

Attachment D

Geotechnical Investigation



GEOTECHNICAL INVESTIGATION

**PROPOSED MULTI-FAMILY
RESIDENTIAL DEVELOPMENT
1917 - 2005½ WEST 3RD STREET
LOS ANGELES, CALIFORNIA
TRACT: EXTENSION OF THE NOB
HILL TRACT
LOTS: 1-6; PARCELS: 2-5**



GEOCON
WEST, INC.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

**MIRAMAR DEVELOPMENT, LP
NEW YORK, NEW YORK**

**JULY 7, 2020
PROJECT NO. W1182-06-01**



Project No. W1182-06-01
July 7, 2020

Miramar Development, LP
c/o Jonathan Rose Companies
551 Fifth Avenue, 23rd Floor
New York, NY 10176

Subject: GEOTECHNICAL INVESTIGATION
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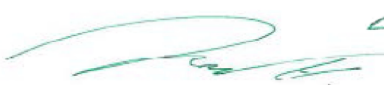
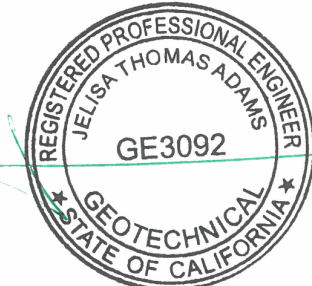
Dear Ladies and Gentlemen:

In accordance with your authorization of our proposal dated May 1, 2020, we have performed a geotechnical investigation for the proposed multi-family residential development located at 1917 -2005½ West 3rd Street in the City of Los Angeles, California. The accompanying report presents the findings of our study, and our conclusions and recommendations pertaining to the geotechnical aspects of proposed design and construction. Based on the results of our investigation, it is our opinion that the site can be developed as proposed, provided the recommendations of this report are followed and implemented during design and construction.

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

Very truly yours,

GEOCON WEST, INC.

Joe Hicks
Staff Engineer

Susan Kirkgard
CEG 1754

Jelisa Thomas Adams
GE 3092

(E-mail) 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 1917 -2005½ West 3rd Street in the City of Los Angeles, California. (see Vicinity Map, Figure 1). The purpose of the investigation was to evaluate subsurface soil and geologic conditions underlying the site and, based on conditions encountered, to provide conclusions and recommendations pertaining to the geotechnical aspects of design and construction.

The scope of this investigation included a site reconnaissance, field exploration, laboratory testing, engineering analysis, and the preparation of this report. The site was explored on May 14 and May 15, 2020. Site exploration included excavating seven 3-inch diameter hand auger borings and one 24-inch diameter bucket auger boring. The approximate locations of the exploratory borings are depicted on the Site Plan (see Figure 2A). A detailed discussion of the field investigation, including the 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 analyses of the data obtained during our investigation, as well our experience with similar soil and geologic conditions. References reviewed to prepare this report are provided in the List of References section.

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

2. SITE AND PROJECT DESCRIPTION

The subject site is located at 1917 -2005½ West 3rd Street in the City of Los Angeles, California. The site consists of two legal lots and is currently occupied by a surface parking lot with seven parking spaces and a driveway and associated hardscape and landscaping. The site is bounded by an eight-story residential structure to the north, by a two-story single-family residential structure to the east, a four-story residential structure to the west, and by West 3rd Street to the south. The subject property slopes downward to the southwest with a topographic relief across the site of approximately 14 feet. Surface water drainage at the site appears to be by sheet flow along the existing ground contours to the city streets. Vegetation on-site consists of lawn areas, trees, and shrubs typical of residential landscaping.

Based on the information provided by the Client, it is our understanding that the proposed development will consist of a seven-story structure with a partial basement level. Due to the sloping nature of the site, the proposed structure will be partially tucked into the existing slope (up to 14 feet below existing grade). The proposed development is depicted on the Site Plan and Geologic Cross-Sections (see Figures 2B through 2D).

Based on our correspondence with the project structural engineer, John Labib and Associates, column loads for the proposed structure are estimated to be up to 850 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 along the south flank of the Elysian Hills at the north edge of the Los Angeles Basin. The Elysian Hills are composed of folded and faulted Miocene age sedimentary bedrock of the Puente Formation that has been uplifted and incised by local drainages. Subsequently, the incised channels have been infilled with alluvial sediments derived from the Elysian Hills.

Regionally, the site is located within the Peninsular Ranges geomorphic province that is characterized by elongate northwest-trending mountain ridges separated by straight-sided sediment-filled valleys. The northwest trend is further reflected in the direction of the dominant geologic structural features of the province that are northwest to west-northwest trending folds and faults including the nearby Newport-Inglewood Fault Zone located approximately 5.9 miles to the west-southwest.

4. SOIL AND GEOLOGIC CONDITIONS

Based on our field investigation and published geologic maps of the area, the site is underlain by artificial fill, colluvium, and Miocene age sedimentary bedrock of the Puente Formation (Lamar, 1970). Detailed stratigraphic profiles of the materials encountered in our borings are provided on the boring logs in Appendix A.

4.1 Artificial Fill

Artificial fill was encountered to a maximum depth of 6¹/₂ feet below existing ground surface. The artificial fill generally consists of dark brown, yellowish brown, or olive brown sandy clay, sandy silt, and silty sand with some bedrock fragments and construction debris. The artificial fill is characterized as fine-grained, slightly moist to moist and medium dense or soft to stiff. 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 Colluvium

Colluvium was encountered in the explorations beneath the artificial fill in borings B1, B2, B3 and B5. The colluvial soils range in thickness from approximately 1 to 5½ feet and generally consist of dark grayish brown, olive brown or blackish brown sandy silt and clay with trace amounts of bedrock fragments (up to 1-inch maximum dimension). The colluvium is characterized as slightly moist to moist and firm to hard.

4.3 Puente Formation

Miocene age sedimentary bedrock of the Puente Formation was encountered beneath the fill and colluvial deposits at depths ranging from 1 foot to 7 feet beneath the existing ground surface. The bedrock generally consists of interbedded yellowish brown or olive brown sandstone and gray siltstone with some localized diatomaceous-rich beds. The bedrock is locally diatomaceous. The bedrock was generally characterized as moderately hard to hard, fine-grained, thinly bedded and highly weathered. Oxidation staining is common along bedding surfaces.

Bedding observed in the bucket auger boring (B7) generally strikes N74°W to N90°E and dips 18 to 36 degrees to the south. This orientation is generally consistent with published geologic maps that indicate the bedding in the area strikes east-west to N75°E and dips 33° to 68° to the southeast (Dibblee, 1991; Lamar, 1970).

5. GROUNDWATER

The site is underlain by shallow bedrock and is not located within the local groundwater basin. Consequently, historic high groundwater level information from groundwater monitoring wells is not available for the site or the immediately surrounding area from the California Geological Survey (CGS, formerly California Division of Mines and Geology, 1998).

Groundwater was not encountered in our explorations at the site. However, the shallow bedrock in the area commonly results in a perched water condition, either on top of the bedrock or within the joints and fractures in the bedrock, that can be variable from year to year based on many local factors such as irrigation and precipitation. Therefore, the contractor should be prepared to encounter groundwater seepage during construction. 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 *Surface Drainage* section of this report (See Section 7.20).

6. GEOLOGIC HAZARDS

6.1 Surface Fault Rupture

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

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

The closest active fault to the site is the Hollywood Fault located approximately 3.5 miles to the north (CGS, 2014). Other nearby active faults are the Raymond Fault, the Newport-Inglewood Fault Zone, and the Santa Monica Fault located approximately 4.6 miles northeast, 5.9 miles west-southwest, and 7.2 miles west of the property, respectively (CGS, 2014; USGS, 2006; Ziony and Jones, 1989). The active San Andreas Fault Zone is located approximately 35 miles northeast of the site (Ziony and Jones, 1989).

Several buried thrust faults, commonly referred to as blind thrusts, underlie the Los Angeles Coastal Plain at depth. These faults are not exposed at the ground surface and are typically identified at depths greater than 3.0 kilometers. The October 1, 1987 M_w 5.9 Whittier Narrows earthquake and the January 17, 1994 M_w 6.7 Northridge earthquake were a result of movement on the Puente Hills Blind Thrust and the Northridge Thrust, respectively. The site is underlain at depth by the Los Angeles segment of the Puente Hills Blind Thrust and the Upper Elysian Park Blind Thrust. These blind 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
Near Redlands	July 23, 1923	6.3	59	E
Long Beach	March 10, 1933	6.4	35	SE
Tehachapi	July 21, 1952	7.5	77	NW
San Fernando	February 9, 1971	6.6	25	NNW
Whittier Narrows	October 1, 1987	5.9	11	E
Sierra Madre	June 28, 1991	5.8	21	NE
Landers	June 28, 1992	7.3	105	E
Big Bear	June 28, 1992	6.4	83	E
Northridge	January 17, 1994	6.7	18	WNW
Hector Mine	October 16, 1999	7.1	120	ENE
Ridgecrest	July 5, 2019	7.1	123	NNE

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

6.3 Seismic Design Criteria

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

2019 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2019 CBC Reference
Site Class	C	Section 1613.2.2
MCE _R Ground Motion Spectral Response Acceleration – Class B (short), S _S	1.997g	Figure 1613.2.1(1)
MCE _R Ground Motion Spectral Response Acceleration – Class B (1 sec), S ₁	0.711g	Figure 1613.2.1(2)
Site Coefficient, F _A	1.2	Table 1613.2.3(1)
Site Coefficient, F _V	1.4	Table 1613.2.3(2)
Site Class Modified MCE _R Spectral Response Acceleration (short), S _{MS}	2.397g	Section 1613.2.3 (Eqn 16-36)
Site Class Modified MCE _R Spectral Response Acceleration – (1 sec), S _{M1}	0.996g	Section 1613.2.3 (Eqn 16-37)
5% Damped Design Spectral Response Acceleration (short), S _{DS}	1.598g	Section 1613.2.4 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (1 sec), S _{D1}	0.664g	Section 1613.2.4 (Eqn 16-39)

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

ASCE 7-16 PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-16 Reference
Mapped MCE _G Peak Ground Acceleration, PGA	0.855g	Figure 22-7
Site Coefficient, F _{PGA}	1.2	Table 11.8-1
Site Class Modified MCE _G Peak Ground Acceleration, PGA _M	1.026g	Section 11.8.3 (Eqn 11.8-1)

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

Deaggregation of the MCE peak ground acceleration was performed using the USGS online Unified Hazard Tool, 2014 Conterminous U.S. Dynamic edition (v4.2.0). The result of the deaggregation analysis indicates that the predominant earthquake contributing to the MCE peak ground acceleration is characterized as a 6.89 magnitude event occurring at a hypocentral distance of 8.46 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 6.75 magnitude event occurring at a hypocentral distance of 11.19 kilometers from the site.

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

6.4 Liquefaction Potential

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

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

The State of California Seismic Hazard Zone Map for the Hollywood Quadrangle (CDMG, 1999; CGS, 2014) indicates that the site is not located within an area identified as having a potential for liquefaction. The site is underlain shallow sedimentary bedrock of the Miocene age Puente Formation that is not susceptible to liquefaction. Based on these considerations, it is our opinion that the potential for liquefaction to occur beneath the site is considered very low.

6.5 Slope Stability

Topography at the site slopes gently to the southwest. The site is located within a City of Los Angeles Hillside Grading Area but is not located within a city-designated Hillside Ordinance Area (City of Los Angeles, 2020). According to the County of Los Angeles Safety Element (Leighton, 1990), the site is not located within an area identified as being susceptible to slope stability hazards. Additionally, the site is not within zone of required investigation for earthquake-induced landslides (CDMG, 1999; CGS, 2014, 2020b). There are no known landslides near the site, nor is the site in the path of any known or potential landslides. Therefore, the potential for slope stability hazards to adversely affect the proposed development is considered low.

As previously described, Miocene age Puente Formation bedrock was encountered at depths of 1 foot to 7 feet beneath the existing ground surface. Bedding observed at the site generally strikes N74°W to N90°E and dips 18 to 36 degrees to the south.

The planned excavation for the proposed development will be a maximum of 14 feet along the northern project boundary. The upper portion of the excavation will expose artificial fill and colluvium that are generally massive to horizontally stratified and lack any well-defined planar features or discontinuities (such as bedding planes or joints) which would act as planes of weakness. Bedrock is anticipated to be exposed in the lower portion of the planned excavation and, based on the orientation of the bedrock, is anticipated to expose adversely oriented bedrock along the project north and west excavation walls and will result in lateral surcharge loading on the proposed basement walls at these locations (see Figures 2B through 2D).

Where bedrock is exposed in the excavation, it is very important that a Professional Geologist (a representative of Geocon) observe the cuts during the grading process in order to confirm the bedrock orientation and identify any unforeseen variations within the bedrock. Any necessary recommendations to eliminate or support any unfavorably oriented bedding planes will be made at that time.

6.6 Earthquake-Induced Flooding

Earthquake-induced flooding is inundation caused by failure of dams or other water-retaining structures due to earthquakes. The Los Angeles County Safety Element (Leighton, 1990) indicates that the site is not located within a designated dam inundation area. Therefore, the potential for inundation at the site, as a result of an earthquake-induced dam failure, is considered low.

6.7 Tsunamis, Seiches, and Flooding

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

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

The site is within an area of minimal flooding (Zone X) as defined by the Federal Emergency Management Agency (FEMA, 2020; LACDPW, 2020b).

6.8 Oil Fields & Methane Potential

Based on a review of the California Geologic Energy Management Division (CalGEM) Well Finder Website (2020) the site is located within the limits of the Los Angeles City Oil Field. The majority of wells in this oil field are located to the north of the site. The nearest documented well to the site is the Conoco Inc. Well Number 1-C, an idle oil and gas well, located approximately 250 feet to the north. Due to the voluntary nature of record reporting by the oil well drilling companies, wells may be improperly located or not shown on the location map and undocumented wells could be encountered during construction. Any wells encountered will need to be properly abandoned in accordance with the current requirements of the CalGEM.

The site is located within the boundaries of a city-designated Methane Zone (City of Los Angeles, 2020). A methane study will be required by the city prior to site development. A methane specialist, Terra – Petra, has been contacted to perform a site-specific methane study. Their findings should be incorporated into the final design.

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. As previously indicated, the site is located within the City of Los Angeles Oil Field. However, the limited oil production from this field has not resulted in regional or local subsidence. No known large-scale extraction of groundwater, gas, oil, or geothermal energy is occurring or planned at the site or in the general site vicinity. Therefore, the potential for ground subsidence due to withdrawal of fluids or gases at the site is considered low.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

- 7.1.1 It is our opinion that neither soil nor geologic conditions were encountered during the investigation that would preclude the construction of the proposed development provided the recommendations presented herein are followed and implemented during design and construction.
- 7.1.2 Up to 6½ 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. Demolition of the existing structure that occupies the site is anticipated to disturb the upper few feet of existing site soils. It is our opinion that the existing fill and colluvial soils, in their present condition, are not suitable for direct support of proposed foundations or slabs. The existing fill and colluvial 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 Based on these considerations, the proposed structure may be supported on a conventional foundation system deriving support exclusively in the undisturbed bedrock. Based on our discussions with the design team, it is our understanding that the preferred design is to excavate all existing artificial fill and colluvial soils in order to construct conventional foundations directly in competent bedrock. Subsequent to foundation construction, engineered fill will be placed and compacted in order to bring the site elevation back to the proposed finished floor elevations. At this time the team has not yet determined if excavations to remove the existing artificial fill and colluvium will be conducted through the use of sloping techniques, shoring measures, or a combination of both. For the purposes of this report, we have assumed that shoring techniques will be utilized, resulting in undisturbed soils being retained by proposed below grade walls.
- 7.1.4 It is anticipated that bedrock will be exposed throughout the basement area excavation bottom, and the bedrock is considered suitable for support of the concrete slab-on-grade. At the ground floor level, it is recommended that all existing artificial fill and/or soft colluvium be excavated and properly compacted for slab support. The limits of existing fill and/or soft colluvium removal will be confirmed by the Geocon representative during site grading activities.
- 7.1.5 If a methane mitigation system is required for this project, a mat foundation system may be a more cost-effective foundation system for this project. The mat foundation system allows for more efficient construction when performed in conjunction with a methane mitigation system. In addition, the bottom of the mat can be shaped to channel the methane simplifying the passive mitigation system. A qualified methane consultant should be retained for the design of the mitigation system.

- 7.1.6 Foundations for small outlying structures, such as block walls up to 6 feet in height, planter walls or trash enclosures, which will not be tied to the proposed structure, may be supported on conventional foundations deriving support on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and compaction cannot be performed or is undesirable, such as adjacent to property lines, foundations may be deepened as necessary to maintain a minimum maintain a minimum 24-inch embedment below the ground surface, and a minimum 12-inch embedment into the undisturbed colluvium and/or bedrock. If the soils exposed in the excavation bottom are soft or loose, compaction of the soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved by a Geocon representative.
- 7.1.7 Although not encountered during site exploration, groundwater seepage conditions could develop on top of the bedrock or within the joints and fractures in the bedrock. If this occurs during construction, temporary dewatering activities may be necessary and recommendations are provided in Section 7.3 of this report.
- 7.1.8 Excavations on the order of 16 feet in vertical height are anticipated for construction of the proposed structure, including foundation depths. The proposed excavations will expose bedrock, with strikes N74°W to N90°E and dips 18 to 36 degrees to the south. These conditions are considered unfavorable to the proposed excavations along the north and west property lines. The recommendations for shoring and retaining wall design presented herein incorporate the anticipated surcharge loads generated by out-of-slope bedding. Excavations must be conducted in a manner that maintains stability and must be observed and approved by the Project Geologist (a representative of Geocon West, Inc.). A geologist should periodically observe the excavation to confirm that the orientation of the exposed bedrock is consistent with the conditions observed during our investigation.
- 7.1.9 Due to the depth of the excavation and the proximity to the property lines, city streets, and adjacent on-site structures, as well as the unfavorable geologic conditions for proposed excavations along the south property line, excavation of the proposed subterranean levels will require sloping and/or shoring measures in order to provide a stable excavation. Where shoring is required it is recommended that a soldier pile shoring system be utilized. Where excavation depths exceed 12 feet or surcharges are imposed on the shoring system, raker braces or tie-back anchors may be required in conjunction with the soldier piles. The need for lateral bracing and the acceptable shoring deflection should be determined by the project shoring engineer.

- 7.1.10 Due to the nature of the proposed design and intent for a partial subterranean basement level, waterproofing of subterranean walls and slabs is suggested. Particular care should be taken in the design and installation of waterproofing to avoid moisture problems, or actual water seepage into the structure through any normal shrinkage cracks which may develop in the concrete walls, floor slab, foundations and/or construction joints. The design and inspection of the waterproofing is not the responsibility of the geotechnical engineer. A waterproofing consultant should be retained in order to recommend a product or method, which would provide protection to subterranean walls, floor slabs and foundations.
- 7.1.11 Where new paving is to be placed, it is recommended that all existing fill soils and soft 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 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 properly compacted. Paving recommendations are provided in the *Preliminary Pavement Recommendations* section of this report (see Section 7.12).
- 7.1.12 Based on the likely impermeable nature of the bedrock which underlies the site, infiltration of stormwater at this site is not considered feasible and would be considered detrimental to the project. It is recommended that stormwater be retained, filtered, and discharged in accordance with the requirements of the local governing agency.
- 7.1.13 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. If the final foundation loading configuration is greater than the assumed loading in this report, the potential for settlement should be re-evaluated by this office.
- 7.1.14 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.

7.2 Soil and Excavation Characteristics

- 7.2.1 The in-situ soils can be excavated with moderate effort using conventional excavation equipment. Some caving should be anticipated in unshored excavations, especially if granular soils are encountered. The surficial bedrock is moderately to highly weathered and should be rippable with conventional equipment; however, concretions or well cemented layers may be encountered in the bedrock which could make excavation or drilling conditions difficult. Coring or jack-hammering may be required if concretions are encountered and the contractor should be prepared for these conditions.

- 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 soils and bedrock encountered during this investigation are considered to have a “low” to “high” expansive potential ($EI = 36$ to 108) and are classified as “expansive” based on the 2019 California Building Code (CBC) Section 1803.5.3. Recommendations presented herein assume that the building foundations and slabs will derive support in these materials.

7.3 Minimum Resistivity, pH and Water-Soluble Sulfate Content

- 7.3.1 Potential of Hydrogen (pH) and resistivity testing as well as chloride content testing were performed on representative samples of soil to generally evaluate the corrosion potential to surface utilities. The tests were performed in accordance with California Test Method Nos. 643 and 422 and indicate that the soils are considered “severely corrosive” with respect to corrosion of buried ferrous metals onsite. The results are presented in Appendix B (Figure B12) and should be considered for design of underground structures. Due to the corrosive potential of the soils, it is recommended that PVC, ABS or other approved plastic piping be utilized in lieu of cast-iron when in direct contact with the site soils.
- 7.3.2 Laboratory tests were performed on representative samples of the site materials to measure the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate tests are presented in Appendix B (Figure B12) and indicate that the on-site materials possess a sulfate exposure class of “S2” to concrete structures as defined by 2019 CBC Section 1904 and ACI 318-14 Table 19.3.1.1. The table below presents a summary of concrete requirements set forth by 2019 CBC Section 1904 and ACI 318. The presence of water-soluble sulfates is not a visually discernible characteristic; therefore, other soil samples from the site could yield different concentrations. Additionally, over time landscaping activities (i.e., addition of fertilizers and other soil nutrients) may affect the concentration.

REQUIREMENTS FOR CONCRETE EXPOSED TO SULFATE-CONTAINING SOLUTIONS

Exposure Class	Water-Soluble Sulfate Percent by Weight	Cement Type (ASTM C150)	Maximum Water to Cement Ratio by Weight	Minimum Compressive Strength (psi)
S0	$SO_4 < 0.10$	--	--	2,500
S1	$0.10 \leq SO_4 < 0.20$	II	0.50	4,000
S2	$0.20 \leq SO_4 \leq 2.00$	V	0.45	4,500
S3	$SO_4 > 2.00$	V+Pozzolan or Slag	0.45	4,500

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

7.4 Temporary Dewatering

- 7.4.1 Groundwater seepage conditions could develop on top of the bedrock or within the joints and fractures in the bedrock. The presence of groundwater seepage can be further verified at the time of construction. If groundwater seepage occurs above the excavation bottom, temporary dewatering will be necessary to maintain a safe working environment during excavation and construction activities. Dewatering system design, operation, and proper discharge is the responsibility of the contractor.
- 7.4.2 Temporary dewatering may be accomplished by constructing gravel filled trenches (French drains) to collect and control any encountered seepage. The number and locations of the French drains can be adjusted during excavation activities as necessary. The French drains will then direct the collected seepage to a sump where it will be pumped out of the excavation. If a French drain is to remain on a permanent basis, it must be lined with filter fabric to prevent soil migration into the gravel. Alternative methods of temporary dewatering may be feasible, and it is recommended that the contractor meet with the Geotechnical Engineer to discuss dewatering procedures in more detail.

7.5 Grading

- 7.5.1 Grading is anticipated to include excavation of the site soils for the proposed structure, foundations, and utility trenches, as well as placement of backfill for ramps and trenches.
- 7.5.2 Earthwork should be observed, and compacted fill tested by representatives of Geocon West, Inc. The existing site soils and bedrock encountered during exploration are suitable for re-use as engineered fill, provided any encountered oversized material (greater than 6 inches) and any encountered deleterious debris are removed. If bedrock is to be utilized as an engineered fill, it may be blocky and may have to be crushed and moisture conditioned prior to utilization. The contractor should also be aware the bedrock has some localized diatomaceous-rich beds, which may make use of this material as engineered fill difficult.
- 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, geotechnical engineer, and, if applicable, building official 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.) and the City of Los Angeles Inspector.
- 7.5.5 It is recommended that foundations for the proposed structure penetrate through the existing artificial fill and colluvium in order to derive support exclusively in the underlying competent bedrock. All excavation bottoms must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.) prior to the placement of engineered fill.
- 7.5.6 It is anticipated that bedrock will be exposed throughout the basement area excavation bottom, and the bedrock is considered suitable for support of the concrete slab-on-grade. Any bedrock that is unintentionally disturbed should be compacted for slab support. At the ground floor level, it is recommended that all existing artificial fill and/or soft colluvium be excavated and properly compacted for slab support. The limits of existing fill and/or soft colluvium removal will be confirmed by the Geocon representative during site grading activities.

- 7.5.7 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. Imported soils should have an expansion index less than 50 and soils corrosivity properties that are equally or less detrimental to that of the existing onsite soils (see Figure B12).
- 7.5.8 Prior to placing engineered fill over bedrock, the bedrock exposed at the excavation bottom must be cut or benched to create horizontal planes so that fill can be placed and compacted on horizontal surfaces.
- 7.5.9 The City of Los Angeles Department of Building and Safety requires a minimum compactive effort of 95 percent of the laboratory maximum dry density in accordance with ASTM D 1557 (latest edition) where the soils to be utilized in the fill have less than 15 percent finer than 0.005 millimeters. Soils with more than 15 percent finer than 0.005 millimeters may be compacted to 90 percent of the laboratory maximum dry density in accordance with ASTM D 1557 (latest edition). All fill and backfill soils should be placed in horizontal loose layers approximately 6 to 8 inches thick, moisture conditioned to near optimum moisture content, and properly compacted to the required degree of compaction in accordance with ASTM D 1557 (latest edition).
- 7.5.10 Where new paving is to be placed, it is recommended that all existing fill soils and soft soils be excavated and properly compacted for paving support. As a minimum, the upper 12 inches of soil should be scarified, moisture conditioned to two percent above optimum moisture content, and compacted to at least 92 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition).
- 7.5.11 Utility trenches should be properly backfilled in accordance with the requirements of the Green Book (latest edition). The pipe should be bedded with clean sands (Sand Equivalent greater than 30), to a depth of at least 1 foot over the pipe, and the bedding material must be inspected and approved in writing by the Geotechnical Engineer (a representative of Geocon). The use of gravel is not acceptable unless used in conjunction with filter fabric to prevent the gravel from having direct contact with soil. The remainder of the trench backfill may be derived from onsite soil or approved import soil, compacted as necessary, until the required compaction is obtained. The use of minimum 2-sack slurry is also acceptable as backfill (see Section 7.7). Prior to placing any bedding material or pipes, the trench excavation bottom must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon).
- 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 sands, fill, steel, gravel, or concrete.

7.6 Controlled Low Strength Material (CLSM)

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

Standard Requirements

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

Requirements for CLSM that will be used for support of footings

1. The cement content of the CLSM shall not be less than 188 pounds per cubic yard (min. 2 sacks);
2. The excavation bottom must be level, cleaned of loose soils and approved in writing by Geocon prior to placement of the CLSM;
3. The ultimate compressive strength of the CLSM shall be no less than 100 pounds per square inch (psi) when tested on the 28th-day per ASTM D4832 (latest edition), Standard Test Method for Preparation and Testing of Controlled Low Strength Material Test Cylinders. Compression testing will be performed in accordance with ASTM C39 and City of Los Angeles requirements;
4. Samples of the CLSM will be collected during placement, a minimum of one test (two cylinders) for each 50 cubic yards or fraction thereof;
5. Overexcavation for CLSM placement shall extend laterally beyond the footprint of any proposed footings as required for placement of compacted fill, unless justified otherwise by the soil engineer that footings will have adequate vertical and horizontal bearing capacity.

7.7 Conventional Foundation Design

- 7.7.1 The proposed structure may be supported on a conventional foundation system deriving support in the competent bedrock. Proposed foundations should penetrate through the existing artificial fill, colluvium and any soft or highly weathered bedrock, in order to derive support exclusively in competent bedrock. Foundations excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.).
- 7.7.2 Continuous footings may be designed for an allowable bearing capacity of 3,000 pounds per square foot (psf), and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade, and 12 inches into the recommended bearing material.
- 7.7.3 Isolated spread foundations may be designed for an allowable bearing capacity of 3,700 psf, and should be a minimum of 24 inches in width, 18 inches in depth below the lowest adjacent grade, and 12 inches into the recommended bearing material.
- 7.7.4 The allowable soil bearing pressure above may be increased by 100 psf and 380 psf for each additional foot of foundation width and depth, respectively, up to a maximum allowable soil bearing pressure of 6,200 psf.
- 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 If depth increases are utilized for the perimeter foundations, this office should be provided a copy of the final construction plans so that the excavation recommendations presented herein could be properly reviewed and revised if necessary.
- 7.7.7 Continuous footings should be reinforced with four No. 4 steel reinforcing bars, two placed near the top of the footing and two near the bottom. Reinforcement for spread footings should be designed by the project structural engineer.
- 7.7.8 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 Provided the foundation excavations and the concrete slab-on-grade subgrade exposes undisturbed bedrock, no special subgrade presaturation is required prior to placement of concrete. However, the slab and foundation subgrade should be sprinkled as necessary to maintain a moist condition as would be expected in any concrete placement. Where engineered fill is placed as subgrade soils, due to the expansive nature of the soils, the moisture content in the slab and foundation subgrade should be maintained at 2 percent above optimum moisture content prior to and 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.), prior to the placement of reinforcing steel and concrete to verify that the excavation and exposed soil conditions are consistent with those anticipated. If unanticipated 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 excavation recommendations presented herein could be properly reviewed and revised if necessary.

7.8 Mat Foundation Design

- 7.8.1 The client should be aware that if a methane mitigation system is required for this project, a mat foundation system may be a more cost-effective foundation system for this project. The mat foundation system allows for more efficient construction when performed in conjunction with a methane mitigation system. The mat foundation may derive support in the competent bedrock, and foundations excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.).
- 7.8.2 The recommended maximum allowable bearing value is 6,200 pounds per square foot (psf). The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.
- 7.8.3 It is recommended that a modulus of subgrade reaction of 200 pounds per cubic inch (pci) be utilized for the design of the mat foundation bearing in the undisturbed bedrock. The modulus should be reduced in accordance with the following equation when used with larger foundations:

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

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

- 7.8.4 The thickness of and reinforcement for the mat foundation should be designed by the project structural engineer.
- 7.8.5 For seismic design purposes, a coefficient of friction of 0.30 may be utilized between the concrete mat and bedrock without a moisture barrier, and 0.15 for slabs underlain by a moisture barrier or methane barrier.

- 7.8.6 Foundation excavations should be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.), prior to the placement of reinforcing steel and concrete to verify that the exposed soil conditions are consistent with those anticipated. If unanticipated soil conditions are encountered, foundation modifications may be required.
- 7.8.7 Waterproofing of subterranean walls and slabs is suggested for this project. 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.
- 7.8.8 The client should be aware that if a methane barrier is installed to envelope the structure, the installation of a waterproofing barrier should not be necessary since it is a redundant system. 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.
- 7.8.9 This office should be provided a copy of the final construction plans so that the recommendations presented herein could be properly reviewed and revised if necessary.

7.9 Foundation Settlement

- 7.9.1 The maximum anticipated settlement for a conventional foundation system designed with the maximum allowable bearing value of 6,200 psf and deriving support in undisturbed bedrock is expected to be less than $\frac{3}{4}$ inch and occur below the heaviest loaded structural element. Differential settlement is not expected to exceed $\frac{1}{2}$ inch over a distance of 20 feet or between adjacent foundations.
- 7.9.2 The maximum expected static settlement for structure supported on a mat foundation system deriving support in undisturbed bedrock is estimated to be less than $\frac{1}{2}$ 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 $\frac{1}{4}$ inch between the center and corner of the mat foundation.
- 7.9.3 Once the design and foundation loading configurations for the proposed structure proceeds to a more finalized plan, the estimated settlements presented in this report should be reviewed and revised, if necessary. If the final foundation loading configurations are greater than the assumed loading conditions, the potential for settlement should be reevaluated by this office.

7.10 Lateral Design

- 7.10.1 Resistance to lateral loading may be provided by friction acting at the base of foundations, slabs and by passive earth pressure. An allowable coefficient of friction of 0.30 may be used with the dead load forces in the newly placed engineered fill, undisturbed colluvium, or undisturbed bedrock.
- 7.10.2 Passive earth pressure for the sides of foundations and slabs poured against competent bedrock may be computed as an equivalent fluid having a density of 900 pounds per cubic foot (pcf), with an increase of 190 psf for each additional foot of embedment, up to a maximum earth pressure of 2,600 pcf. Passive earth pressure for the sides of foundations poured against properly compacted engineered fill or undisturbed colluvium may be computed as an equivalent fluid having a density of 190 pounds per cubic foot with a maximum earth pressure of 1,900 pounds per square foot. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

7.11 Concrete Slabs-on-Grade

- 7.11.1 Concrete slabs-on-grade subject to vehicle loading should be designed in accordance with the recommendations in the *Pavement Recommendations* section of this report (Section 7.12).
- 7.11.2 Unless specifically evaluated and designed by a qualified structural engineer, the slab-on-grade for the structure should be a minimum of 5 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 derive support in the bedrock or properly compacted engineered fill. Any bedrock that is unintentionally disturbed should be compacted for slab support. The finished subgrade must be approved in writing prior to placement of a vapor retarder, reinforcing steel, or concrete.

- 7.11.3 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 selection and 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) as well as ASTM E1745 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 is recommended. The vapor retarder should be installed in direct contact with the concrete slab with proper perimeter seal. If the California Green Building Code requirements apply to this project, the vapor retarder should be underlain by 4 inches of clean aggregate. It is important that the vapor retarder be puncture resistant since it will be in direct contact with angular gravel. As an alternative to the clean aggregate suggested in the Green Building Code, it is our opinion that the concrete slab-on-grade may be underlain by a vapor retarder over 4-inches of clean sand (sand equivalent greater than 30), since the sand will serve a capillary break and will minimize the potential for punctures and damage to the vapor barrier.
- 7.11.4 Due to the nature of the proposed design and intent for a partial subterranean level, waterproofing of subterranean walls and slabs is suggested. Particular care should be taken in the design and installation of waterproofing to avoid moisture problems, or actual water seepage into the structure through any normal shrinkage cracks which may develop in the concrete walls, floor slab, foundations and/or construction joints. The design and inspection of the waterproofing is not the responsibility of the geotechnical engineer. A waterproofing consultant should be retained in order to recommend a product or method, which would provide protection to subterranean walls, floor slabs and foundations.
- 7.11.5 For seismic design purposes, a coefficient of friction of 0.30 may be utilized between concrete slabs and bedrock without a moisture barrier, and 0.15 for slabs underlain by a moisture barrier.

- 7.11.6 Exterior slabs for walkways or flatwork, not subject to traffic loads, should be at least 4 inches thick and reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions, positioned near the slab midpoint. Prior to construction of slabs, the upper 12 inches of subgrade should be moistened to two percent above optimum moisture content and properly compacted to at least 92 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition). Crack control joints should be spaced at intervals not greater than 10 feet and should be constructed using saw-cuts or other methods as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. The project structural engineer should design construction joints as necessary.
- 7.11.7 Due to the expansive potential of the anticipated subgrade soils, the moisture content of the slab subgrade should be maintained and sprinkled as necessary to maintain a moist condition as would be expected in any concrete placement. Furthermore, consideration should be given to doweling slabs into adjacent curbs and foundations to minimize movements and offsets which could lead to a potential tripping hazard. As an alternative, the upper 18 inches of soil could be replaced with granular, non-expansive soils which will reduce the potential for movements and offsets.
- 7.11.8 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.

7.12 Preliminary Pavement Recommendations

- 7.12.1 Where new paving is to be placed, it is recommended that all existing fill and soft materials be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all existing artificial fill and soft soils in the area of new paving is not required; however, paving constructed over existing uncertified fill or unsuitable colluvium, material may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper 12 inches of paving subgrade should be scarified, moisture conditioned to two percent above optimum moisture content, and properly compacted to at least 92 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition).

- 7.12.2 The following pavement sections are based on an assumed R-Value of 20. Once site grading activities are complete an R-Value should be obtained by laboratory testing to confirm the properties of the soils serving as paving subgrade, prior to placing pavement.
- 7.12.3 The Traffic Indices listed below are estimates. Geocon does not practice in the field of traffic engineering. The actual Traffic Index for each area should be determined by the project civil engineer. If pavement sections for Traffic Indices other than those listed below are required, Geocon should be contacted to provide additional recommendations. Pavement thicknesses were determined following procedures outlined in the *California Highway Design Manual* (Caltrans). It is anticipated that the majority of traffic will consist of automobile and large truck traffic.

PRELIMINARY PAVEMENT DESIGN SECTIONS

Location	Estimated Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
Automobile Parking and Driveways	4.0	3.0	4.0
Trash Truck & Fire Lanes	7.0	4.0	12.0

- 7.12.4 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 lieu of Class 2 aggregate base is acceptable. Crushed Miscellaneous Base should conform to Section 200-2.4 of the “*Standard Specifications for Public Works Construction*” (Green Book).
- 7.12.5 Unless specifically designed and evaluated by the project structural engineer, where exterior concrete paving will be utilized for support of vehicles, it is recommended that the concrete be a minimum of 5 inches of concrete reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions. Concrete paving supporting vehicular traffic should be underlain by a minimum of 4 inches of aggregate base and a properly compacted subgrade. The subgrade and base material should be compacted to 92 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition).
- 7.12.6 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.

7.13 Retaining Wall Design

- 7.13.1 The recommendations presented below are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 14 feet. In the event that walls significantly higher than 14 feet are planned, Geocon should be contacted for additional recommendations.
- 7.13.2 Retaining wall foundations may be designed in accordance with the recommendations provided in the *Foundation Design* sections of this report (see Sections 7.7 and 7.8).
- 7.13.3 Based on the orientation of the proposed excavation with respect to the strike and dip of the bedrock, the apparent dips for north and west retaining walls are anticipated to range from approximately 26° to 36°. The retaining wall recommendations presented below include consideration of the surcharge due to out-of-slope bedding on the north and west retaining wall. Excavations into bedrock should be observed and approved in writing by the Project Geologist (a representative of Geocon West, Inc.) during excavation to check for the presence of jointing or bedding which may require revised retaining wall recommendations.
- 7.13.4 Retaining walls with a level backfill surface that are not restrained at the top should be designed utilizing a triangular distribution of pressure (active pressure). Restrained walls are those that are not allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall in feet) at the top of the wall. Where walls are restrained from movement at the top, walls may be designed utilizing a triangular distribution of pressure (at-rest pressure). The table below presents recommended pressures to be used in retaining wall design, assuming that proper drainage will be maintained. Calculation of the recommended wall pressures are provided on Figures 5 and 6.

RETAINING WALL WITH LEVEL BACKFILL SURFACE (East and South Retaining Walls – Favorable Bedding)

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 14	30	68

RETAINING WALL WITH SURCHARGE FROM JOINTING OR BEDDING (North and West Retaining Walls – Adverse Bedding)

HEIGHT OF RETAINING WALL (FEET)	EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (ACTIVE PRESSURE)	EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (AT-REST PRESSURE)
Up to 14	55	83

- 7.13.5 The wall pressures provided above assume that the proposed retaining walls will support relatively undisturbed colluvial soils and bedrock. If sloping techniques are to be utilized for construction of proposed walls, which would result in a wedge of engineered fill behind the retaining walls, revised earth pressures may be required to account for the expansive potential of the soil placed as engineered fill. This should be evaluated once the use of sloping measures is established and once the geotechnical characteristics of the engineered backfill soils can be further evaluated.
- 7.13.6 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 95 pcf. The value includes hydrostatic pressures plus buoyant lateral earth pressures.
- 7.13.7 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.
- 7.13.8 It is recommended that line-load surcharges from adjacent wall footings, use horizontal pressures generated from NAV-FAC DM 7.2. The governing equations are:

For $x/H \leq 0.4$

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

and

For $x/H > 0.4$

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

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

- 7.13.9 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:

$$\text{For } x/H \leq 0.4$$

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

and

$$\text{For } x/H > 0.4$$

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

then

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

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

- 7.13.10 In addition to the recommended earth pressure, the upper 10 feet of the subterranean wall adjacent to the street and parking lot should be designed to resist a uniform lateral pressure of 100 psf, acting as a result of an assumed 300 psf surcharge behind the walls due to normal street traffic. If the traffic is kept back at least 10 feet from the subterranean walls, the traffic surcharge may be neglected.
- 7.13.11 Seismic lateral forces should be incorporated into the design as necessary, and recommendations for seismic lateral forces are presented below.

7.14 Dynamic (Seismic) Lateral Forces

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

- 7.14.2 A seismic load of 10 pcf should be used for design of walls that support more than 6 feet of backfill in accordance with Section 1803.5.12 of the 2019 CBC. The seismic load is applied as an equivalent fluid pressure along the height of the wall and the calculated loads result in a maximum load exerted at the base of the wall and zero at the top of the wall. This seismic load should be applied in addition to the active earth pressure. The earth pressure is based on half of two-thirds of PGA_M calculated from ASCE 7-16 Section 11.8.3.

7.15 Retaining Wall Drainage

- 7.15.1 Retaining walls should be provided with a drainage system. At the base of the drain system, a subdrain covered with a minimum of 12 inches of gravel should be installed, and a compacted fill blanket or other seal placed at the surface (see Figure 7). The clean bottom and subdrain pipe, behind a retaining wall, should be observed by the Geotechnical Engineer (a representative of Geocon), prior to placement of gravel or compacting backfill.
- 7.15.2 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 8). These vertical columns of drainage material would then be connected at the bottom of the wall to a collection panel or a 1-cubic-foot rock pocket drained by a 4-inch subdrain pipe.
- 7.15.3 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. Elevator pit walls may be designed in accordance with the recommendations in the *Foundation Design* and *Retaining Wall Design* sections of this report (see Sections 7.7, 7.8 and 7.13).

- 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 7.13).
- 7.16.4 It is suggested that the elevator pit 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 shoring pile, or the drilled excavation could compromise the existing foundation or pile support, especially if the drilling is performed subsequent to the foundation or pile construction.
- 7.17.2 Although caving is not anticipated in the bedrock, casing may be required if groundwater seepage is encountered. 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 16 feet in vertical height may be required for excavation and construction of the partial subterranean level, including foundation depths. Excavation is anticipated to expose artificial fill, colluvial soils and bedrock, which are considered suitable for vertical excavations up to 5 feet in height where loose soils or caving sands are not present, and where excavations are not surcharged by adjacent traffic or structures. Where bedrock with adverse bedding is exposed in excavations, vertical cuts should not exceed 3 feet in height.
- 7.18.2 Excavations into bedrock should be observed by the Project Geologist (a representative of Geocon West, Inc.) during excavation to check for the presence of jointing or bedding which

may require special recommendations for sloping and/or shoring. Any recommendations deemed necessary will be provided at that time.

- 7.18.3 Vertical excavations greater than the permissible heights outlined above will require sloping and/or shoring measures in order to provide a stable excavation. Where sufficient space is available, temporary unsurcharged embankments could be sloped back at a uniform 1.5:1 (H:V) slope gradient or flatter up to a maximum height of 10 feet. Temporary unsurcharged embankments may also be sloped back at a uniform 2:1 (H:V) slope gradient or flatter up to a maximum height of 20 feet. A uniform slope does not have a vertical portion. Where space is limited, shoring measures will be required. Shoring recommendations are provided in Section 7.19 of this report.
- 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. The soils exposed in the cut slopes should be inspected during excavation by our personnel 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 Shoring – Soldier Pile Design and Installation

- 7.19.1 The following information on the design and installation of shoring is preliminary. Review of the final shoring plans and specifications should be made by this office prior to bidding or negotiating with a shoring contractor.
- 7.19.2 Installation of shoring on a sloping ground surface requires careful consideration of excavation sequencing. Prior to installation of shoring, grading to create a relatively flat pad for equipment access may be required. Excavation of unsupported vertical cuts into a sloping ground surface would remove support from the ascending portion of the slope and create a potentially unstable condition. Unsupported vertical excavation into a slope is not permitted. Equipment access can be created by placement of additional fill to build up a temporary equipment pad against the existing slope.

- 7.19.3 One method of shoring would consist of steel soldier piles, placed in drilled holes and backfilled with concrete. Installation of shoring piles by vibration is not recommended at this site due to the dense nature of the underlying bedrock. Where maximum excavation heights are less than 12 feet the soldier piles are typically designed as cantilevers. Where excavations exceed 12 feet or are surcharged, soldier piles may require lateral bracing utilizing drilled tie-back anchors or raker braces to maintain an economical steel beam size and prevent excessive deflection. The size of the steel beam, the need for lateral bracing, and the acceptable shoring deflection should be determined by the project shoring engineer. Should it be determined that temporary tiebacks or rakers are necessary, additional recommendations can be provided under separate cover.
- 7.19.4 The design embedment of the shoring pile toes must be maintained during excavation activities. The toes of the perimeter shoring piles should be deepened to take into account any required excavations necessary for grading activities, foundations, and/or adjacent drainage systems.
- 7.19.5 The proposed soldier piles may also be designed as permanent piles. The required pile depth, dimension, spacing should be determined and designed by the project structural and shoring engineers. All piles utilized for shoring can also be incorporated into a permanent retaining wall system (shotcrete wall) and should be designed in accordance with the earth pressure provided in the *Retaining Walls* section of this report (see Section 7.13).
- 7.19.6 Drilled cast-in-place soldier piles should be placed no closer than three diameters on center. The minimum diameter of the piles is 18 inches. Structural concrete should be used for the soldier piles below the excavation; lean-mix concrete may be employed above that level. As an alternative, lean-mix concrete may be used throughout the pile where the reinforcing consists of a wideflange section. The slurry must be of sufficient strength to impart the lateral bearing pressure developed by the wideflange section to the soil. For design purposes, an allowable passive value for the bedrock below the bottom plane of excavation may be assumed to be 900 pcf, with an increase of 190 psf for each additional foot of embedment. The allowable passive value may be doubled for isolated piles, spaced a minimum of 3 times the pile diameter. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed bedrock.
- 7.19.7 Casing may be required if caving is experienced 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. If casing is used, extreme care should be employed so that the pile is not pulled apart as the casing is withdrawn. At no time should the distance between the surface of the concrete and the bottom of the casing be less than 5 feet. Continuous observation of the drilling and pouring of the piles by the Geotechnical Engineer (a representative of Geocon West, Inc.), is required.

- 7.19.8 The contractor should be aware that some difficult drilling conditions could be encountered in the bedrock which could require coring and jack-hammering. The contractor should be prepared for these conditions prior to commencement of drilling activities.
- 7.19.9 Groundwater seepage was not encountered at the time of the site exploration. However, the contractor should be prepared for groundwater during pile installation should the need arise. Piles placed below the water level require the use of a tremie to place the concrete into the bottom of the hole. A tremie should consist of a rigid, water-tight tube having a diameter of not less than 6 inches with a hopper at the top. The tube should be equipped with a device that will close the discharge end and prevent water from entering the tube while it is being charged with concrete. The tremie should be supported so as to permit free movement of the discharge end over the entire top surface of the work and to permit rapid lowering when necessary to retard or stop the flow of concrete. The discharge end should be closed at the start of the work to prevent water entering the tube and should be entirely sealed at all times, except when the concrete is being placed. The tremie tube should be kept full of concrete. The flow should be continuous until the work is completed and the resulting concrete seal should be monolithic and homogeneous. The tip of the tremie tube should always be kept about 5 feet below the surface of the concrete and definite steps and safeguards should be taken to insure that the tip of the tremie tube is never raised above the surface of the concrete.
- 7.19.10 A special concrete mix should be used for concrete to be placed below water. The design should provide for concrete with an unconfined compressive strength psi of 1,000 pounds per square inch (psi) over the initial job specification. An admixture that reduces the problem of segregation of paste/aggregates and dilution of paste should be included. The slump should be commensurate to any research report for the admixture, provided that it should also be the minimum for a reasonable consistency for placing when water is present.
- 7.19.11 The frictional resistance between the soldier piles and retained soil may be used to resist the vertical component of the anchor load. The coefficient of friction may be taken as 0.30 based on uniform contact between the steel beam and retained earth. The portion of soldier piles below the plane of excavation may also be employed to resist the downward loads. The downward capacity may be determined using a frictional resistance of 360 pounds per square foot.
- 7.19.12 Due to the nature of the site soils, it is expected that continuous lagging between soldier piles will be required. However, it is recommended that the exposed soils be observed by the Geotechnical Engineer (a representative of Geocon West, Inc.), to verify the presence of any cohesive soils and the areas where lagging may be omitted.

- 7.19.13 The time between lagging excavation and lagging placement should be as short as possible. Soldier piles should be designed for the full-anticipated pressures. Due to arching in the soils, the pressure on the lagging will be less. It is recommended that the lagging be designed for the full design pressure but be limited to a maximum of 400 pounds per square foot.
- 7.19.14 For the design of shoring, it is recommended that an equivalent fluid pressure based on the following tables, be utilized for design. A trapezoidal distribution of lateral earth pressure may be used where shoring will be restrained at the top by bracing or tie backs. The recommended active and trapezoidal pressure are provided in the following tables. A diagram depicting the trapezoidal pressure distribution of lateral earth pressure is provided below the table. Calculation of the recommended shoring pressures are provided on Figures 9 and 10.

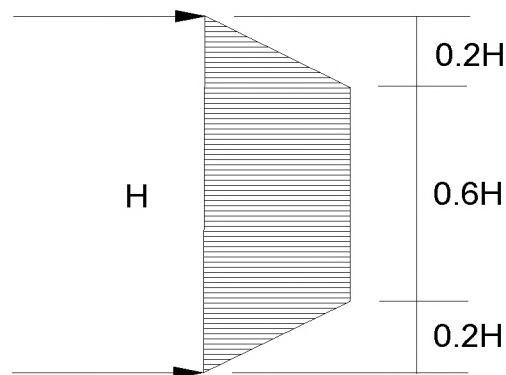
**SHORING WALL WITH LEVEL BACKFILL SURFACE
(East and South Shoring Walls – Favorable Bedding)**

HEIGHT OF SHORING (FEET)	EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (ACTIVE PRESSURE) Triangular	EQUIVALENT FLUID PRESSURE (Pounds Per Square Foot per Foot) Trapezoidal (Where H is the height of the shoring in feet)
Up to 16	25	$16H$

**SHORING WALL WITH LEVEL BACKFILL SURFACE
(North and West Shoring Walls – Adverse Bedding)**

HEIGHT OF SHORING (FEET)	EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (ACTIVE PRESSURE) Triangular	EQUIVALENT FLUID PRESSURE (Pounds Per Square Foot per Foot) Trapezoidal (Where H is the height of the shoring in feet)
Up to 16	25	$16H$

Trapezoidal Distribution of Pressure



- 7.19.15 Where a combination of sloped embankment and shoring is utilized, the pressure will be greater and must be determined for each combination. Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic, or adjacent structures and must be determined for each combination.
- 7.19.16 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:

$$\text{For } x/H \leq 0.4$$

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

and

$$\text{For } x/H > 0.4$$

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

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

- 7.19.17 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:

$$\begin{aligned} & \text{For } x/H \leq 0.4 \\ & \sigma_H(z) = \frac{0.28 \times \left(\frac{z}{H}\right)^2}{\left[0.16 + \left(\frac{z}{H}\right)^2\right]^3} \times \frac{Q_P}{H^2} \\ & \text{and} \\ & \text{For } x/H > 0.4 \\ & \sigma_H(z) = \frac{1.77 \times \left(\frac{x}{H}\right)^2 \times \left(\frac{z}{H}\right)^2}{\left[\left(\frac{x}{H}\right)^2 + \left(\frac{z}{H}\right)^2\right]^3} \times \frac{Q_P}{H^2} \\ & \text{then} \\ & \sigma'_H(z) = \sigma_H(z) \cos^2(1.1\theta) \end{aligned}$$

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

- 7.19.18 In addition to the recommended earth pressure, the upper 10 feet of the shoring adjacent to the street should be designed to resist a uniform lateral pressure of 100 psf, acting as a result of an assumed 300 psf surcharge behind the shoring due to normal street traffic. If the traffic is kept back at least ten feet from the shoring, the traffic surcharge may be neglected.
- 7.19.19 It is difficult to accurately predict the amount of deflection of a shored embankment. It should be realized that some deflection will occur. It is recommended that the deflection be minimized to prevent damage to existing structures and adjacent improvements. Where public right-of-ways are present or adjacent offsite structures do not surcharge the shoring excavation, the shoring deflection should be limited to less than 1 inch at the top of the shored embankment. Where offsite structures are within the shoring surcharge area it is recommended that the beam deflection be limited to less than ½ inch at the elevation of the adjacent offsite foundation, and no deflection at all if deflections will damage existing structures. The allowable deflection will be assessed and designed by the project shoring engineer.

- 7.19.20 Because of the depth of the excavation, some means of monitoring the performance of the shoring system is suggested. The monitoring should consist of periodic surveying of the lateral and vertical locations of the tops of all soldier piles and the lateral movement along the entire lengths of selected soldier piles.
- 7.19.21 Due to the depth of the excavation and proximity to adjacent structures, it is suggested that prior to excavation the existing improvements be inspected to document the present condition. For documentation purposes, photographs should be taken of preconstruction distress conditions and level surveys of adjacent grade and pavement should be considered. During excavation activities, the adjacent structures and pavement should be periodically inspected for signs of distress. In the event that distress or settlement is noted, an investigation should be performed and corrective measures taken so that continued or worsened distress or settlement is mitigated. Documentation and monitoring of the offsite structures and improvements is not the responsibility of the geotechnical engineer.

7.20 Surface Drainage

- 7.20.1 Proper surface drainage is critical to the future performance of the project. Uncontrolled infiltration of irrigation excess and storm runoff into the foundation 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 in building areas should be maintained at all times.
- 7.20.2 All site drainage should be collected and controlled in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2019 CBC 1804.4 or other applicable standards. In addition, drainage should not be allowed to flow uncontrolled over any descending slope. The proposed structure should be provided with roof gutters. Discharge from downspouts, roof drains and scuppers not recommended onto unprotected soils within five feet of the building perimeter. Planters which are located adjacent to foundations should be sealed to prevent moisture intrusion into the engineered fill providing foundation support. Landscape irrigation is not recommended within 5 feet of the building perimeter footings except when enclosed in protected planters.
- 7.20.3 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.

- 7.20.4 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.

7.21 Plan Review

- 7.21.1 Grading, foundation, and shoring plans should be reviewed by the Geotechnical Engineer (a representative of Geocon), 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

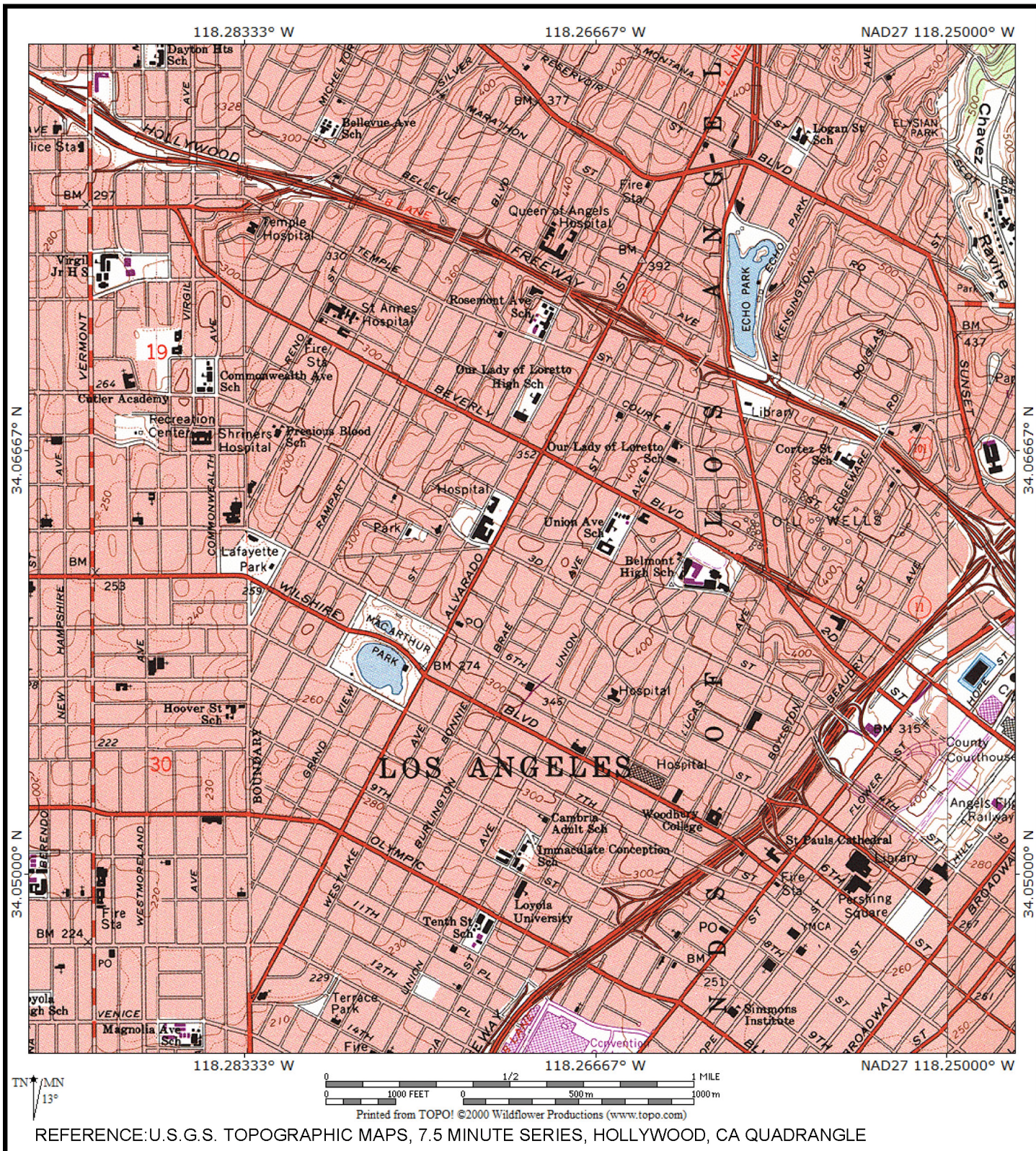
1. The recommendations of this report pertain only to the site investigated and are based upon the assumption that the soil conditions do not deviate from those disclosed in the investigation. If any variations or undesirable conditions are encountered during construction, or if the proposed construction 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 this 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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CHECKED BY: SFK

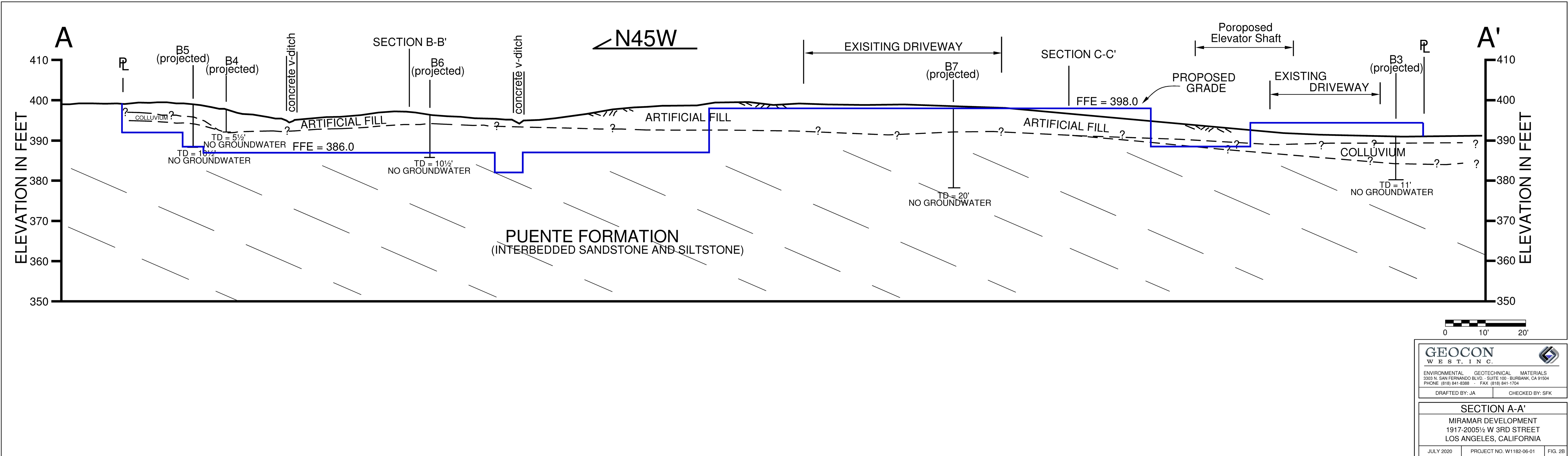
VICINITY MAP

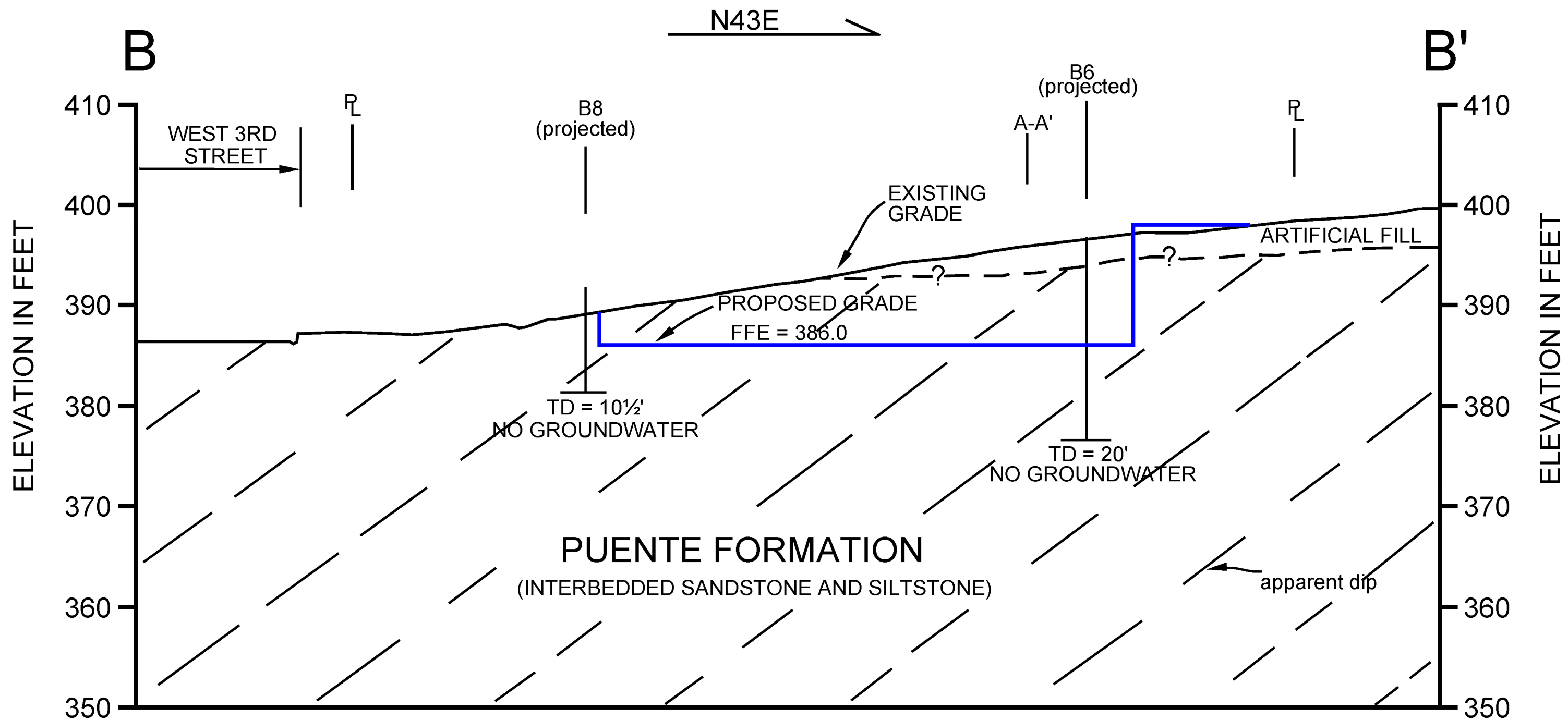
MIRAMAR DEVELOPMENT
1917-2005 1/2 WEST 3RD STREET
LOS ANGELES, CALIFORNIA

JULY 2020

PROJECT NO. W1182-06-01

FIG. 1





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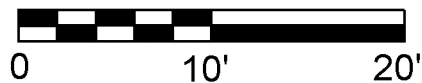
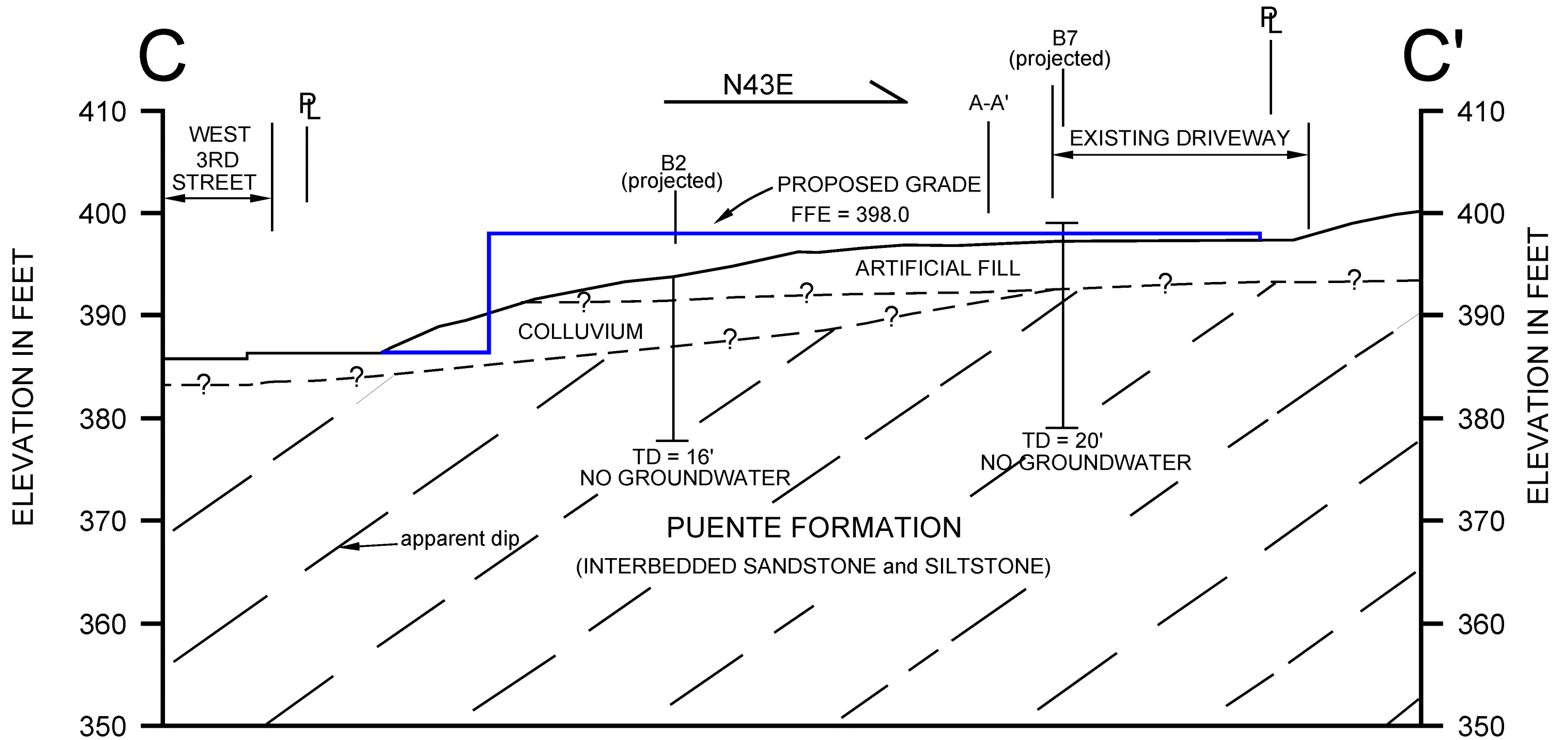
SECTION B-B'

MIRAMAR DEVELOPMENT
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FIG. 2C



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DRAFTED BY: JA

CHECKED BY: JTA

SECTION C-C'

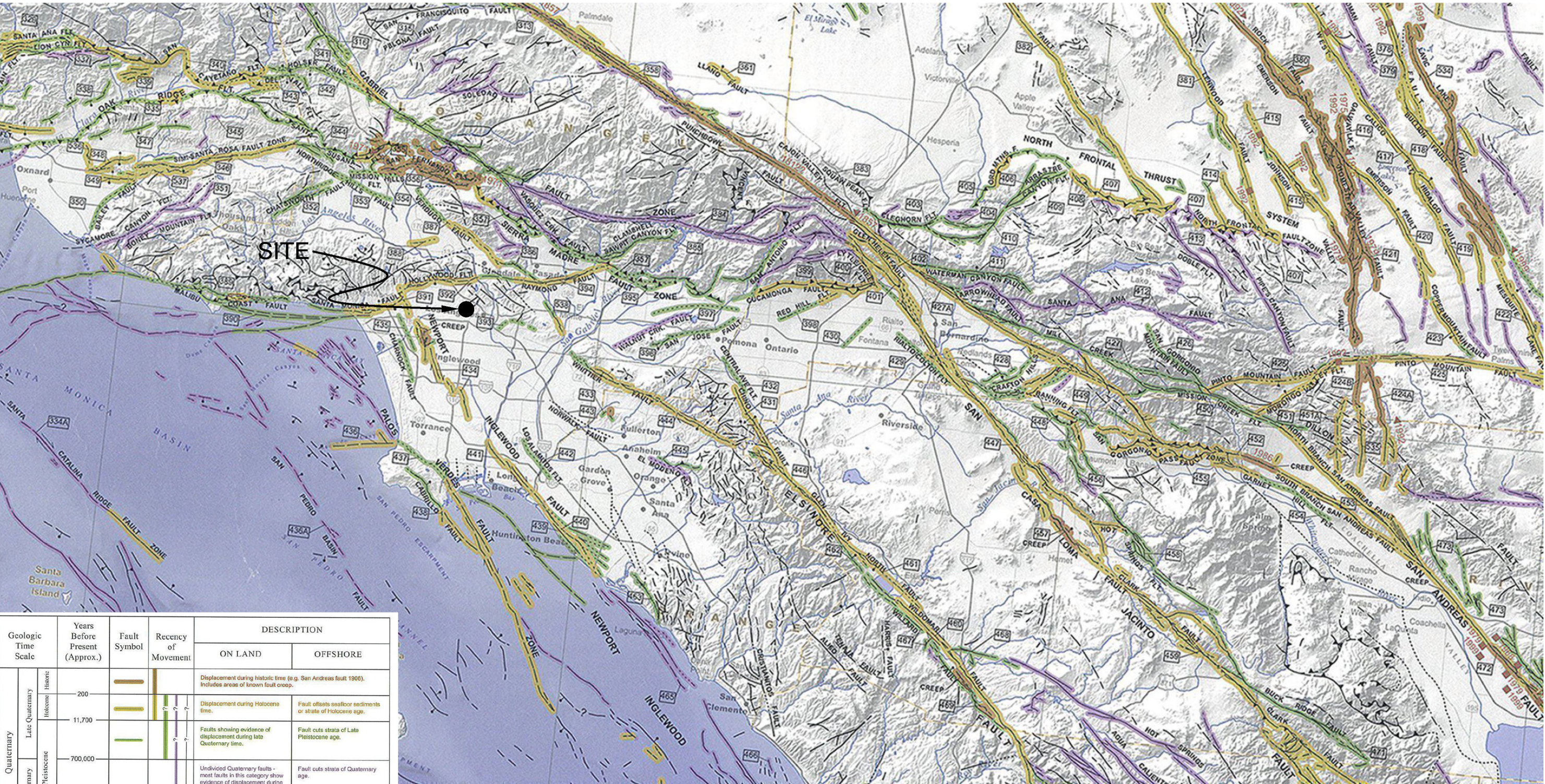
MIRAMAR DEVELOPMENT
1917-2005 1/2 WEST 3RD STREET
LOS ANGELES, CALIFORNIA

JULY 2020

PROJECT NO. W1182-06-01

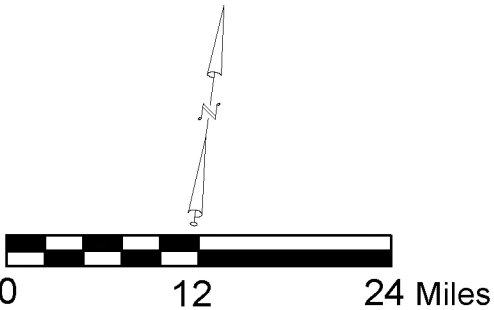
FIG. 2D

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



Geologic Time Scale		Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION	
					ON LAND	OFFSHORE
Quaternary	Late Quaternary	200			Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
	Holocene	11,700			Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
	Pleistocene	700,000			Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
Pre-Quaternary	Early Quaternary	1,600,000			Undiscovered 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.
		4.5 billion (Age of Earth)			Faults without recognized Quaternary displacement or showing evidence of no displacement during Quaternary time. Not necessarily inactive.	Fault cuts strata of Pliocene or older age.

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



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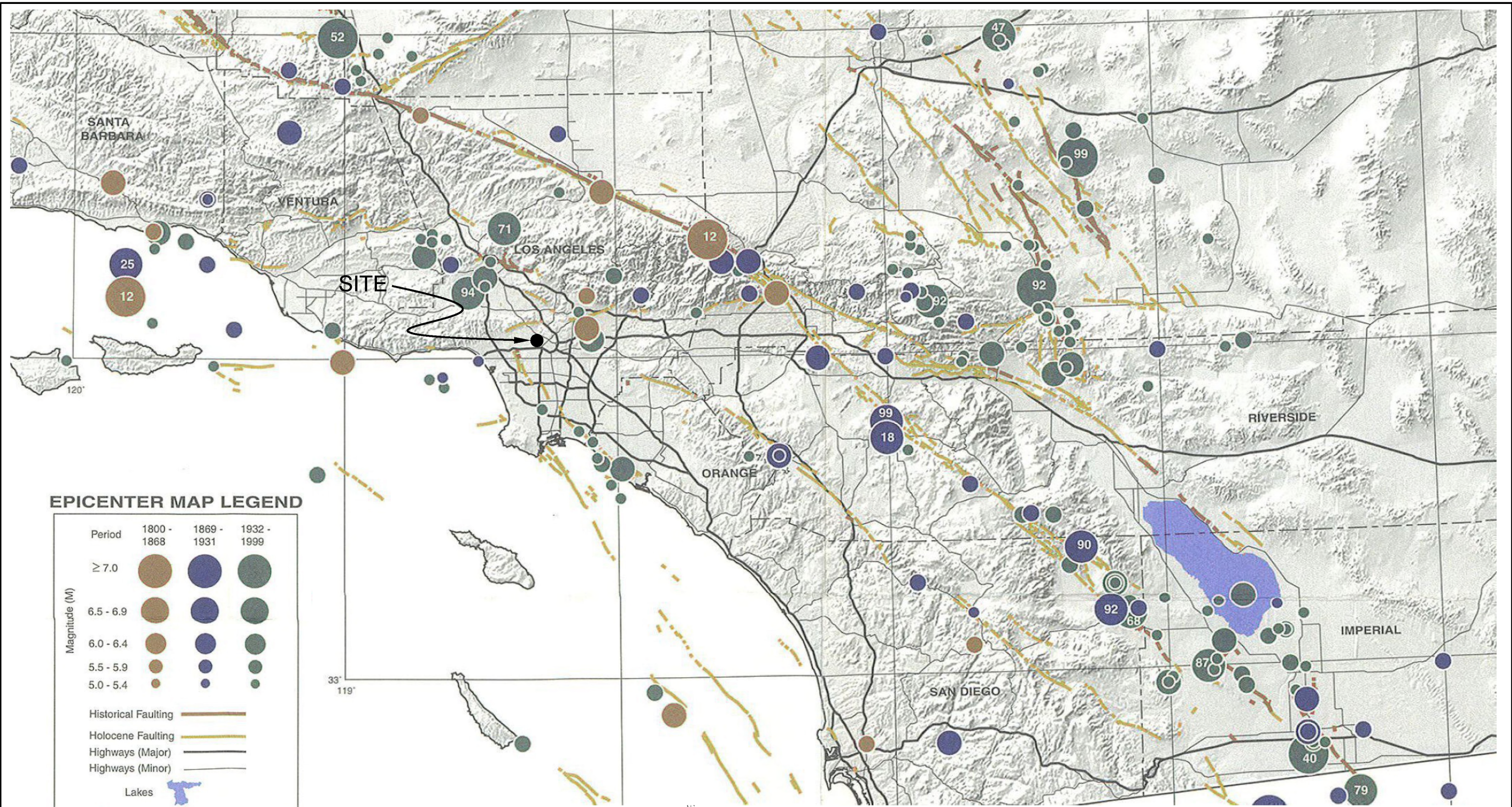
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REGIONAL FAULT MAP

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LOS ANGELES, CALIFORNIA

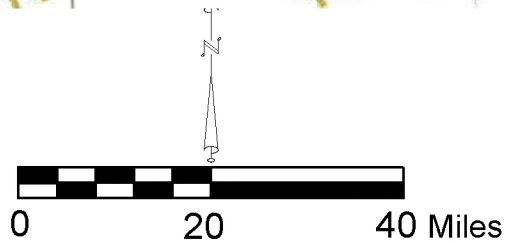
JULY 2020 PROJECT NO. W1182-06-01 FIG. 3



EPICENTER MAP LEGEND

Period	1800 - 1868	1869 - 1931	1932 - 1999
≥ 7.0			
6.5 - 6.9			
6.0 - 6.4			
5.5 - 5.9			
5.0 - 5.4			
Historical Faulting			
Holocene Faulting			
Highways (Major)			
Highways (Minor)			
Lakes			
	Last two digits of M ≥ 6.5 earthquake year		

Reference: Topozada, 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

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JULY 2020

PROJECT NO. W1182-06-01

FIG.4

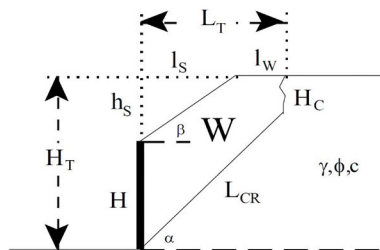
Retaining Wall Design with Transitioned Backfill (Vector Analysis)

Input:

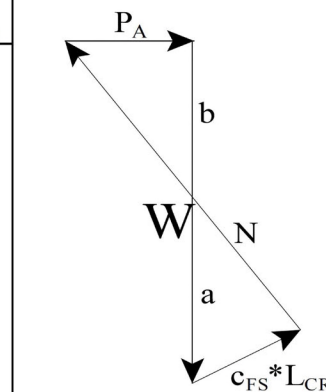
Retaining Wall Height (H) 14.00 feet
 Slope Angle of Backfill (b) 0.0 degrees
 Height of Slope above Wall (h_s) 0.0 feet
 Horizontal Length of Slope (l_s) 0.0 feet
 Total Height (Wall + Slope) (H_T) 14.0 feet

Unit Weight of Retained Soils (g) 115.0 pcf
 Friction Angle of Retained Soils (f) 24.0 degrees
 Cohesion of Retained Soils (c) 364.0 psf
 Factor of Safety (FS) 1.50

Factored Parameters: (f_{FS}) 16.5 degrees
 (c_{FS}) 242.7 psf



Failure Angle (a) degrees	Height of Tension Crack (H _c) feet	Area of Wedge (A) feet ²	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L _{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P _A) lbs/lineal foot
45	6.0	80	9198.8	11.3	5520.4	3678.4	1994.6
46	5.9	78	8937.5	11.2	5312.0	3625.5	2048.5
47	5.8	75	8674.6	11.1	5112.8	3561.8	2095.4
48	5.8	73	8411.1	11.0	4922.5	3488.6	2135.2
49	5.7	71	8147.8	10.9	4740.7	3407.2	2167.9
50	5.7	69	7885.4	10.8	4566.9	3318.4	2193.8
51	5.7	66	7624.2	10.7	4400.8	3223.4	2212.7
52	5.7	64	7364.6	10.6	4241.9	3122.7	2224.8
53	5.7	62	7106.8	10.4	4089.6	3017.2	2230.0
54	5.7	60	6851.0	10.3	3943.5	2907.5	2228.4
55	5.7	57	6597.2	10.2	3803.1	2794.1	2220.0
56	5.7	55	6345.5	10.0	3668.0	2677.6	2204.7
57	5.7	53	6095.9	9.9	3537.6	2558.3	2182.5
58	5.8	51	5848.3	9.7	3411.5	2436.8	2153.5
59	5.8	49	5602.5	9.5	3289.2	2313.3	2117.4
60	5.9	47	5358.6	9.4	3170.2	2188.3	2074.3
61	6.0	44	5116.2	9.2	3054.1	2062.1	2024.2
62	6.0	42	4875.3	9.0	2940.4	1935.0	1966.8
63	6.1	40	4635.7	8.8	2828.4	1807.2	1902.3
64	6.3	38	4397.0	8.6	2717.8	1679.1	1830.4
65	6.4	36	4159.0	8.4	2608.0	1551.1	1751.2
66	6.5	34	3921.5	8.2	2498.2	1423.3	1664.6
67	6.7	32	3684.1	7.9	2387.9	1296.2	1570.6
68	6.9	30	3446.3	7.7	2276.3	1170.0	1469.3
69	7.1	28	3207.8	7.4	2162.5	1045.3	1360.7
70	7.4	26	2968.0	7.1	2045.5	922.5	1245.2



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

$P_{A, max}$

2230.0 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2 * P_A / H^2$

EFP

22.8 pcf

At-Rest = $\gamma * (1 - \sin(\phi))$

68.2 pcf

Design Wall for an Equivalent Fluid Pressure:

30 pcf

68 pcf

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DRAFTED BY: JMH

CHECKED BY: JTA

RETAINING WALL - FAVORABLE BEDDING

MIRAMAR DEVELOPMENT
 1917-2005 1/2 WEST 3RD STREET
 LOS ANGELES, CALIFORNIA

JULY 2020

PROJECT NO. W1182-06-01

FIG. 5

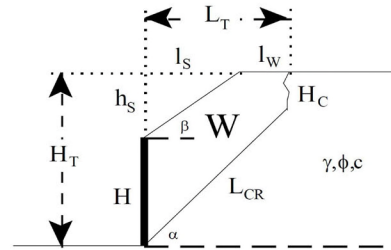
Retaining Wall Design with Transitioned Backfill (Vector Analysis)

Input:

Retaining Wall Height (H) 14.00 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) 14.0 feet

Unit Weight of Retained Soils (g) 115.0 pcf
 Friction Angle of Retained Soils (f) 16.0 degrees
 Cohesion of Retained Soils (c) 120.0 psf
 Factor of Safety (FS) 1.50

Factored Parameters: (f_{FS}) 10.8 degrees
 (c_{FS}) 80.0 psf



Failure Angle (a) degrees	Height of Tension Crack (H _c) feet	Area of Wedge (A) feet ²	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L _{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P _A) lbs/lineal foot
25	3.1	200	23000.3	25.8	8291.0	14709.3	3715.9
26	2.9	192	22112.9	25.3	7597.0	14515.9	3937.8
27	2.8	185	21263.7	24.8	6987.2	14276.5	4141.7
28	2.6	178	20453.3	24.2	6449.1	14004.2	4329.0
29	2.5	171	19681.1	23.7	5972.5	13708.6	4501.2
30	2.4	165	18945.7	23.2	5548.6	13397.1	4659.5
31	2.3	159	18245.3	22.7	5170.1	13075.2	4805.0
32	2.2	153	17578.1	22.2	4830.9	12747.1	4938.6
33	2.2	147	16941.8	21.7	4525.9	12415.9	5061.2
34	2.1	142	16334.6	21.3	4250.7	12083.9	5173.6
35	2.0	137	15754.6	20.9	4001.6	11753.0	5276.5
36	2.0	132	15199.9	20.4	3775.4	11424.5	5370.5

Design Equations (Vector Analysis):
 $a = c_{FS} * L_{CR} * \sin(90 + f_{FS}) / \sin(a - f_{FS})$
 $b = W - a$
 $P_A = b * \tan(a - f_{FS})$
 $EFP = 2 * P_A / H^2$

Maximum Active Pressure Resultant

$P_{A, max}$

5370.5 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2 * P_A / H^2$

EFP

54.8 pcf

At-Rest = $\gamma * (1 - \sin(\phi))$

83.3 pcf

Design Wall for an Equivalent Fluid Pressure:

55 pcf

83 pcf

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CHECKED BY: JTA

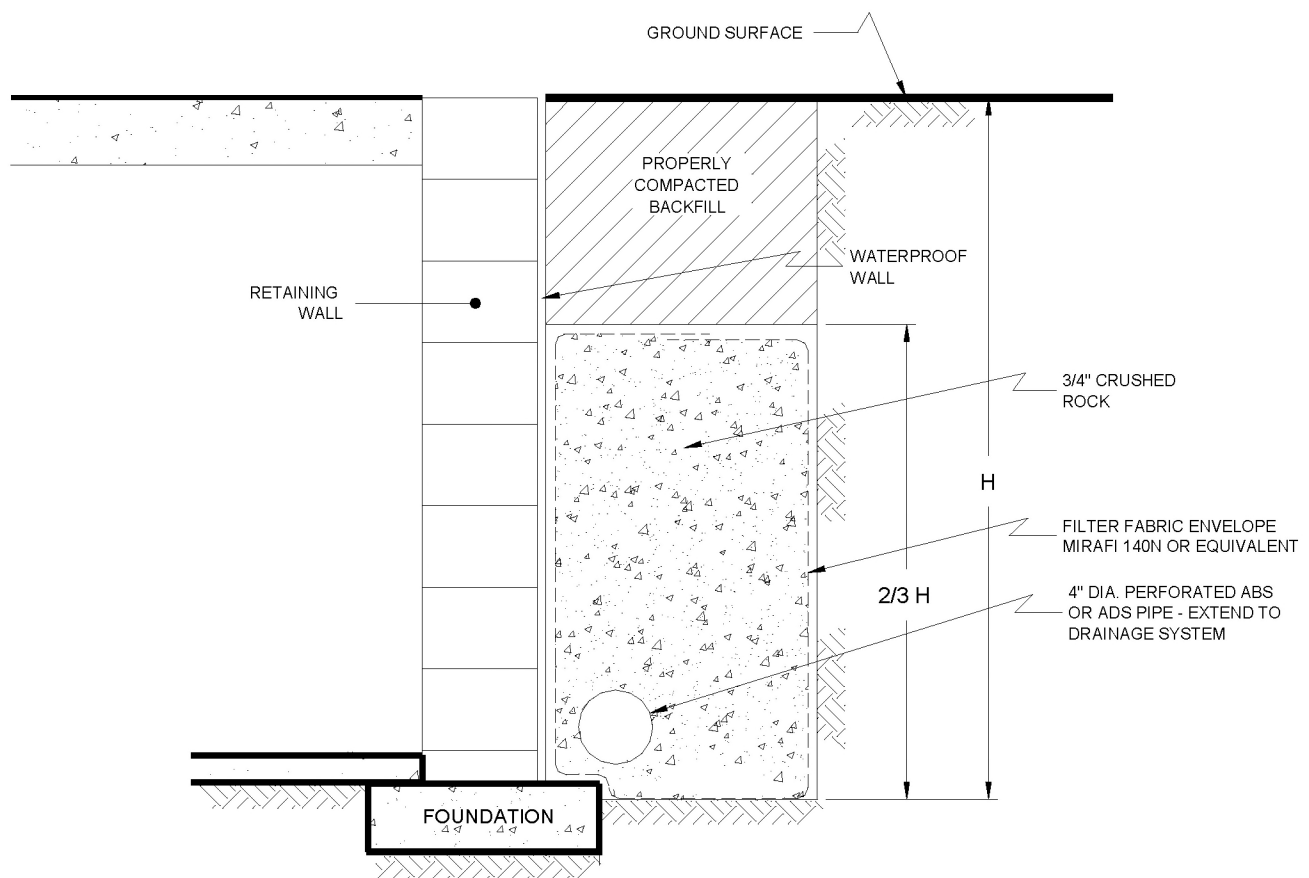
RETAINING WALL - ADVERSE BEDDING

MIRAMAR DEVELOPMENT
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 LOS ANGELES, CALIFORNIA

JULY 2020

PROJECT NO. W1182-06-01

FIG. 6



NO SCALE

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CHECKED BY: JTA

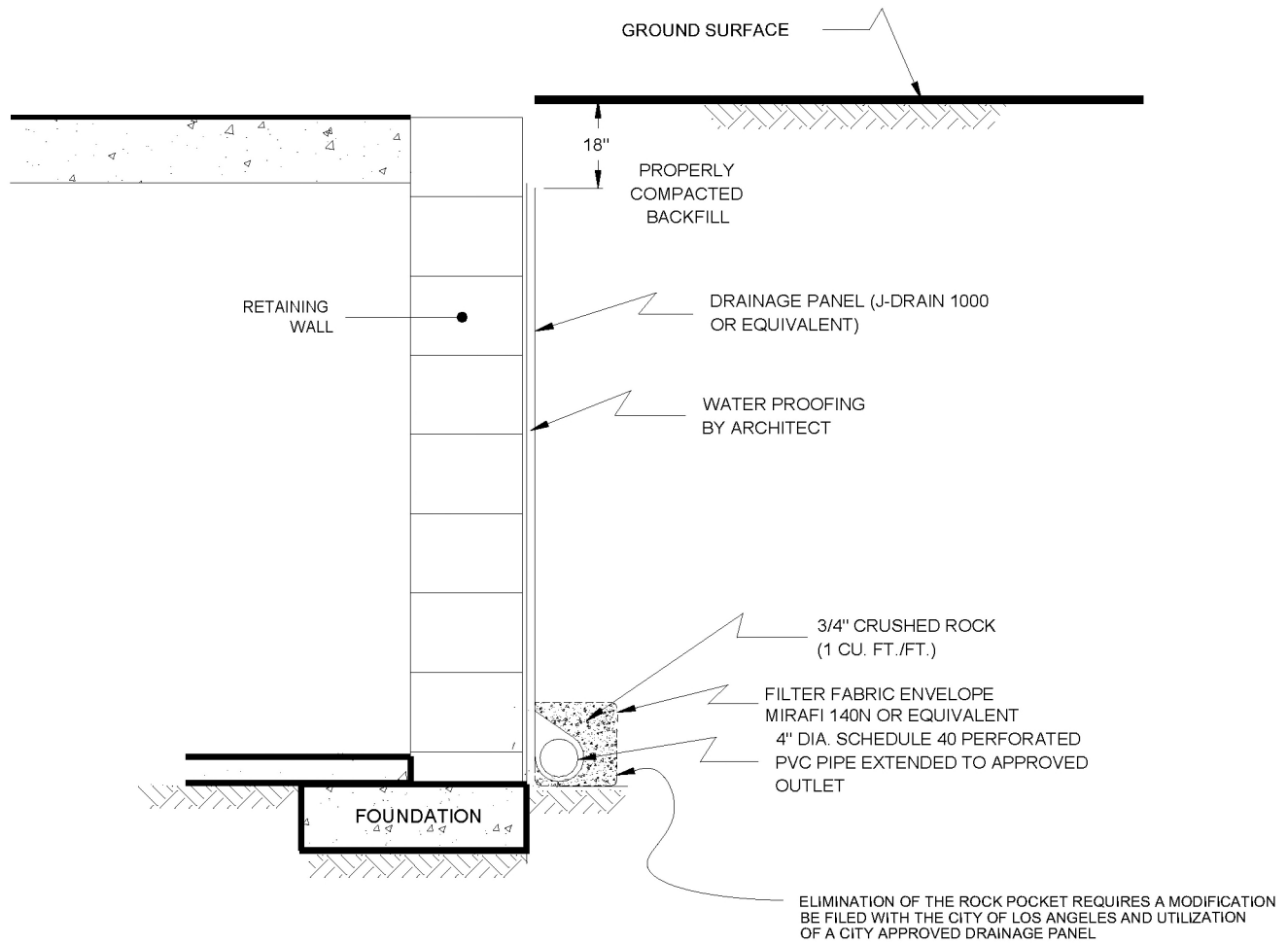
RETAINING WALL DRAIN DETAIL

MIRAMAR DEVELOPMENT
1917-2005 1/2 WEST 3RD STREET
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FIG. 7



NO SCALE

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RETAINING WALL DRAIN DETAIL

MIRAMAR DEVELOPMENT
1917-2005 1/2 WEST 3RD STREET
LOS ANGELES, CALIFORNIA

JULY 2020

PROJECT NO. W1182-06-01

FIG. 8

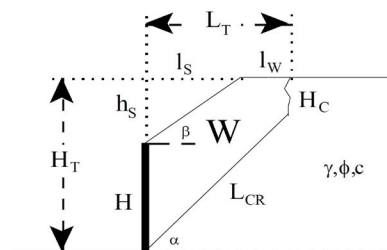
Shoring Design with Transitioned Backfill (Vector Analysis)

Input:

Shoring Height (H) **16.00** feet
 Slope Angle of Backfill (b) **0.0** degrees
 Height of Slope above Shoring (h_s) **0.0** feet
 Horizontal Length of Slope (l_s) **0.0** feet
 Total Height (Shoring + Slope) (H_T) **16.0** feet

Unit Weight of Retained Soils (g) **125.0** pcf
 Friction Angle of Retained Soils (f) **25.8** degrees
 Cohesion of Retained Soils (c) **712.0** psf
 Factor of Safety (FS) **1.25**

Factored Parameters: (f_{FS}) **21.1** degrees
 (c_{FS}) **569.6** 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	14.9	18	2197.3	1.6	2116.2	81.1	35.9
46	14.6	21	2665.0	2.0	2538.9	126.0	58.4
47	14.3	24	3020.5	2.3	2849.9	170.6	82.7
48	14.1	26	3282.4	2.6	3070.5	211.9	107.3
49	13.9	28	3465.7	2.8	3217.8	247.9	131.0
50	13.7	29	3582.5	3.0	3305.1	277.4	152.9
51	13.6	29	3642.8	3.1	3342.9	300.0	172.2
52	13.5	29	3654.7	3.2	3339.6	315.2	188.3
53	13.4	29	3625.1	3.3	3301.8	323.2	200.9
54	13.3	28	3559.3	3.3	3234.9	324.4	209.5
55	13.3	28	3462.0	3.3	3143.0	319.0	214.0
56	13.3	27	3337.0	3.3	3029.2	307.7	214.3
57	13.3	25	3187.3	3.2	2896.2	291.1	210.4
58	13.4	24	3015.6	3.1	2745.7	269.9	202.3
59	13.4	23	2823.9	3.0	2579.0	244.9	190.3
60	13.5	21	2613.7	2.8	2396.9	216.8	174.7
61	13.7	19	2386.5	2.7	2199.9	186.6	155.8
62	13.8	17	2143.2	2.4	1988.0	155.2	134.3
63	14.0	15	1884.3	2.2	1760.7	123.6	110.8
64	14.3	13	1610.3	1.9	1517.3	92.9	86.2
65	14.5	11	1321.1	1.6	1256.8	64.3	61.8
66	14.8	8	1016.6	1.3	977.5	39.1	38.9
67	15.2	6	696.3	0.9	677.5	18.8	19.4
68	15.5	3	359.2	0.5	354.1	5.1	5.5
69	16.0	0	4.2	0.0	4.2	0.0	0.0
70	16.5	-3	-370.3	-0.5	-376.1	5.7	6.5

Design Equations (Vector Analysis):
 $a = c_{FS} \cdot L_{CR} \cdot \sin(90 + f_{FS}) / \sin(a - f_{FS})$
 $b = W - a$
 $P_A = b \cdot \tan(a - f_{FS})$
 $EFP = 2 \cdot P_A / H^2$

Maximum Active Pressure Resultant

$P_{A, \max}$

214.3 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2 \cdot P_A / H^2$

EFP

1.7 pcf

Design Shoring for an Equivalent Fluid Pressure:

25 pcf

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CHECKED BY: JTA

SHORING WALL - FAVORABLE BEDDING

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 1917-2005 1/2 WEST 3RD STREET
 LOS ANGELES, CALIFORNIA

JULY 2020

PROJECT NO. W1182-06-01

FIG. 9

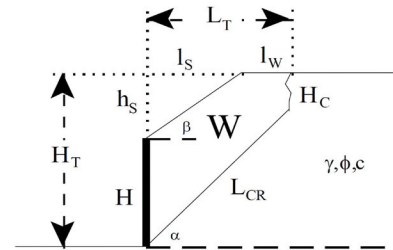
Shoring Design with Transitioned Backfill (Vector Analysis)

Input:

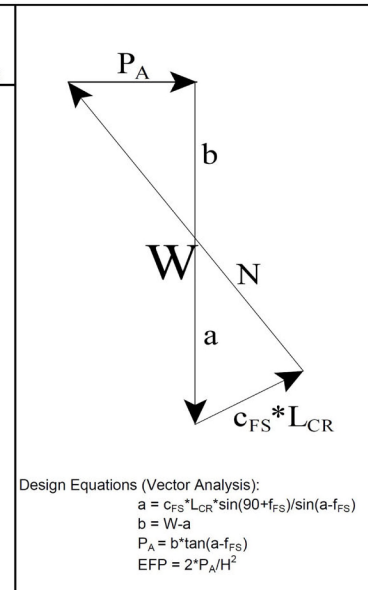
Shoring Height (H) **16.00** feet
 Slope Angle of Backfill (b) **0.0** degrees
 Height of Slope above Shoring (h_s) **0.0** feet
 Horizontal Length of Slope (l_s) **0.0** feet
 Total Height (Shoring + Slope) (H_T) **16.0** feet

Unit Weight of Retained Soils (g) **125.0** pcf
 Friction Angle of Retained Soils (f) **20.6** degrees
 Cohesion of Retained Soils (c) **262.0** psf
 Factor of Safety (FS) **1.25**

Factored Parameters: (f_{FS}) **16.7** degrees
 (c_{FS}) **209.6** psf



Failure Angle (a) degrees	Height of Tension Crack (Hc) 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
25	12.3	112	13946.0	8.7	12137.6	1808.4	262.7
26	11.1	136	17022.0	11.2	13942.7	3079.3	502.3
27	10.1	151	18853.4	13.0	14604.2	4249.2	769.5
28	9.3	159	19901.7	14.2	14641.9	5259.8	1047.6
29	8.6	164	20441.4	15.2	14339.1	6102.3	1326.5
30	8.1	165	20642.8	15.8	13855.0	6787.8	1600.1
31	7.6	165	20615.4	16.3	13281.5	7333.9	1864.5
32	7.2	163	20431.3	16.6	12672.2	7759.2	2117.4
33	6.8	161	20139.7	16.8	12058.4	8081.3	2357.6
34	6.5	158	19774.0	16.9	11458.1	8315.9	2584.4
35	6.3	155	19358.1	17.0	10881.6	8476.5	2797.4
36	6.0	151	18908.6	17.0	10333.9	8574.7	2996.7



Maximum Active Pressure Resultant

$P_{A, \max}$

2996.7 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

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

EFP

23.4 pcf

Design Shoring for an Equivalent Fluid Pressure:

25 pcf

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CHECKED BY: JTA

SHORING WALL - ADVERSE BEDDING

MIRAMAR DEVELOPMENT
 1917-2005 1/2 WEST 3RD STREET
 LOS ANGELES, CALIFORNIA

JULY 2020

PROJECT NO. W1182-06-01

FIG. 10

APPENDIX

A

APPENDIX A

FIELD INVESTIGATION

The site was explored on May 14 and May 15, 2020. Site exploration included excavating seven 3-inch diameter hand auger borings to depths between 5½ and 16 feet below existing ground surface, and one 24-inch diameter bucket auger boring to a depth of 20 feet below existing ground surface.

The soil conditions encountered in the explorations were visually examined, classified and logged in general accordance with the Unified Soil Classification System (USCS). The logs of the borings are presented on Figures A1 through A8. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained. The logs also include our interpretation of the conditions between sampling intervals. Therefore, the logs contain both observed and interpreted data. We determined the lines designating the interface between soil materials on the logs using visual observations, penetration rates, excavation characteristics and other factors. The transition between materials may be abrupt or gradual. Where applicable, the logs were revised based on subsequent laboratory testing. The location of the borings are shown on Figure 2A.

W1182-06-01 BORING LOGS.GPJ

SAMPLE SYMBOLS

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

GEOCON

W1182-06-01 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.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 3		PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					ELEV. (MSL.)	DATE COMPLETED			
					391	5/14/20			
					EQUIPMENT HAND AUGER BY: JMH				
					MATERIAL DESCRIPTION				
0					ARTIFICIAL FILL Sandy Silt, soft, slightly moist, olive brown.				
2	B3@1.5'			ML	COLLUVIUM Sandy Silt, firm, slightly moist, grayish brown.			80.4	16.0
4	B3@3'			CL	Clay, stiff, slightly moist, olive brown.			84.8	21.2
6	B3@5.5'				- blackish brown			94.5	19.4
8	B3@7.5'				PUENTE FORMATION Sandstone, olive brown, thinly bedded, highly weathered.			86.4	23.3
10	B3@10.5'				Siltstone with Sandstone interbeds, olive brown with oxidation mottles, thinly bedded, highly weathered.				
					Total depth of boring: 11 feet Fill to 1.5 feet. No groundwater encountered. Backfilled with soil cuttings and tamped.			97.4	26.1

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

W1182-06-01 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.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 4 ELEV. (MSL.) 397 DATE COMPLETED 5/14/20 EQUIPMENT HAND AUGER BY: JMH	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					MATERIAL DESCRIPTION			
2	B4@1.5'				ARTIFICIAL FILL Sandy Clay, firm, moist, dark brown. - ceramic debris - trace brick		85.5	27.8
4	B4@5'				- brick		84.0	20.2
					Refusal at 5.5 feet. All fill. No groundwater encountered. Backfilled with soil cuttings and tamped.			

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

W1182-06-01 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.

W1182-06-01 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.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 6 ELEV. (MSL.) 397 DATE COMPLETED 5/14/20 EQUIPMENT HAND AUGER BY: JMH	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					MATERIAL DESCRIPTION			
					ARTIFICIAL FILL Sandy Clay, soft, moist, dark grayish brown.			
2	B6@1.5'						75.5	17.4
					PUENTE FORMATION Siltstone with Sandstone interbeds, olive brown with oxidation mottles, thinly bedded, highly weathered.			
4	B6@3'						89.6	30.3
6	B6@5'						90.7	29.9
8	B6@7.5'						89.6	25.4
10	B6@10'				Sandstone with Siltstone interbeds, blackish brown, thinly bedded, highly weathered.		98.8	25.9
					Total depth of boring: 10.5 feet Fill to 2 feet. No groundwater encountered. Backfilled with soil cuttings and tamped.			

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

W1182-06-01 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.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 7		PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					ELEV. (MSL.) _____	DATE COMPLETED <u>5/15/2020</u>			
					EQUIPMENT <u>BUCKET AUGER</u> BY: <u>SK</u>				
					MATERIAL DESCRIPTION				
0					ASPHALT AC: 3" BASE: NONE ARTIFICIAL FILL Sand with Silt, yellowish brown, slightly moist, fine-grained, some gravel (to 1"), some oxidation staining on gravel clasts. Clay with Sand, dark grayish brown to black, dry to slightly moist, fine- to medium-grained, some oxidation staining on gravel clasts. - some coarse-grained, olive brown Silt with Sand, olive brown, dry to slightly moist, fine to medium-grained, trace clay, some brick fragments. - trace bedrock fragments (highly weathered sandstone with some oxidation staining), trace rootlets. - some clay pockets (to 4"), decrease in sand content				
2									
4									
6									
8					PUENTE FORMATION Silty Sandstone to Sandy Siltstone, light yellowish gray, slightly moist, fine-grained, massive, highly weathered, some oxidation staining. 7.4' - trace disseminated CaCO ₃ nodules 7.8' - poorly developed bedding defined by oxidation staining, bedding continuous around the hole, Bedding: N78W, 18SW				
10									
12					Interbedded Sandstone and Siltstone, yellowish brown and light gray, slightly moist, thinly bedded, some diatomaceous-rich beds, highly weathered, oxidation staining along bedding. 9.6' - Bedding: N90E, 41S 9.8' - Bedding: N86W, 34SW, bedding continuous around boring 11.5' - Bedding: N81W, 36SW 11.7' - Bedding: N90E, 31S				
14									
16					Silty Sandstone, grayish brown, slightly moist, thinly bedded to laminated (to 1mm thick) - Bedding: N74W, 31SW - Bedding: N86W, 33SW				
18									
20					Total depth of boring: 20 feet Fill to 6.5 feet. Groundwater not encountered. Backfilled with cuttings and tamped. Asphalt patched.				

Figure A7,
Log of Boring 7, Page 1 of 1

W1182-06-01 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.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 8		PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					ELEV. (MSL.)	DATE COMPLETED			
					<u>392</u>	<u>5/14/20</u>			
					EQUIPMENT <u>HAND AUGER</u> BY: <u>CB</u>				
					MATERIAL DESCRIPTION				
0	BULK 0-5'				ARTIFICIAL FILL Silty Sand, poorly graded, medium dense, dry, grayish brown, trace clay and rootlets.				
2	B8@1.5'				PUENTE FORMATION Siltstone, yellowish brown, highly weathered, oxidation staining, interbeds of			80.7	20.4
4	B8@3'				Sandy Siltstone, olive gray, highly weathered, moderately developed bedding.			69.1	28.0
6	B8@5'				Sandstone, gray, poorly cemented, massive, fine- to medium-grained.			74.5	42.0
8	B8@7'				Interbedded Siltstone and Sandstone, dark gray, thinly bedded, highly weathered.			74.4	43.6
10	B8@10'								
					Total depth of boring: 10.5 feet Fill to 1 foot. No groundwater encountered. Backfilled with soil cuttings and tamped.			84.3	31.2

Figure A8,
Log of Boring 8, Page 1 of 1

W1182-06-01 BORING LOGS.GPJ

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

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

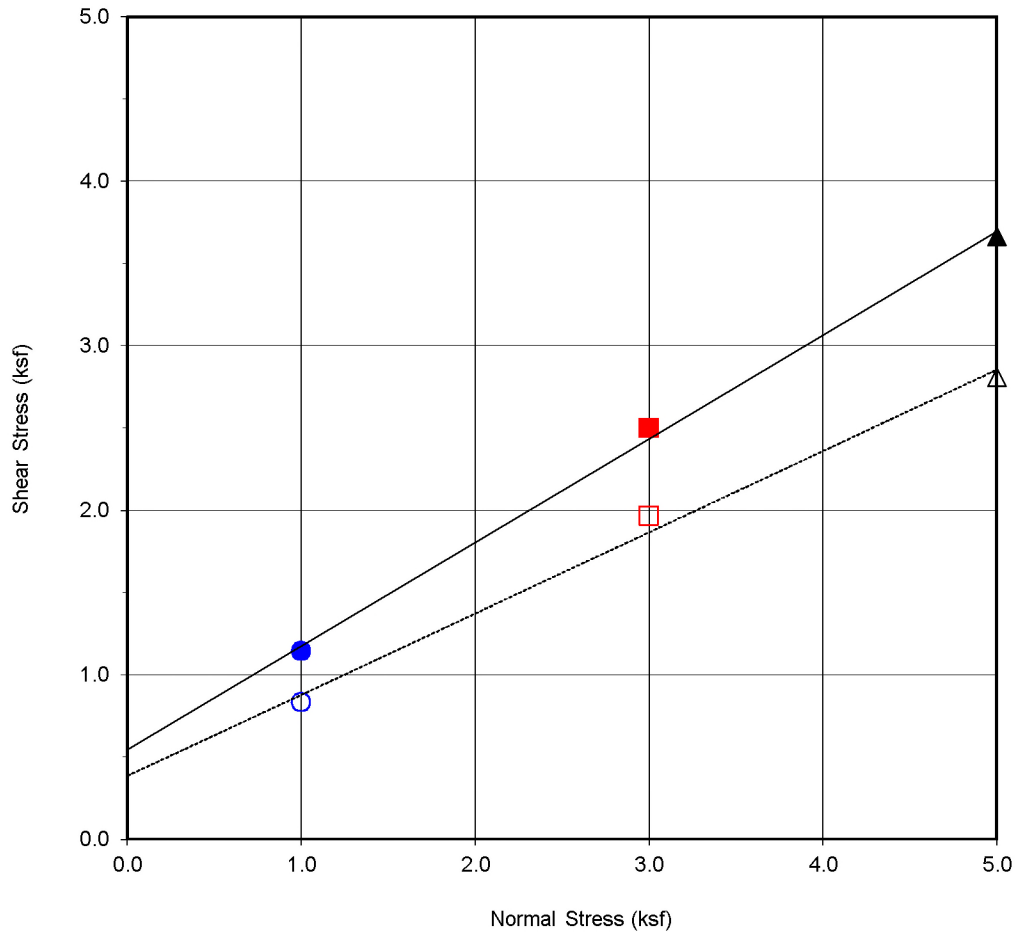
APPENDIX

B

APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the “American Society for Testing and Materials (ASTM)”, or other suggested procedures. Selected samples were tested for direct shear strength, consolidation and expansion characteristics, corrosivity, in-place dry density and moisture content. The results of the laboratory tests are summarized in Figures B1 through B12. The in-place dry density and moisture content of the samples tested are presented on the boring logs, Appendix A.



Boring No.	B2
Sample No.	B2@7.5'
Depth (ft)	7.5
<u>Sample Type:</u>	Ring

<u>Soil Identification:</u>		
Yelowish Brown Sandstone		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	543	32.2
Ultimate	386	26.3

Normal Strest (kip/ft ²)	1	3	5
Peak Shear Stress (kip/ft ²)	● 1.14	■ 2.50	▲ 3.66
Shear Stress @ End of Test (ksf)	○ 0.83	□ 1.96	△ 2.81
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	26.6	25.5	22.2
Initial Dry Density (pcf)	89.4	90.8	97.5
Initial Degree of Saturation (%)	81.2	80.4	82.1
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	34.9	31.9	28.0



DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

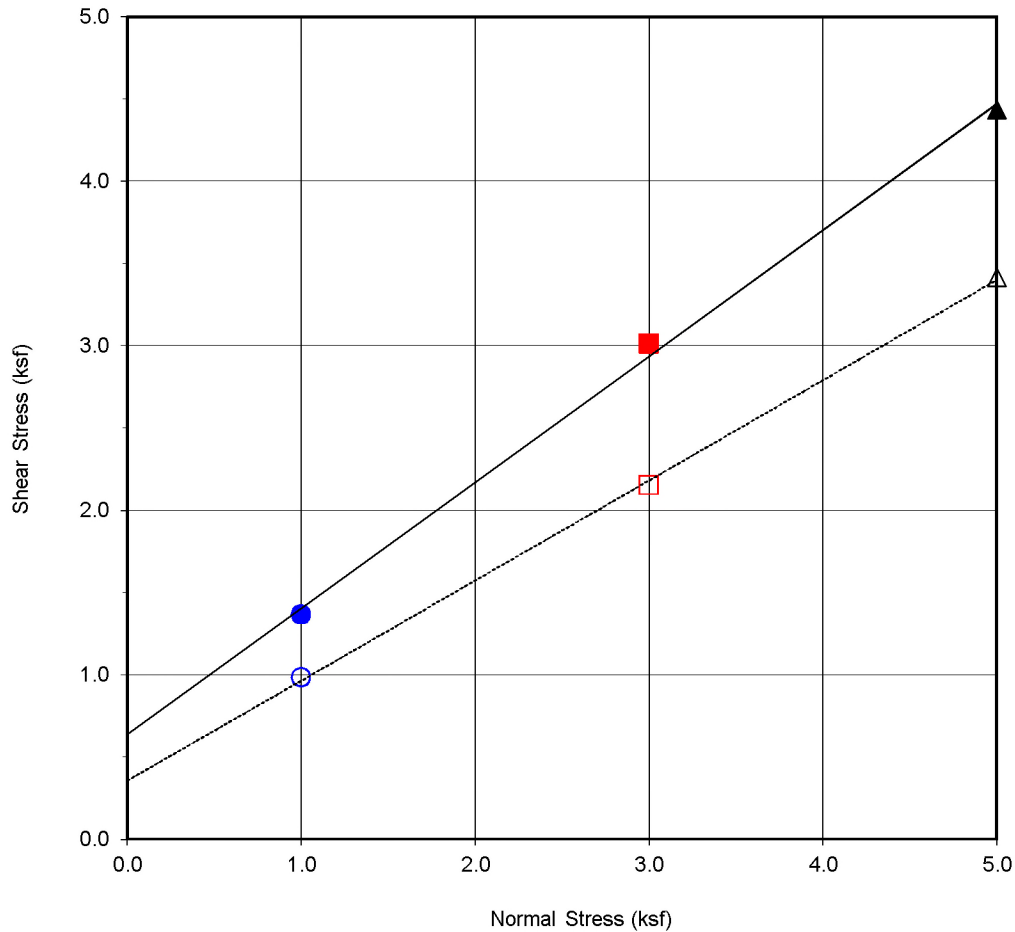
Checked by: JMH

Project No.: W1182-06-01

MIRAMAR DEVELOPMENT
1917-2005½ W 3RD STREET
LOS ANGELES, CA

JULY 2020

Figure B1



Boring No.	B3
Sample No.	B3@7.5'
Depth (ft)	7.5
Sample Type:	Ring

Soil Identification:		
Olive Brown Sandstone		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	636	37.5
Ultimate	356	31.3

Normal Stress (kip/ft ²)	1	3	5
Peak Shear Stress (kip/ft ²)	● 1.37	■ 3.01	▲ 4.43
Shear Stress @ End of Test (ksf)	○ 0.98	□ 2.15	△ 3.42
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	25.6	25.4	25.3
Initial Dry Density (pcf)	88.6	89.1	92.2
Initial Degree of Saturation (%)	76.7	76.8	82.7
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	34.5	33.7	31.5



DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

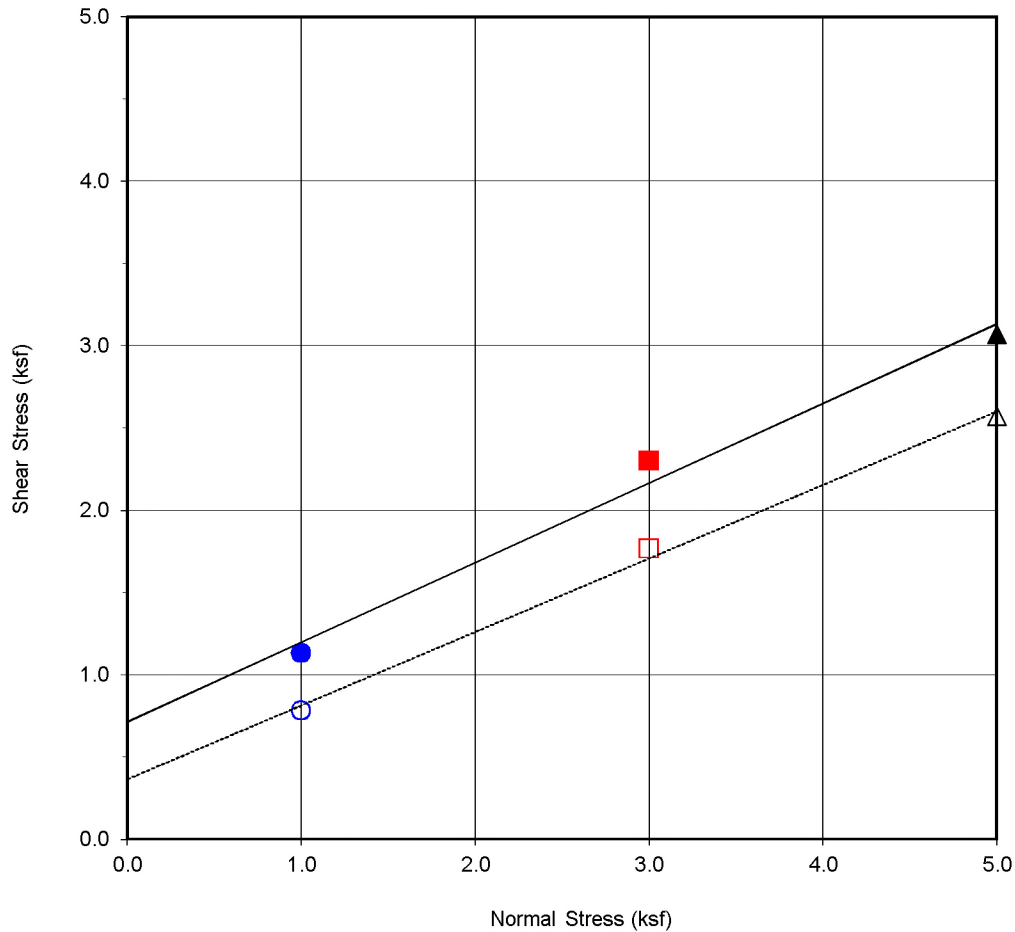
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MIRAMAR DEVELOPMENT
1917-2005½ W 3RD STREET
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Figure B2



Boring No.	B8
Sample No.	B8@3'
Depth (ft)	3
Sample Type:	Ring

Soil Identification:		
Yellowish Brown Siltstone		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	712	25.8
Ultimate	364	24.1

Normal Stress (kip/ft ²)	1	3	5
Peak Shear Stress (kip/ft ²)	● 1.13	■ 2.30	▲ 3.07
Shear Stress @ End of Test (ksf)	○ 0.78	□ 1.76	△ 2.57
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	31.8	30.7	32.7
Initial Dry Density (pcf)	74.1	78.1	80.4
Initial Degree of Saturation (%)	67.5	71.6	80.3
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	46.9	42.2	41.0



DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

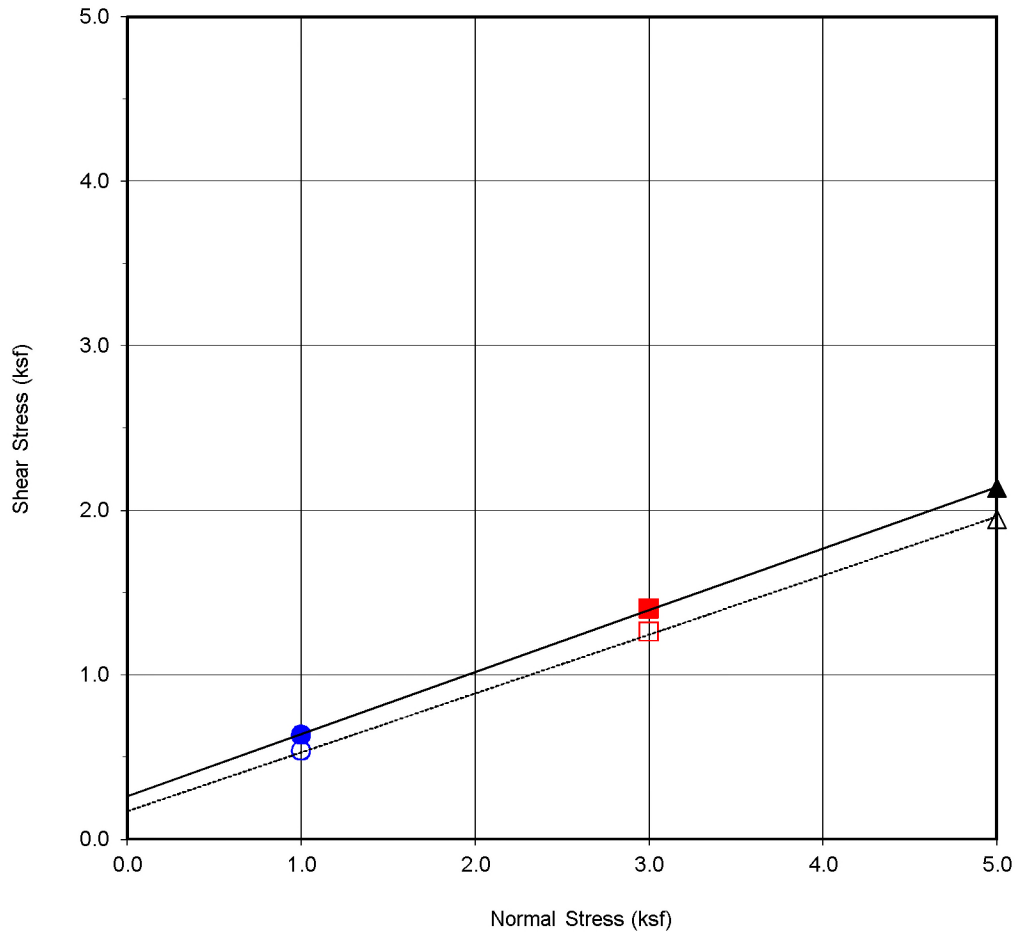
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Project No.: W1182-06-01

MIRAMAR DEVELOPMENT
1917-2005½ W 3RD STREET
LOS ANGELES, CA

JULY 2020

Figure B3



ORIENTED SAMPLE ALONG BEDDING PLANE; SATURATED

Boring No.	B7
Sample No.	B7@9.5'
Depth (ft)	9.5'
Sample Type:	Ring

Soil Identification:		
Yellowish Brown Siltstone		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	262	20.6
Ultimate	170	19.7

Normal Stress (kip/ft ²)	1	3	5
Peak Shear Stress (kip/ft ²)	0.63	1.40	2.14
Shear Stress @ End of Test (ksf)	0.53	1.26	1.94
Deformation Rate (in./min.)	0.01	0.00	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	33.3	30.4	27.2
Initial Dry Density (pcf)	79.2	80.5	77.3
Initial Degree of Saturation (%)	79.7	75.0	62.2
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	42.7	37.5	37.4



DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

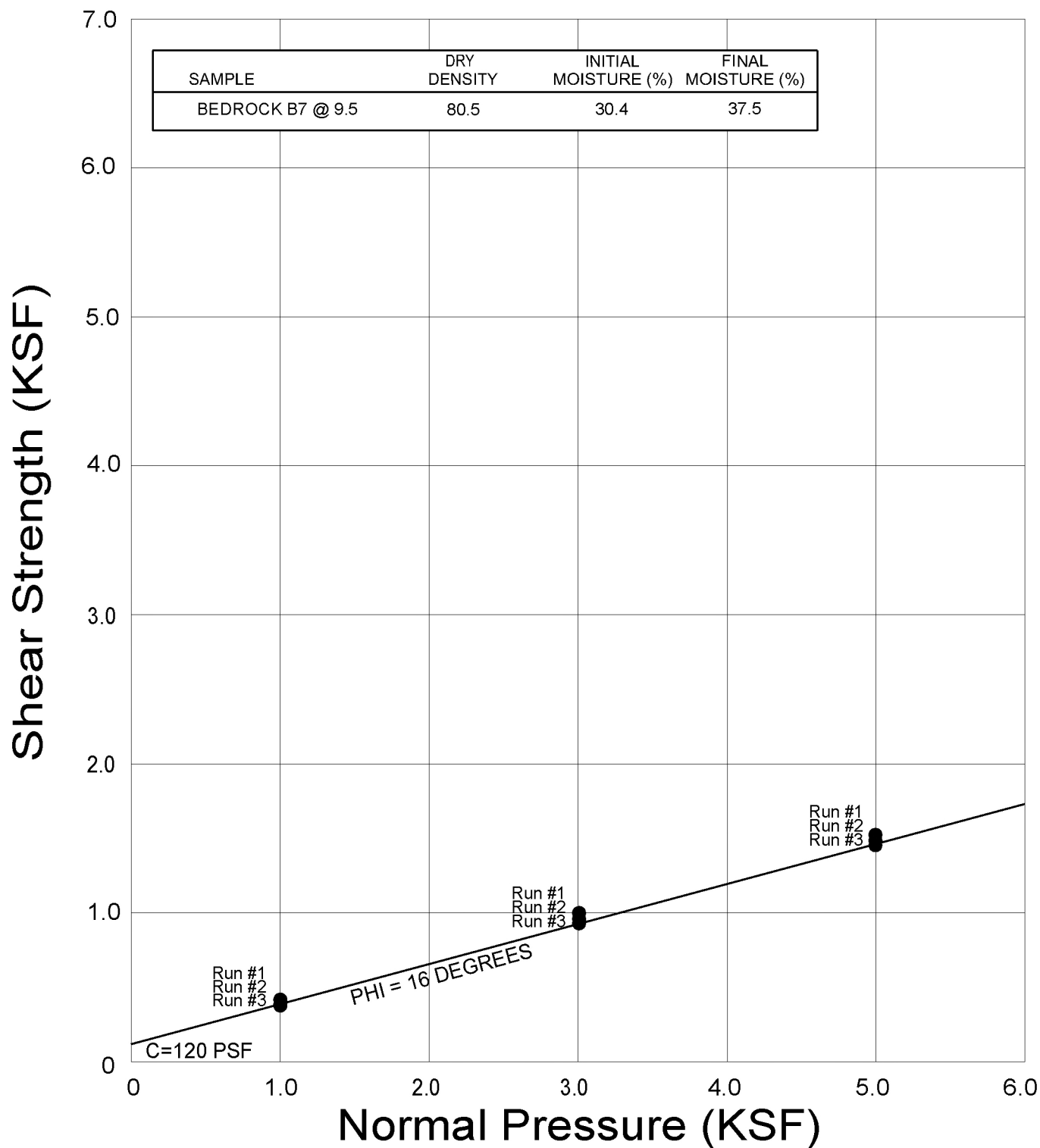
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Project No.: W1182-06-01

MIRAMAR DEVELOPMENT
1917-2005½ W 3RD STREET
LOS ANGELES, CA

JULY 2020

Figure B4



● Direct Shear, Saturated, Residual Values, Along Bedding

RATE OF DEFORMATION: 0.005 INCHES PER MINUTE

GEOCON
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ENVIRONMENTAL GEOTECHNICAL MATERIALS
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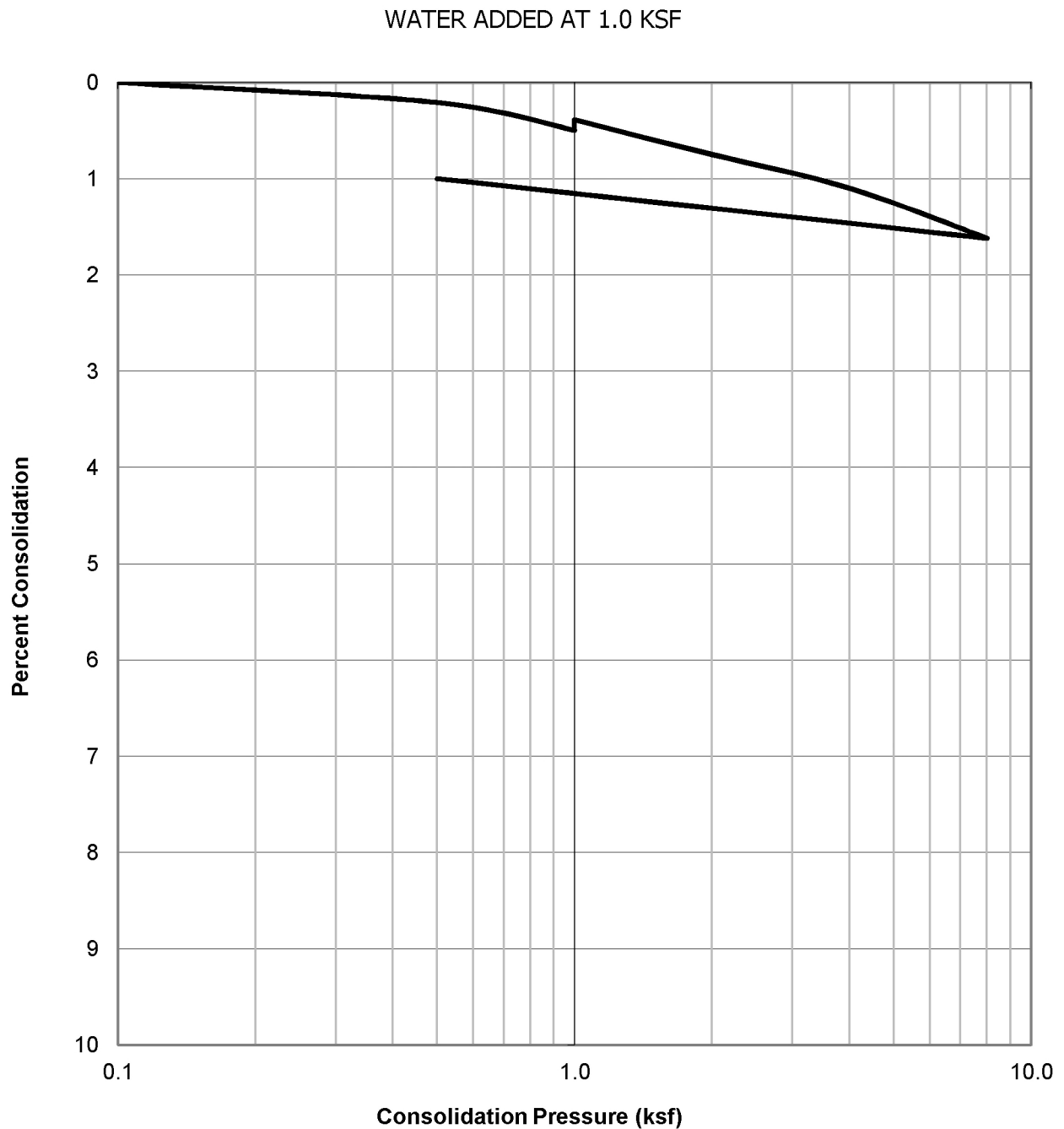
DIRECT SHEAR TEST RESULTS

MIRAMAR DEVELOPMENT
1917-2005½ WEST 3RD STREET
LOS ANGELES, CALIFORNIA

JULY 2020

PROJECT NO. W1182-06-01

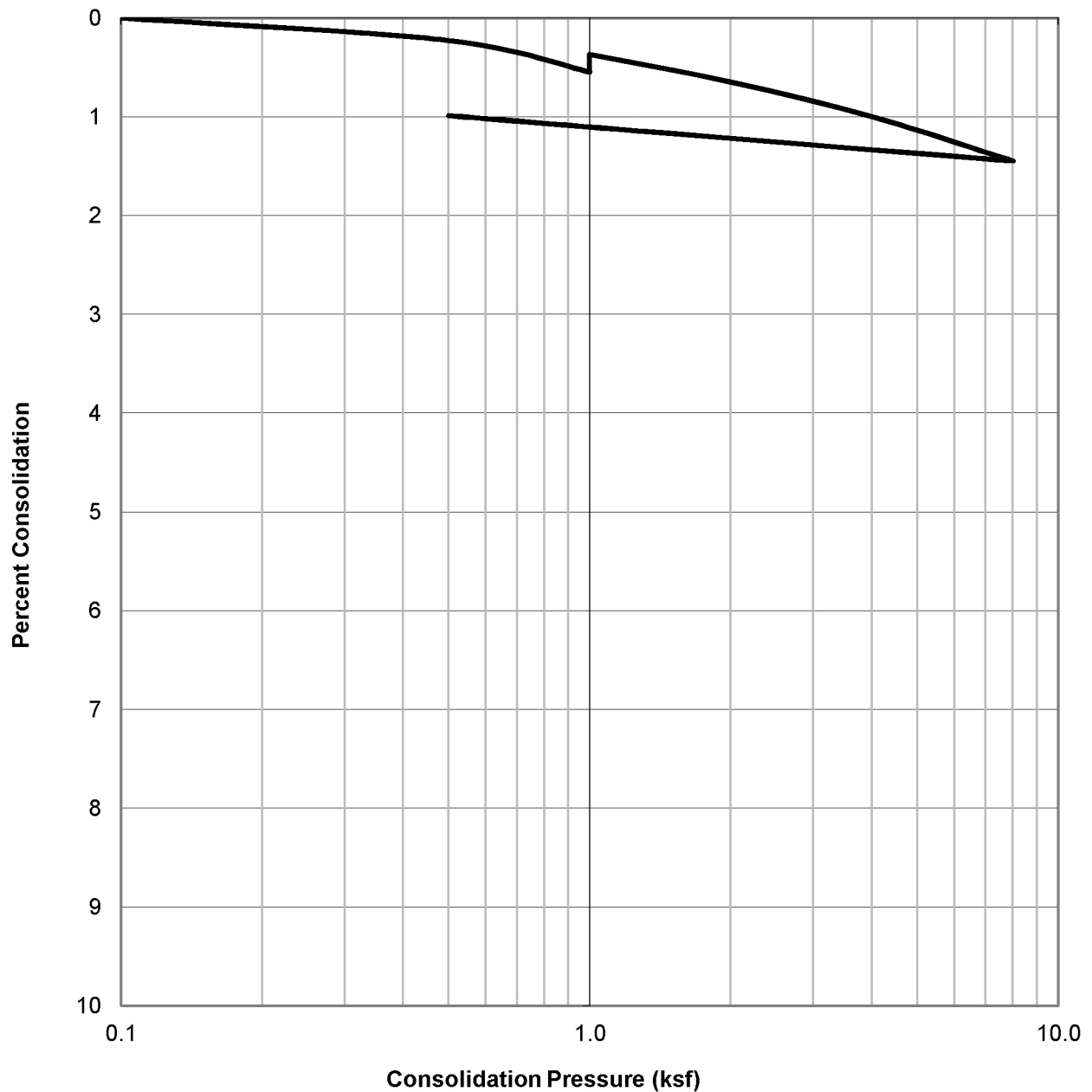
FIG. B5



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@3	Olive Brown Siltstone	85.4	32.6	34.3

	CONSOLIDATION TEST RESULTS ASTM D-2435	Project No.: W1182-06-01
		MIRAMAR DEVELOPMENT 1917-2005½ W 3RD STREET LOS ANGELES, CA
	Checked by: JMH	JULY 2020 Figure B6

WATER ADDED AT 1.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@5	Olive Brown Siltstone	91.7	29.9	30.9



CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JMH

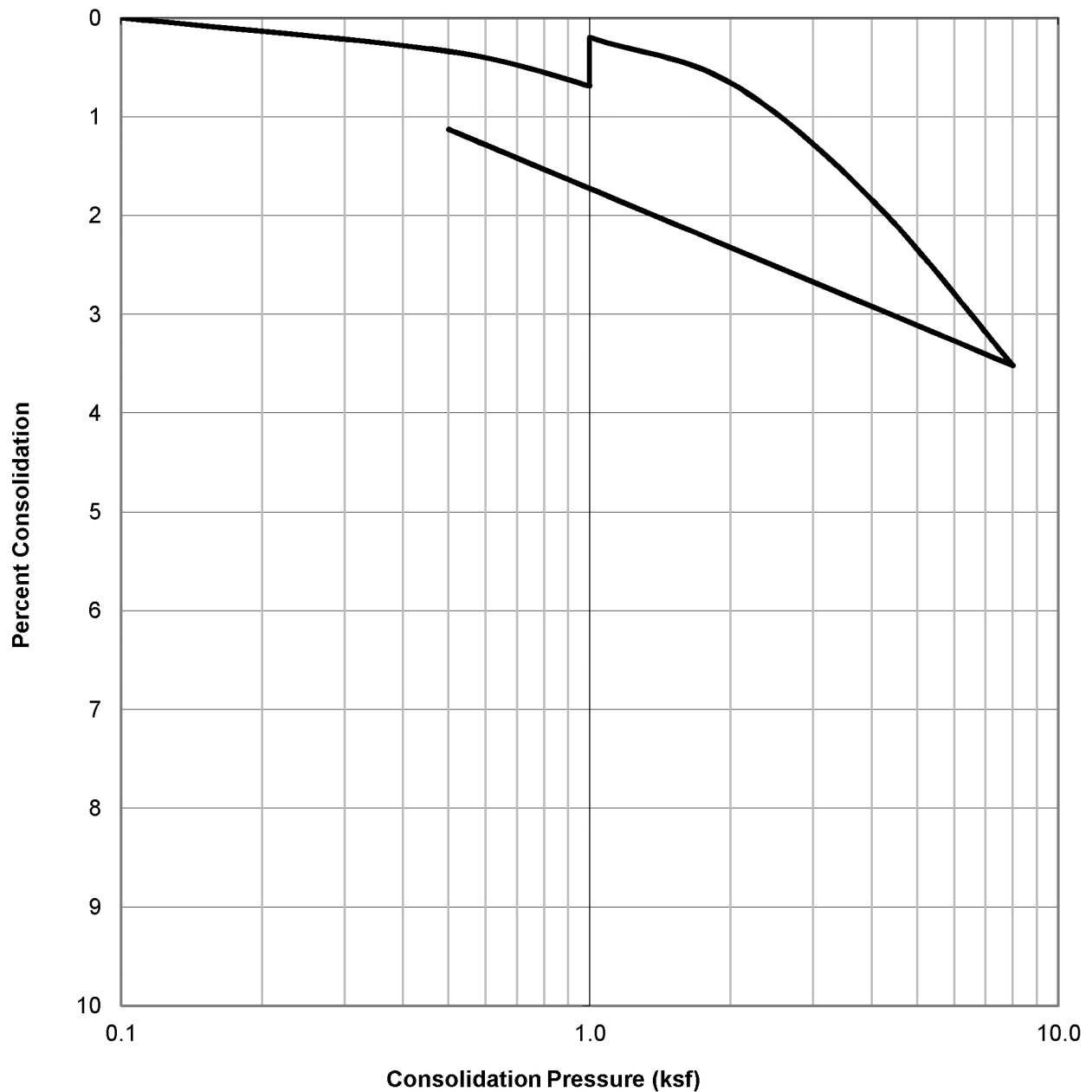
Project No.: W1182-06-01

MIRAMAR DEVELOPMENT
1917-2005½ W 3RD STREET
LOS ANGELES, CA

JULY 2020

Figure B7

WATER ADDED AT 1.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@3	Dark Grayish Brown Clay (CL)	96.1	20.8	26.5



CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JMH

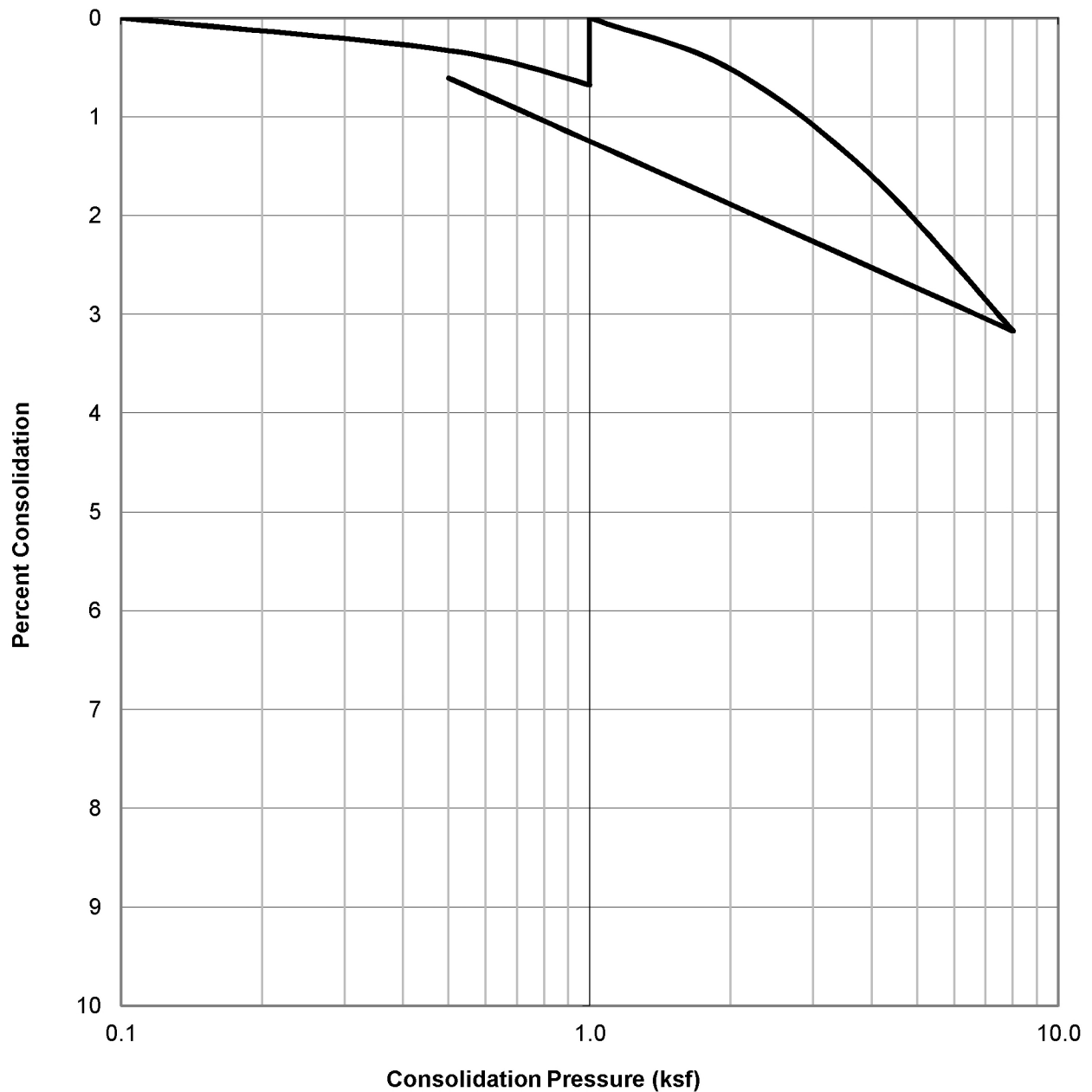
Project No.: W1182-06-01

MIRAMAR DEVELOPMENT
1917-2005½ W 3RD STREET
LOS ANGELES, CA

JULY 2020

Figure B8

WATER ADDED AT 1.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@5.5	Dark Grayish Brown Clay (CL)	101.7	19.8	23.6



CONSOLIDATION TEST RESULTS
ASTM D-2435

Checked by: JMH

Project No.: W1182-06-01
MIRAMAR DEVELOPMENT
1917-2005½ W 3RD STREET
LOS ANGELES, CA
JULY 2020 Figure B9

B8@0-5'

MOLDED SPECIMEN		BEFORE TEST	AFTER TEST
Specimen Diameter	(in.)	4.0	4.0
Specimen Height	(in.)	1.0	1.1
Wt. Comp. Soil + Mold	(gm)	703.4	771.2
Wt. of Mold	(gm)	368.4	368.4
Specific Gravity	(Assumed)	2.7	2.7
Wet Wt. of Soil + Cont.	(gm)	498.5	771.2
Dry Wt. of Soil + Cont.	(gm)	455.3	286.8
Wt. of Container	(gm)	198.5	368.4
Moisture Content	(%)	16.8	40.4
Wet Density	(pcf)	101.1	121.3
Dry Density	(pcf)	86.5	86.4
Void Ratio		0.9	1.2
Total Porosity		0.5	0.5
Pore Volume	(cc)	100.8	123.1
Degree of Saturation	(%) [S_{meas}]	48.2	94.2

Date	Time	Pressure (psi)	Elapsed Time (min)	Dial Readings (in.)
5/21/2020	10:00	1.0	0	0.2795
5/21/2020	10:10	1.0	10	0.279
Add Distilled Water to the Specimen				
5/22/2020	10:00	1.0	1430	0.387
5/22/2020	11:00	1.0	1490	0.387

Expansion Index (EI meas) =	108
Expansion Index (Report) =	108

Expansion Index, EI_{50}	CBC CLASSIFICATION *	UBC CLASSIFICATION **
0-20	Non-Expansive	Very Low
21-50	Expansive	Low
51-90	Expansive	Medium
91-130	Expansive	High
>130	Expansive	Very High

* Reference: 2019 California Building Code, Section 1803.5.3

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

 GEOCON	EXPANSION INDEX TEST RESULTS ASTM D-4829	Project No.: W1182-06-01
		MIRAMAR DEVELOPMENT 1917-2005½ W 3RD STREET LOS ANGELES, CA
	Checked by: JMH	JULY 2020 Figure B10

B1@3'

MOLDED SPECIMEN		BEFORE TEST	AFTER TEST
Specimen Diameter	(in.)	2.4	2.4
Specimen Height	(in.)	1.0	1.0
Wt. Comp. Soil + Mold	(gm)	201.8	204.5
Wt. of Mold	(gm)	65.2	65.2
Specific Gravity	(Assumed)	2.7	2.7
Wet Wt. of Soil + Cont.	(gm)	363.4	204.5
Dry Wt. of Soil + Cont.	(gm)	319.8	116.7
Wt. of Container	(gm)	63.4	65.2
Moisture Content	(%)	17.0	19.3
Wet Density	(pcf)	117.4	70.8
Dry Density	(pcf)	100.4	59.4
Void Ratio		0.7	0.7
Total Porosity		0.4	0.4
Pore Volume	(cc)	29.4	29.1
Degree of Saturation	(%) [S_{meas}]	68.0	77.6

Date	Time	Pressure (psi)	Elapsed Time (min)	Dial Readings (in.)
5/21/2020	10:00	0.4	0	0.499
5/21/2020	10:10	0.4	10	0.51
Add Distilled Water to the Specimen				
5/22/2020	10:00	0.4	1430	0.546
5/22/2020	11:00	0.4	1490	0.546

Expansion Index (EI meas) =	36
Expansion Index (Report) =	36

Expansion Index, EI_{50}	CBC CLASSIFICATION *	UBC CLASSIFICATION **
0-20	Non-Expansive	Very Low
21-50	Expansive	Low
51-90	Expansive	Medium
91-130	Expansive	High
>130	Expansive	Very High

* Reference: 2019 California Building Code, Section 1803.5.3

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

	EXPANSION INDEX TEST RESULTS ASTM D-4829	Project No.: W1182-06-01
		MIRAMAR DEVELOPMENT 1917-2005½ W 3RD STREET LOS ANGELES, CA
	Checked by: JMH	JULY 2020 Figure B11

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

Sample No.	pH	Resistivity (ohm centimeters)
B1 @ 0-5'	6.5	650 (Severely Corrosive)
B8 @ 0-5'	5.6	430 (Severely Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS
EPA NO. 325.3

Sample No.	Chloride Ion Content (%)
B1@0-5'	0.024
B8@0-5'	0.042

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

Sample No.	Water Soluble Sulfate (% SQ ₄)	Sulfate Exposure*
B1@0-5'	0.090	S0
B8@0-5'	0.647	S2

	CORROSIVITY TEST RESULTS		Project No.: W1182-06-01
			MIRAMAR DEVELOPMENT 1917-2005½ W 3RD STREET LOS ANGELES, CA
	Checked by: JMH	JULY 2020	Figure B12