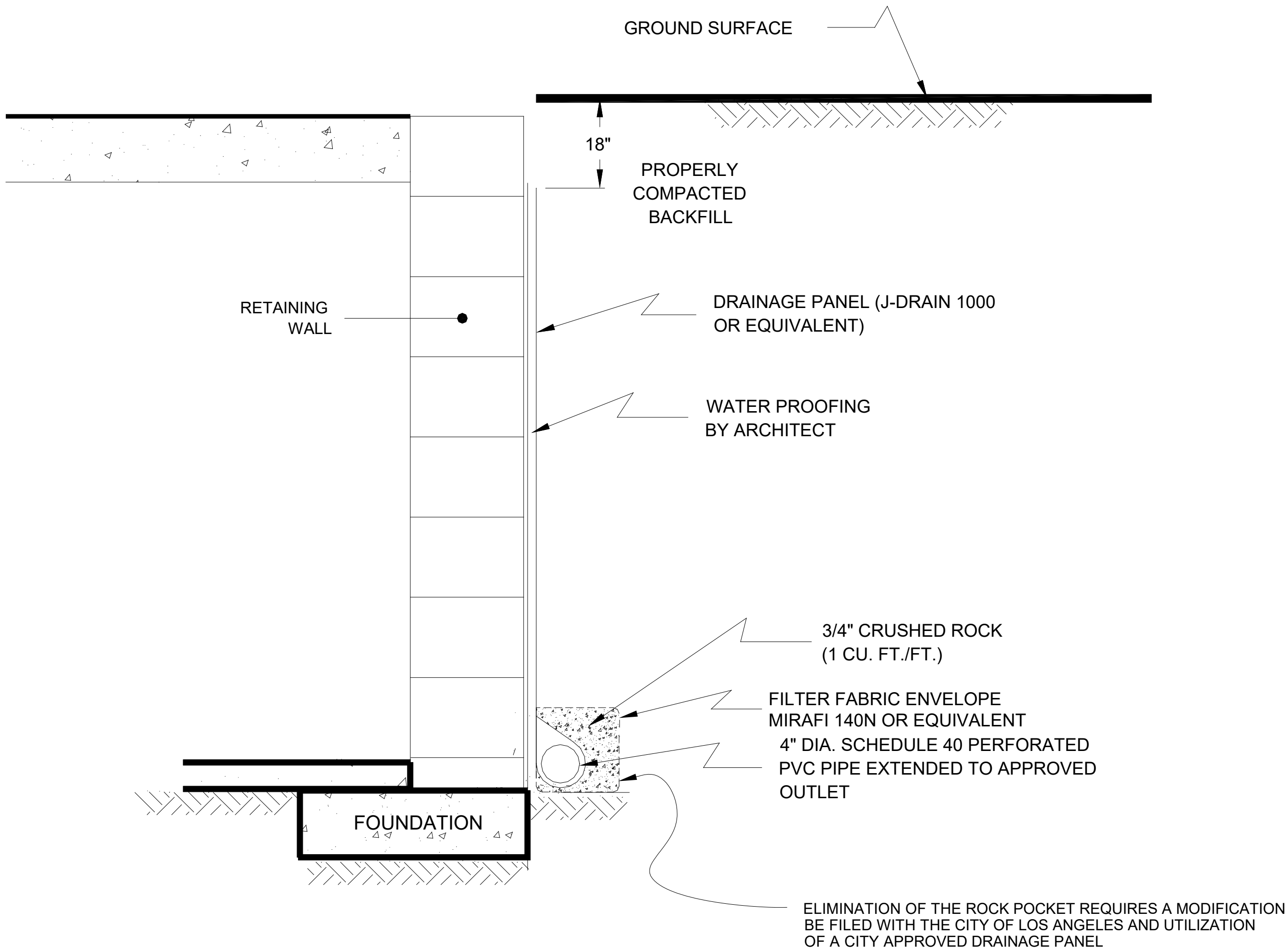
DRAFTED BY: SJB	CHECKED BY: AG

RETAINING WALL DRAIN DETAIL

TRAMMELL CROW RESIDENTIAL
1625 SOUTH MAGNOLIA AVENUE
MONROVIA, CALIFORNIA

JULY 2017	PROJECT NO. A9621-06-01	FIG. 6



NO SCALE

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RETAINING WALL DRAIN DETAIL

TRAMMELL CROW RESIDENTIAL
1625 SOUTH MAGNOLIA AVENUE
MONROVIA, CALIFORNIA

JULY 2017	PROJECT NO. A9621-06-01	FIG. 7

PERCOLATION TEST RESULTS

Boring B6

Project No:	A9621-06-01	Boring Diameter, DIA:	8	inches
Project Name:	1625 Magnolia Ave.	Boring Depth:	25.5	feet
Testing Date:	6/13/2017	Boring Depth:	306	inches
Tested By:	MA	Height of Pipe above Ground:	0.0	feet

Reading Number	Initial Water Depth (ft)	Final Water Depth (ft)	Adjusted Initial Water Depth (ft)	Adjusted Final Water Depth (ft)	Water Drop (ft)	Water Drop (in)	ΔT (min)	Percolation Rate (in/hour)
1	10.00	10.98	10.00	10.98	0.98	11.76	30.00	23.52
2	10.00	10.92	10.00	10.92	0.92	11.04	30.00	22.08
3	10.00	10.86	10.00	10.86	0.86	10.32	30.00	20.64
		Average Readings:	10.00	10.92		Preadjusted Perc Rate*		22.08

* Based only on Stabilized Readings

Initial Water Depth, d_1 =	186	inches
Final Water Depth, d_2 =	174.96	inches
Water Level Drop, Δd =	11.04	inches
Boring Diameter, DIA =	8	inches

$$R_f = \left(\frac{2d_1 - \Delta d}{DIA}\right) + 1$$

Reduction Factor, R_f = 46.12

Infiltration Rate = 0.48 inches/hour

APPENDIX A

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APPENDIX A

FIELD INVESTIGATION

The site was explored on June 12, 2017, by excavating six 8-inch diameter borings to depths of approximately 25½ feet below the existing ground surface utilizing a truck-mounted hollow-stem auger drilling machine. Representative and relatively undisturbed samples were obtained by driving a 3 inch, O. D., California Modified Sampler into the "undisturbed" soil mass with blows from a 140-pound auto-hammer falling 30 inches. The California Modified Sampler was equipped with 1-inch high by 2 $^{3}/_{8}$-inch diameter brass sampler rings to facilitate soil removal and testing. Bulk samples were also obtained.

The soil conditions encountered in the borings were visually examined, classified and logged in general accordance with the Unified Soil Classification System (USCS). Logs of the borings are presented on Figures A1 through A6. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained. The locations of the borings are shown on Figure 2.

BORING 1

ELEV. (MSL.) - - DATE COMPLETED 6/12/17

EQUIPMENT HOLLOW STEM AUGER BY: SJB

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ASPHALT: 2" BASE: 4"**			
					ARTIFICIAL FILL Silty Sand, loose, moist, light yellowish brown, fine-grained, trace medium- to coarse-grained, trace gravel (to 2").			
2	BULK 2-5'							
					ALLUVIUM Silty Sand, loose, moist, yellowish brown, fine-grained, trace medium- to coarse-grained, trace gravel (to 2").			
4	B1@4'					13	116.8	4.9
6								
8	B1@8'					17	115.5	8.6
10								
					- medium dense, increase in silt content			
12	B1@12'			SM		24	110.2	4.5
14								
					- increase in medium- to coarse-grained			
16	B1@16'					40	118.1	4.8
18								
					- very dense, slightly porous, increase in gravel content			
20	B1@20'					50 (5")	111.3	2.7
22								
24					- no recovery			
	B1@25'					50 (3")	--	--

Total depth of boring: 25.5 feet
Fill to 1 foot.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

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

A9621-06-01 BORING LOGS.GPJ

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

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

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

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BORING 2

ELEV. (MSL.) - - DATE COMPLETED 6/12/17

EQUIPMENT HOLLOW STEM AUGER BY: SJB

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ASPHALT: 2" BASE: 6"** **ARTIFICIAL FILL** Silty Sand, medium dense, moist, dark brown to grayish brown, abundant gravel (to 2").			
2	BULK 2-5'				**ALLUVIUM** Silty Sand, loose, moist, brown, fine-grained, trace fine gravel (to 1.5").			
4	B2@4'					17	108.3	12.8
6								
8	B2@8'				- medium dense, slightly moist, increase in silt content, decrease in gravel content	19	139.3	11.3
10				SM				
12	B2@12'				- loose	15	109.7	12.1
14								
16	B2@16'				- trace medium- to fine-grained, slightly porous	21	122.5	10.6
18								
20	B2@20'				- increase in fine- to medium-grained	26	118.8	6.9
22				SP	Sand, poorly graded, dense, slightly moist, fine- to medium-grained, trace coarse-grained, trace fine gravel (to 2").			
24	B2@25'				- increase in gravel content	63	126.8	2.6
					Total depth of boring: 25.5 feet Fill to 2 feet. No groundwater encountered. Backfilled with soil cuttings and tamped. *Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.			

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

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

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

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BORING 3

ELEV. (MSL.) - - DATE COMPLETED 6/12/17

EQUIPMENT HOLLOW STEM AUGER BY: SJB

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ASPHALT: 5" BASE: 3"**			
					ARTIFICIAL FILL Silty Sand, loose, moist, dark grayish brown, fine-grained, trace medium- to coarse-grained, abundant gravel (to 1.5"), trace roots (to 0.5").			
2					**ALLUVIUM** Silty Sand, loose, moist, dark brown, fine-grained, trace medium- to coarse-grained, trace gravel (to 1.5"), trace roots (to 0.5"). - no roots			
4	B3@4'					15	115.9	6.6
6					- slightly moist, brown			
8	B3@8'					18	114.2	10.6
10				SM	- loose			
12	B3@12'					14	118.4	8.3
14								
16	B3@16'					16	121.4	8.6
18								
				ML	Silt, firm, moist, light brown, trace fine-grained sand.			
20	B3@20'			SW	Sand, well-graded, medium dense, slightly moist, light gray, abundant gravel (to 2.5").	21	113.4	7.4
22				SP	Sand, poorly graded, medium dense, slightly moist, light gray, fine- to medium-grained, trace coarse-grained, trace fine gravel (to 2.5").			
24								
	B3@25'						117.2	3.1

Total depth of boring: 25.5 feet
Fill to 1.5 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

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

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

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

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

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BORING 4

ELEV. (MSL.) - - DATE COMPLETED 6/12/17

EQUIPMENT HOLLOW STEM AUGER BY: SJB

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ASPHALT: 4.5" BASE: NONE**			
					ARTIFICIAL FILL Silty Sand, loose, moist, dark brown, fine-grained, trace medium-grained, abundant gravel (to 1").			
2								
					ALLUVIUM Silty Sand, loose, moist, dark brown, fine-grained, trace medium- to coarse-grained, trace gravel (to 2").			
4	B4@4'					15	120.2	11.1
6								
					- no coarse-grained			
8	B4@8'					14	101.2	18.9
10								
12	B4@12'					14	112.3	6.7
				SM				
14								
					- increase in medium-grained, some coarse-grained			
16	B4@16'					13	110.5	11.2
18								
					- medium dense, reddish brown, increase in gravel content			
20	B4@20'					23	124.6	8.4
22								
24					- sand lenses (to 2"), slightly moist, light yellowish brown			
	B4@25'					--	118.4	11.5

Total depth of boring: 25.5 feet
Fill to 2.5 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

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

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

A9621-06-01 BORING LOGS.GPJ

SAMPLE SYMBOLS

... SAMPLING UNSUCCESSFUL ... STANDARD PENETRATION TEST ... DRIVE SAMPLE (UNDISTURBED)

... DISTURBED OR BAG SAMPLE ... CHUNK SAMPLE ... WATER TABLE OR SEEPAGE

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

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BORING 5

ELEV. (MSL.) - - DATE COMPLETED 6/12/17

EQUIPMENT HOLLOW STEM AUGER BY: SJB

DEPTH IN FEET	SAMPLE NO.	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0			**ASPHALT: 1" BASE: NONE**			
			ARTIFICIAL FILL Silty Sand, loose, moist, dark brown, fine-grained, trace medium- to coarse-grained, some gravel (to 1").			
2	BULK 2-5'					
		SM	**ALLUVIUM** Silty Sand, loose, moist, light brown, fine-grained, trace medium- to coarse-grained.			
4	B5@4'			42	117.6	5.8
6						
			- slightly moist			
8	B5@8'			32	117.4	2.5
10		SP	Sand, poorly graded, medium dense, slightly moist, fine-grained, some medium-grained, trace coarse-grained.			
12	B5@12'			32	116.0	1.0
14						
			- some coarse-grained			
16	B5@16'			34	113.7	1.8
18						
			- trace fine gravel (to 1")			
20	B5@20'			48	115.6	1.9
22						
24			- trace silt, trace medium-grained, no coarse-grained, no gravel			
	B5@25'				121.5	3.1

Total depth of boring: 25.5 feet
Fill to 1 foot.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

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

A9621-06-01 BORING LOGS.GPJ

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

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

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

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BORING 6

ELEV. (MSL.) - - DATE COMPLETED 6/12/17

EQUIPMENT **HOLLOW STEM AUGER** BY: **SJB**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL** Sandy Silt, firm, moist, light brown, fine-grained, trace medium- to coarse-grained.			
2	BULK 2-5'				**ALLUVIUM** Sandy Silt, firm, slightly moist, light brown, trace medium- to coarse-grained.			
4	B6@4'					14	102.0	12.4
6								
8	B6@8'			ML		20	115.0	7.3
10								
12	B6@12'					15	115.5	6.2
14								
16	B6@16'				Silty Sand, dense, slightly moist, light yellowish brown, fine-grained, trace medium- to coarse-grained, trace fine gravel (to 2").	62	122.8	4.4
18								
20	B6@20'			SM	- medium dense	50	124.8	3.3
22								
24								
	B6@25'					51	114.9	9.4

Total depth of boring: 25.5 feet
Fill to 1 foot.
No groundwater encountered.
Percolation testing performed.
Backfilled with soil cuttings and tamped.

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

Figure A6,
Log of Boring 6, Page 1 of 2

A9621-06-01 BORING LOGS.GPJ

SAMPLE SYMBOLS

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

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

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DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**BORING 6** ELEV. (MSL.) **- -** DATE COMPLETED **6/12/17** EQUIPMENT **HOLLOW STEM AUGER** BY: **SJB**	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
					auto-hammer.			

Figure A6,
Log of Boring 6, Page 2 of 2

A9621-06-01 BORING LOGS.GPJ

SAMPLE SYMBOLS

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

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

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APPENDIX B

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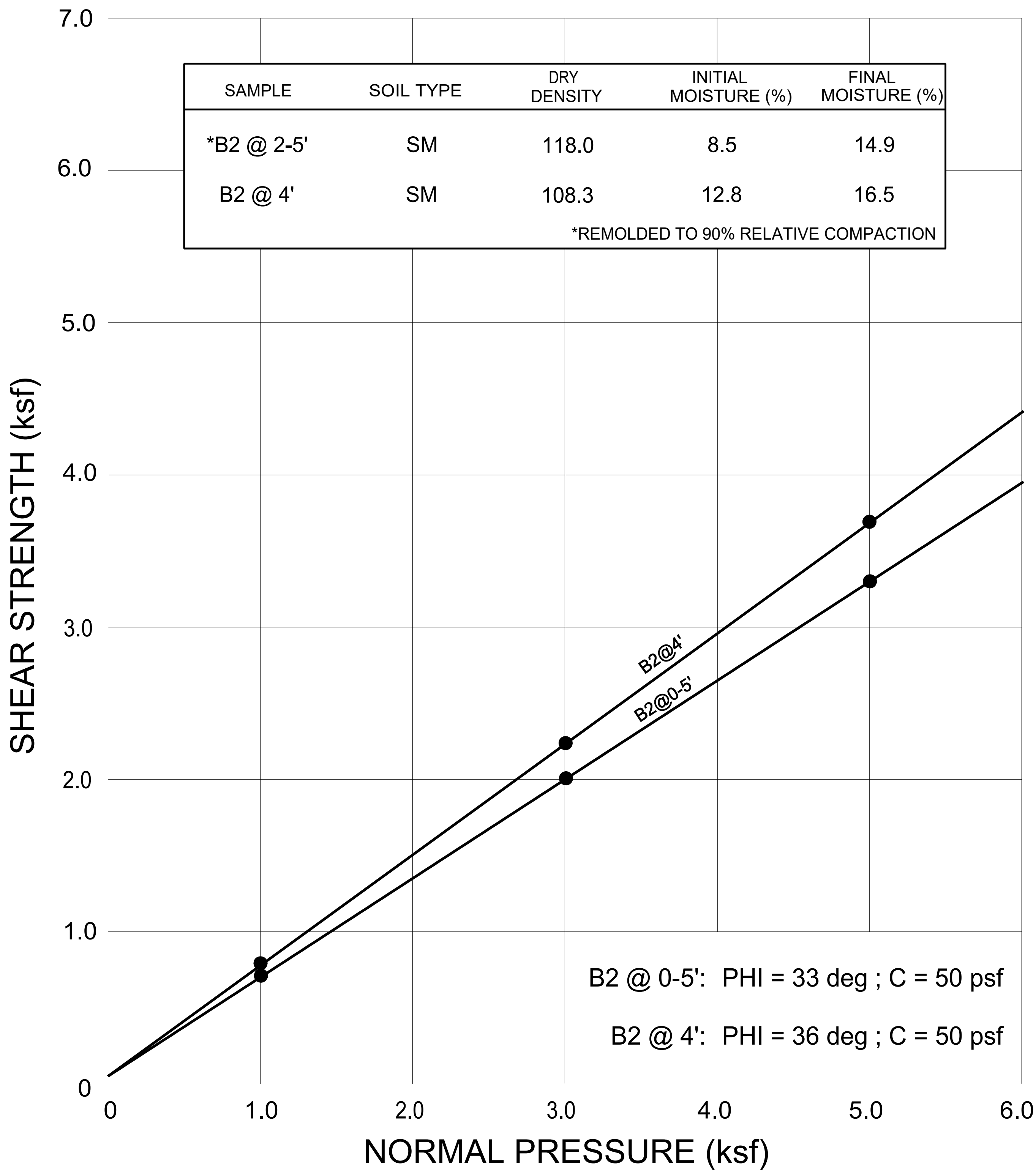
APPENDIX B

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, compaction, expansion characteristics, corrosivity, and in-place dry density and moisture content. The results of the laboratory tests are summarized in Figures B1 through B9. The in-place dry density and moisture content of the samples tested are presented on the boring logs, Appendix A.



SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
*B2 @ 2-5'	SM	118.0	8.5	14.9
B2 @ 4'	SM	108.3	12.8	16.5

*REMOLDED TO 90% RELATIVE COMPACTION

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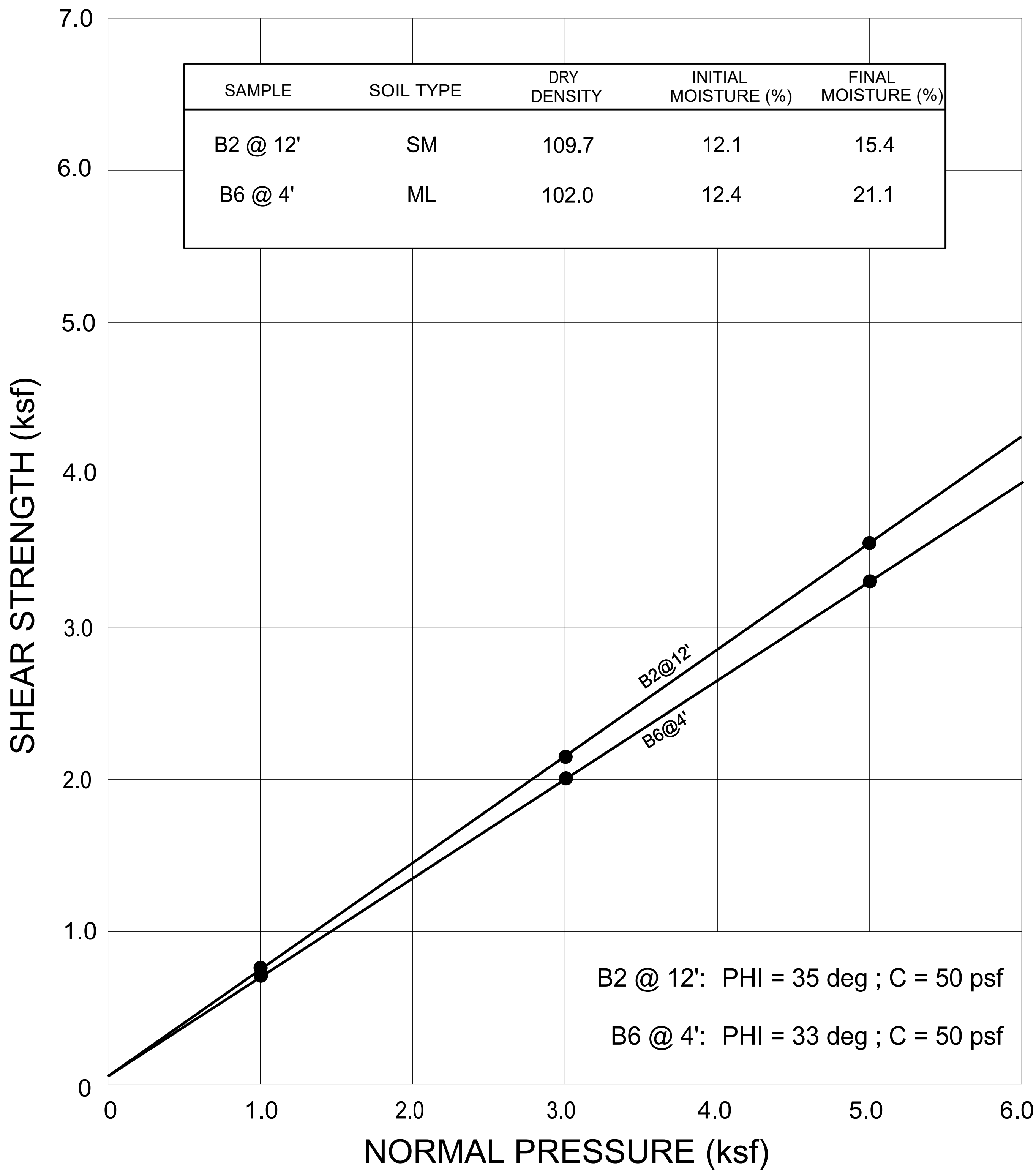
DRAFTED BY: AG | CHECKED BY: JTA

DIRECT SHEAR TEST RESULTS

TRAMMELL CROW RESIDENTIAL
1625 SOUTH MAGNOLIA AVENUE
MONROVIA, CALIFORNIA

JULY 2017 | PROJECT NO. A9621-06-01 | FIG. B1



SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2 @ 12'	SM	109.7	12.1	15.4
B6 @ 4'	ML	102.0	12.4	21.1

● DIRECT SHEAR, SATURATED

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DIRECT SHEAR TEST RESULTS

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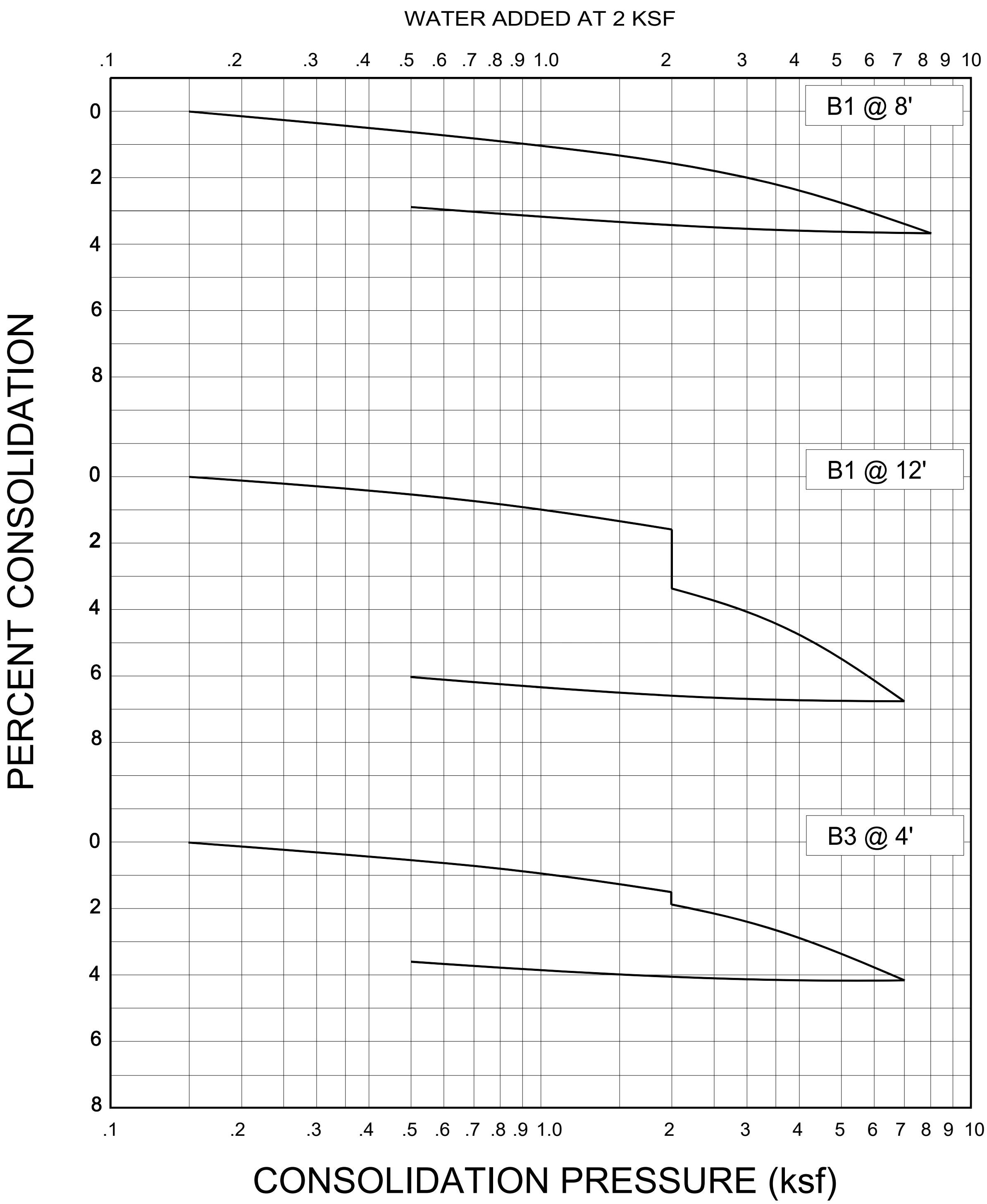
DRAFTED BY: AG	CHECKED BY: JTA	JULY 2017	PROJECT NO. A9621-06-01	FIG. B2



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CONSOLIDATION TEST RESULTS

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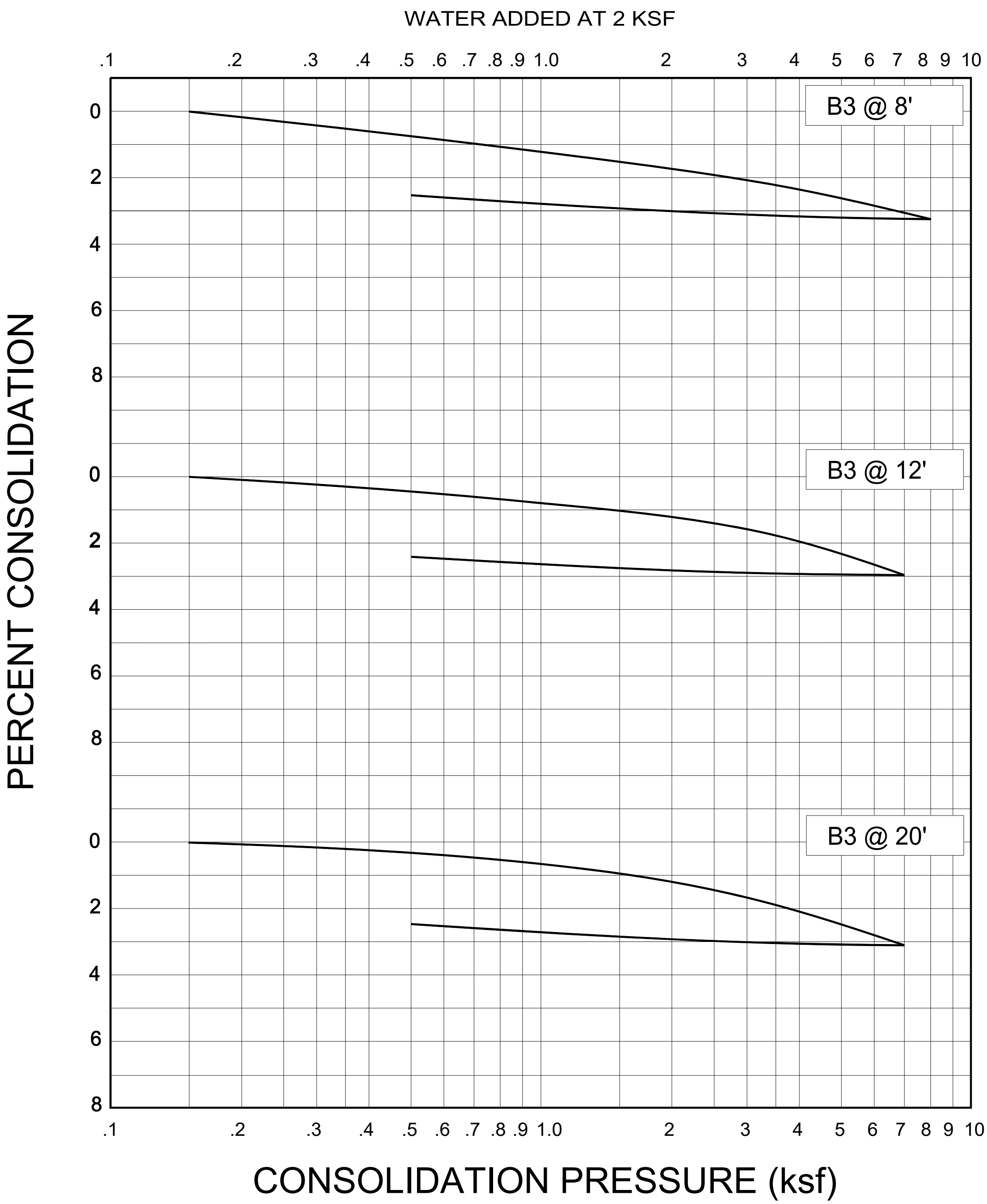
JULY 2017	PROJECT NO. A9621-06-01	FIG. B3



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CONSOLIDATION TEST RESULTS

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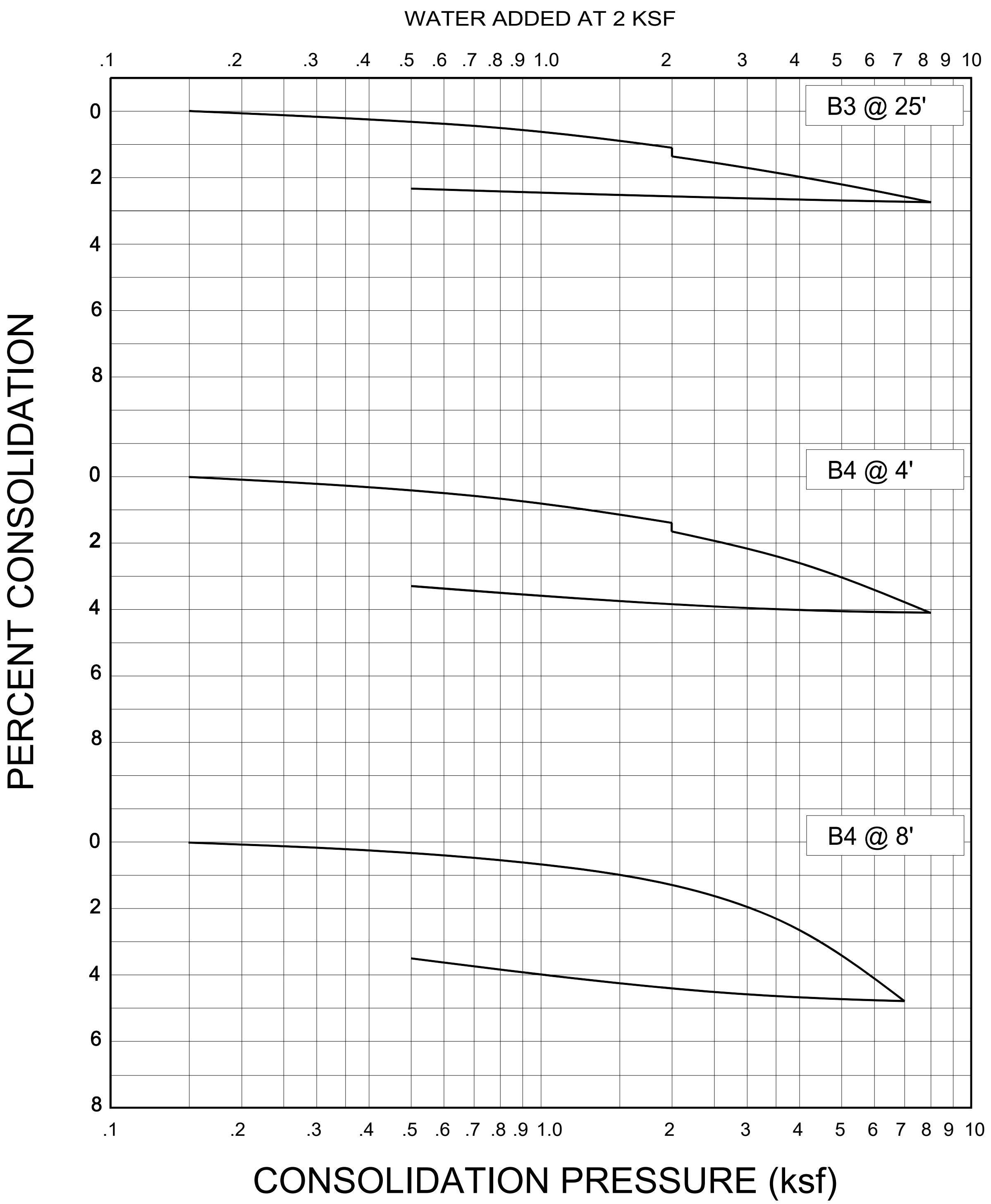
JULY 2017 | PROJECT NO. A9621-06-01 | FIG. B4



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CONSOLIDATION TEST RESULTS

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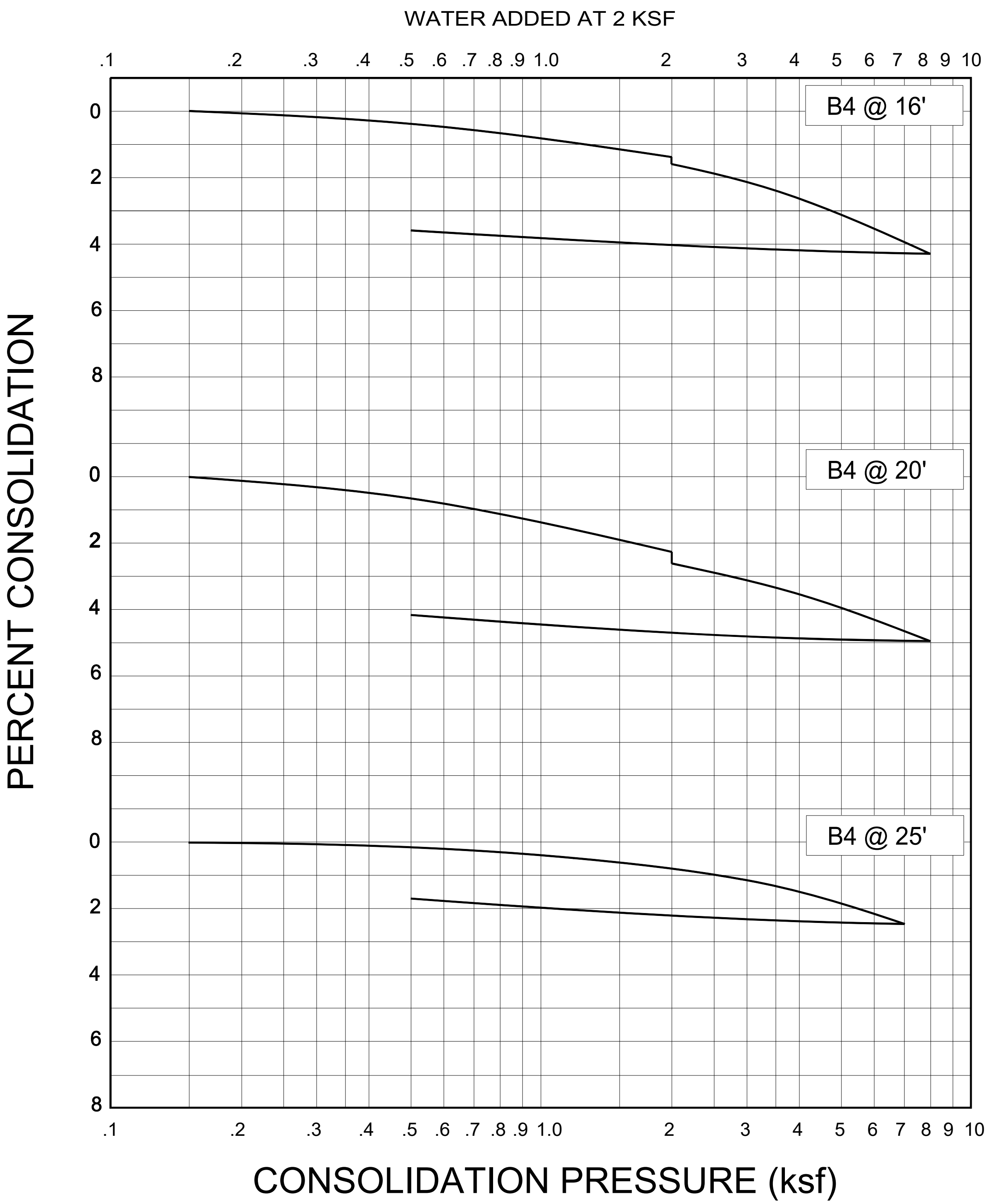
JULY 2017	PROJECT NO. A9621-06-01	FIG. B5



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CONSOLIDATION TEST RESULTS

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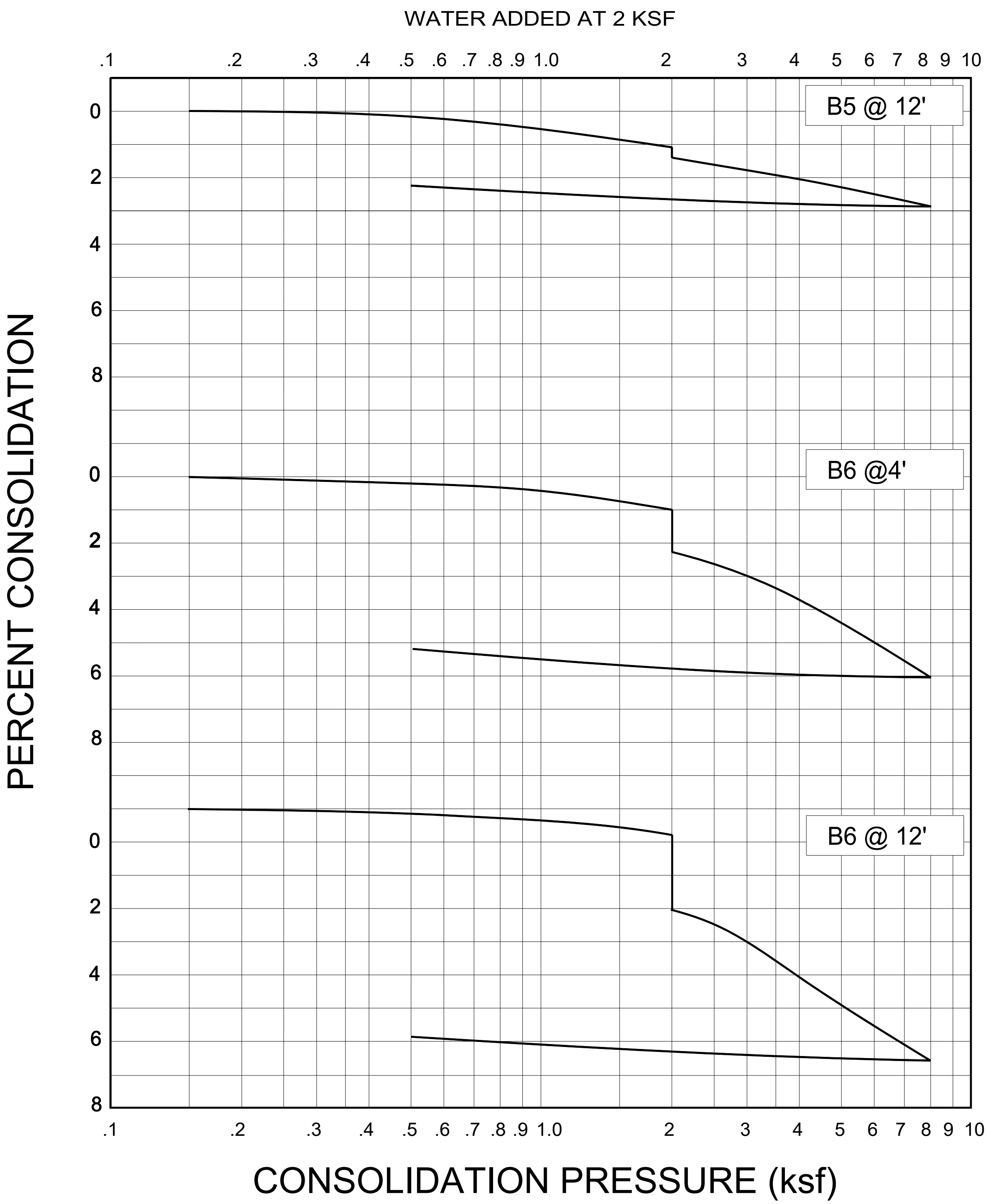
JULY 2017	PROJECT NO. A9621-06-01	FIG. B6



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CONSOLIDATION TEST RESULTS

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JULY 2017 | PROJECT NO. A9621-06-01 | FIG. B7

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

SAMPLE NO.	MOISTURE CONTENT(%)		DRY DENSITY (PCF)	EXPANSION INDEX	*UBC CLASSIFICATION	**CBC CLASSIFICATION
	BEFORE	AFTER				
B2 @ 2-5'	7.4	13.1	120.1	4	Very Low	Non-Expansive

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

** Reference: 2016 California Building Code, Section 1803.5.3

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

SAMPLE NO.	SOIL DESCRIPTION	MAXIMUM DRY DENSITY (PCF)	OPTIMUM MOISTURE CONTENT (%)
B2 @ 2-5'	Dark Brown, Silty Sand	131.5	8.5

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LABORATORY TEST RESULTS

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JULY 2017 | PROJECT NO. A9621-06-01 | FIG. B8

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

SAMPLE NO.	pH	RESISTIVITY (OHM CENTIMETERS)
B2 @ 2-5'	7.5	4,800 (Moderately Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS EPA NO. 325.3

SAMPLE NO.	CHLORIDE ION CONTENT (%)
B2 @ 2-5'	0.005

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

SAMPLE NO.	WATER SOLUBLE SULFATE (% SO_4)	SULFATE EXPOSURE *
B2 @ 2-5'	< 0.001	Not Applicable (S0)

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

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CORROSIVITY TEST RESULTS

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DRAFTED BY: AG	CHECKED BY: JTA	JULY 2017	PROJECT NO. A9621-06-01	FIG. B9